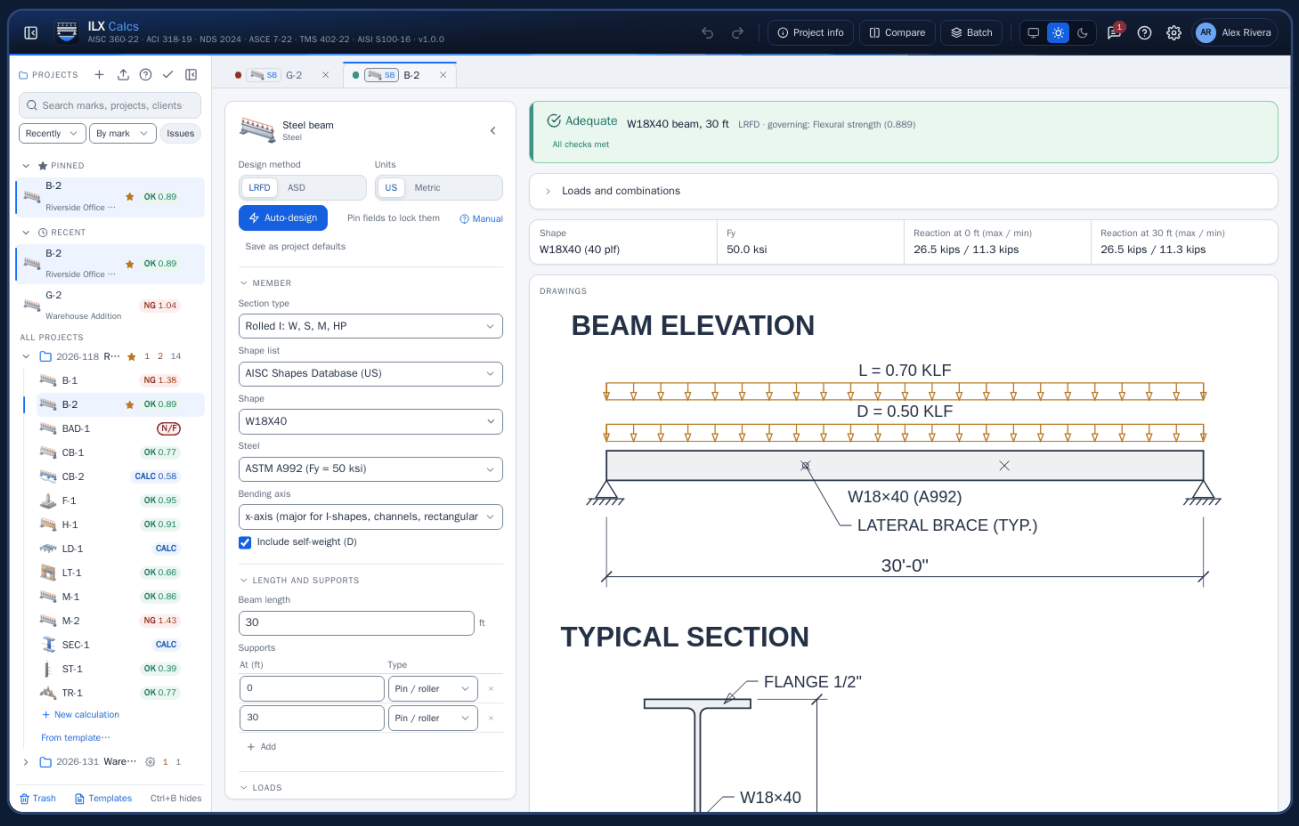
ILX Calcs

STRUCTURAL CALCULATIONS, STEP BY STEP

User Manual

Every calculation type, step by step: inputs, checks, drawings, calculation reports and settings for ILX Calcs v1.0.0.



ILX Studio, LLC · services@ilxstudio.com

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ILX Calcs User Manual

Version 1.0.0, November 2026. Covers the ILX Calcs web application: structural calculations for steel, wood, concrete, foundations, masonry and cold-formed steel, with loads and analysis tools. Each chapter states its code editions and how far it has been verified.

Responsibility

ILX Calcs helps you through the design and documents every check, but the engineer of record reviews and approves the results. See the full disclaimer at https://ilxcalcs.com/legal/disclaimer.

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CHAPTER 1

Overview

ILX Calcs does structural calculations on the web: documented member and element calculations for steel, wood, concrete, foundations, masonry and cold-formed steel, and the loads and analysis tools around them. Every calculation shows its equations with the code clause, the governing load combination and a status for each check, with diagrams, a calculation report and, for most types, a drawing sheet.

You are the engineer of record. ILX Calcs applies the code provisions as written; where a provision leaves a choice to the engineer (a deflection limit, a load factor option, a bracing assumption), the choice is an input with its usual value prefilled, never a hidden assumption. How far each type has been checked is stated plainly in its chapter, under **Verification examples**.

New here: training

Short walk-throughs with real inputs and the results the app gives.

Training	What you learn
1. Getting started	One steel beam, start to report
2. Projects, settings, title block and output	Your firm on every report and sheet
3. Reading checks, steps and clauses	How to read a calculation that fails
4. The Wizard and auto-design	Picture cards, and letting the search propose a member
5. Units, and shapes from other markets	Metric, and European shapes on AISC 360
6. Steel, 7. Wood, 8. Concrete, 9. Foundations, 10. Masonry, 11. Cold-formed steel, 12. Loads, 13. Utilities	One worked calculation per subject
14. A small project	From floor load to footing, chaining calculations
15. The section builder	Properties of any section, built up from parts
16. The truss designer	Layout, automatic sizing, and its preview label
17. The verification report and deviations	What stands behind a result
18. Limits and your responsibility	What stays with you

Using the app

Getting started, Projects and files, Reports and settings, Verification, Keyboard shortcuts and troubleshooting.

Calculation types

This version has 230 types. The New calculation window lists the ones this server has.

Steel

- Steel beam
- Steel column
- Column base plate
- Composite beam
- Steel section selection
- Steel beam with torsion
- Eccentric bolt group
- Eccentric weld group
- Member stability bracing
- Steel joist
- Steel deck (gravity)
- Steel deck diaphragm
- Steel truss
- Concrete-filled HSS column
- Steel lintel
- Floor vibration
- Steel stair
- Beam with web openings
- Crane runway beam
- Castellated or cellular beam
- Guard or barrier post
- Monorail beam
- Steel tension member / brace
- Concentrated forces on beams
- Steel fire resistance
- Roof ponding
- Seismic member checks
- Encased composite column
- Plate girder
- Web-tapered member

Wood

- Sawn lumber beam
- Glulam and SCL beam
- Wood column and post
- Wood stud wall
- CLT panel
- Wood shear wall

- Wood diaphragm
- Wood fastener connection
- Wood bearing and notches
- CLT shear wall
- Continuous rod tie-down
- Wood ledger
- Flitch-plate beam
- Wood stair
- Wood I-joist
- Rafters and ceiling ties
- Wood truss
- Fire design of exposed wood
- Wood deck framing
- Split-ring and shear-plate connections
- Wood design values
- Wood shrinkage and moisture
- Steel side and knife plate connections
- Moment and eccentric fastener groups
- Timber rivet connections
- Notched and tapered member ends
- Wood floor vibration
- Built-up and spaced columns
- Round timber poles and piles
- Curved and tapered glulam

Concrete

- Concrete beam
- Concrete column
- One-way slab
- Two-way slab
- Concrete shear wall
- Punching shear
- Anchors in concrete
- Post-installed anchors
- Rebar development and splices
- Beam-column joint
- Post-tensioned slab strip
- Shear friction
- Embed plate
- Concrete diaphragm
- Deep beam (strut-and-tie)
- Corbel
- Tilt-up wall panel

- Concrete stair
- Coupling beam
- Dapped-end beam
- Bearing on concrete
- Joist and waffle slab
- Continuous concrete beam
- Prestressed concrete beam
- Special moment frame members
- Ledge and inverted-tee beam
- Hollow-core plank
- Concrete deflection
- Liquid-containing tank wall
- Concrete fire resistance

Foundations

- Spread footing
- Wall footing
- Pole footing
- Combined footing
- Strap footing
- Mat foundation
- Pile cap
- Pile group
- Laterally loaded pile
- Beam on elastic foundation
- Slab on grade
- Retaining wall
- Basement (restrained) retaining wall
- Gravity retaining wall
- Segmental retaining wall
- Gabion retaining wall
- Sheet pile wall
- Soldier pile wall
- Bearing capacity and settlement
- Pile axial capacity
- Laterally loaded pile (p-y)
- Drilled pier
- Helical pile
- Pile capacity from CPT
- Braced excavation
- Slope stability
- Liquefaction triggering
- Ground improvement

- Micropile
- Ground anchor

Masonry

- Masonry shear wall
- Masonry lintel and beam
- Masonry column and pilaster
- Masonry wall out-of-plane
- Masonry pilaster with wall
- Masonry bearing
- Masonry anchor bolts
- Masonry wall at openings
- Anchored veneer
- Adhered veneer
- Masonry lap splice
- Masonry retaining wall
- Unreinforced masonry wall
- Masonry infill
- Prestressed masonry wall
- AAC masonry wall
- Glass unit masonry
- Freestanding masonry wall
- Masonry fire resistance
- Masonry strength f′m
- Empirical masonry design
- Corbelled masonry
- Masonry parapet
- Ledger to masonry wall
- Masonry arch
- Bond beam chord and collector
- Movement joints
- Masonry partition wall
- Masonry chimney
- Masonry beam and deep beam

Cold-formed steel

- Cold-formed steel stud
- Cold-formed steel shear wall
- Cold-formed steel joist
- Cold-formed steel header
- Cold-formed steel fastener connection
- Cold-formed steel rafter
- Cold-formed steel curtain wall

- Cold-formed steel bridging
- Cold-formed steel strap braced wall
- Cold-formed steel purlin
- Cold-formed steel truss
- Cold-formed steel clip angle
- Cold-formed steel diaphragm
- Cold-formed steel parapet
- Cold-formed steel ceiling framing
- Cold-formed steel girt
- Built-up cold-formed steel member
- Cold-formed steel jamb studs
- Cold-formed steel track
- Web holes in cold-formed members
- Cold-formed steel track anchorage
- Cold-formed steel floor vibration
- Cold-formed steel soffit framing
- Cold-formed steel bypass and infill framing
- Sheathing-braced cold-formed steel stud
- Cold-formed steel special bolted moment frame
- Cold-formed steel shear wall chord studs
- Cold-formed steel truss bracing
- Cold-formed steel storage rack
- Cold-formed steel sloped and mansard wall

Loads calculations

- Wind loads
- Seismic base shear (ELF)
- Load takeoff
- Snow loads
- Diaphragm forces
- Rain load
- Flood loads
- Lateral pressure from surface loads
- Tornado loads
- Story drift and P-delta
- Seismic site parameters and spectrum
- Wind loads, envelope procedure
- Wind on other structures
- Rigid diaphragm torsion
- Seismic nonstructural components
- Seismic wall anchorage
- Live load reduction
- Atmospheric ice loads

- Simplified seismic design
- Nonbuilding structures (seismic)
- Modal response spectrum
- Gust-effect factor
- Impact and machinery loads
- Fall-protection anchorage
- Fluid and hydrostatic pressure

Utilities

- Continuous beam analysis
- 2-D frame analysis
- 2-D truss analysis
- Section properties
- Load chasedown
- Load combinations
- Wall and pier rigidity
- Plate bending
- Effective length factor
- Moving load
- Dead load build-up
- Unit converter
- Beam with varying stiffness
- Cable and catenary
- Natural frequency and dynamic response
- Stress transformation
- Custom calculation
- Thermal movement
- Quantity takeoff
- Rebar tables
- Roof geometry
- Shape browser and comparison
- Influence lines
- Arch analysis
- Beam grid (grillage)

Code basis

The target editions: AISC 360-22 and the AISC Manual 16th Ed.; ACI 318-19; NDS 2024 and SDPWS 2021; TMS 402/602-22; AISI S100-16 and S400-20; ASCE/SEI 7-22; IBC 2021 for the deflection limits you enter. Not every module has been compared line by line with its target edition: each chapter says what its module rests on, and Verification explains how to read that.

CHAPTER 2

Getting started

New to the app? Training 1 does this with a real beam, and the other training chapters follow it. This chapter is the reference.

First steps

1. Sign in. A project called *My first project* with an example calculation opens the first time.
2. Create a project with **+** in the project panel, or use the example project.
3. Choose **New calculation** under a project. The window has two steps:
   - **Calculation type**: every type has a card with a picture of what it calculates. The tabs narrow the cards to one material group (Steel, Wood, Concrete, Foundations, Masonry, Cold-formed steel, Loads, Utilities); the search box finds a type by name or description. Use the arrow keys to move between cards and **Enter** (or a double click, or **Next**) to continue.
   - **Configure**: the main options of the type as picture cards (for example the span condition and the bracing of a steel beam, the number of piles of a pile cap, a square or rectangular footing), the key inputs (member, span, main loads) in the chosen units, the mark (for example `B-1`), the project, the units and the design method. The **Preview** on the right designs the calculation as you change it and shows the status, the governing check with its ratio and the type's drawing. Where the server can auto-design the type, **Auto-design after create** lets the optimizer choose the member (or plate, footing size) for these loads. Some types spread over several pages: the truss designers (layout, geometry, loads, members and bracing). The cards are presets: each sets several inputs, such as the supports of an overhang, and the editor takes any other layout. Select **Create**. Every other input starts from the defaults (and the project defaults) and is set in the editor. **Escape** closes the window without creating anything.
4. Enter the inputs on the left. The results on the right update as you type; every change is saved automatically.

The editor

The editor has two columns:

- **Inputs** (left): the design method (LRFD or ASD), the units (US or Metric) and the calculation's sections. Select a section title to fold it. **Manual** opens this manual at the calculation's chapter. **Save as project defaults** stores the materials, methods and limits of this calculation as the starting values for new ones of the same type in this project.
- **Results** (right): the status, the loads and combinations, key results, diagrams, the report and sheet buttons, and the checks.

Switching units converts every value; nothing is lost going back and forth. A calculation started in **Metric** begins from round metric values, and the steel beam and steel column also take shapes from other markets: see Training 5.

Loads and combinations

Loads are entered as service (unfactored) values, each with its load type: D (dead), L (live), Lr (roof live), S (snow), R (rain), W (wind) or E (earthquake). ILX Calcs builds every ASCE 7-22 combination of the load types you entered, 2.3.1 and 2.3.6 for LRFD and 2.4.1 and 2.4.5 for ASD, with wind and earthquake acting in both directions, designs for each and reports the governing one for every check. Snow takes the ASCE 7-22 factors (the 7-22 snow loads are strength level): 1.0S as the principal load and 0.3S or 0.15S as a companion in LRFD, 0.7S, 0.525S and 0.1S in ASD. E is the horizontal seismic effect Eh = ρQE. The vertical seismic effect Ev = 0.2SDS D is applied by the modules that ask for SDS (shear walls, masonry wall); elsewhere it is not modeled, nor is the overstrength factor Ω0 unless a module asks for it. The load factor on L is 1.0 (the 0.5L option of 2.3.1 and 2.3.6 is not used). **Loads and combinations** in the results lists what was evaluated.

Reading the checks



The status at the top is **Adequate** when every check is met, **Not adequate** when one is not, and **Not feasible** when the inputs describe something that cannot be calculated (the banner says why, and **Show** takes you to the input).

Each check shows the required value, the available value, their ratio and the load combination that governs it. Open a check to see every equation with its values and the code clause; **Read more** summarizes the clause. **Code compliance** lists the limits and recommendations that are not strength checks.

Auto-designing a calculation

Auto-design (in the editor toolbar of the calculation types that have it) searches the calculation's design variables for the lightest or most economical input that passes every check, using the calculation's own checks for every candidate. It proposes; you decide.

1. Choose the **objective**: for example the lightest member, the shallowest member, the least concrete or the lowest cost index (relative unit costs of concrete, formwork, reinforcement, steel and lumber, meant to rank the alternatives, not to estimate a job).
2. Choose what may change. Each variable is listed with a check box; clear it to keep the value as entered. A value pinned in the input panel (the pin next to the label) is kept too. Material grades start kept: select them to let auto-design choose the material. Options such as a depth limit narrow the candidates.
3. Select **Run**. The table lists the best design and up to five alternatives, each with the inputs it changes (old and new value), the quantity of the objective, the governing check and its ratio.
4. Select **Apply** on a row to put its values into the form. Nothing changes until you apply, and **Undo** (Ctrl+Z) takes the change back. Review the checks of the applied design as for any calculation.

Every candidate keeps all the other inputs (loads, spans, supports, code options) exactly as entered. The search starts from the cheapest candidates, skips the ones that cannot pass (closed-form bounds from the input as entered, and candidates weaker in every respect than one that already failed a strength check), and stops after the best design and five alternatives are found or at its time limit (a few seconds). When no candidate passes, it shows the

closest one and the checks it does not meet: relax a locked value, widen the options or change the layout. The chapter of each calculation says what its auto-design varies. In **Batch design**, *Auto-design each mark* runs the same search for every row and keeps the values given in the grid.

Diagrams

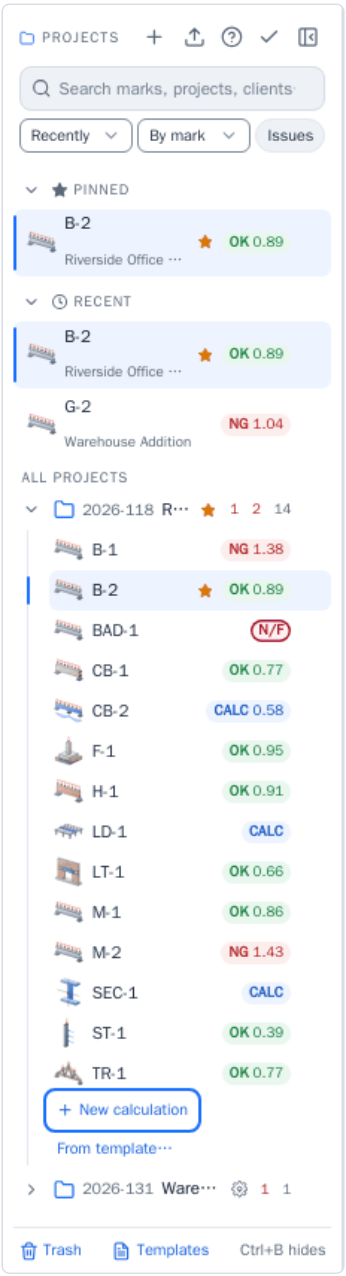
Calculations that analyze a member draw the loading with the supports, and the shear, moment and deflection diagrams of the governing combination, with the extreme values marked. Moment diagrams are drawn on the tension side; deflections are drawn downward.

CHAPTER 3

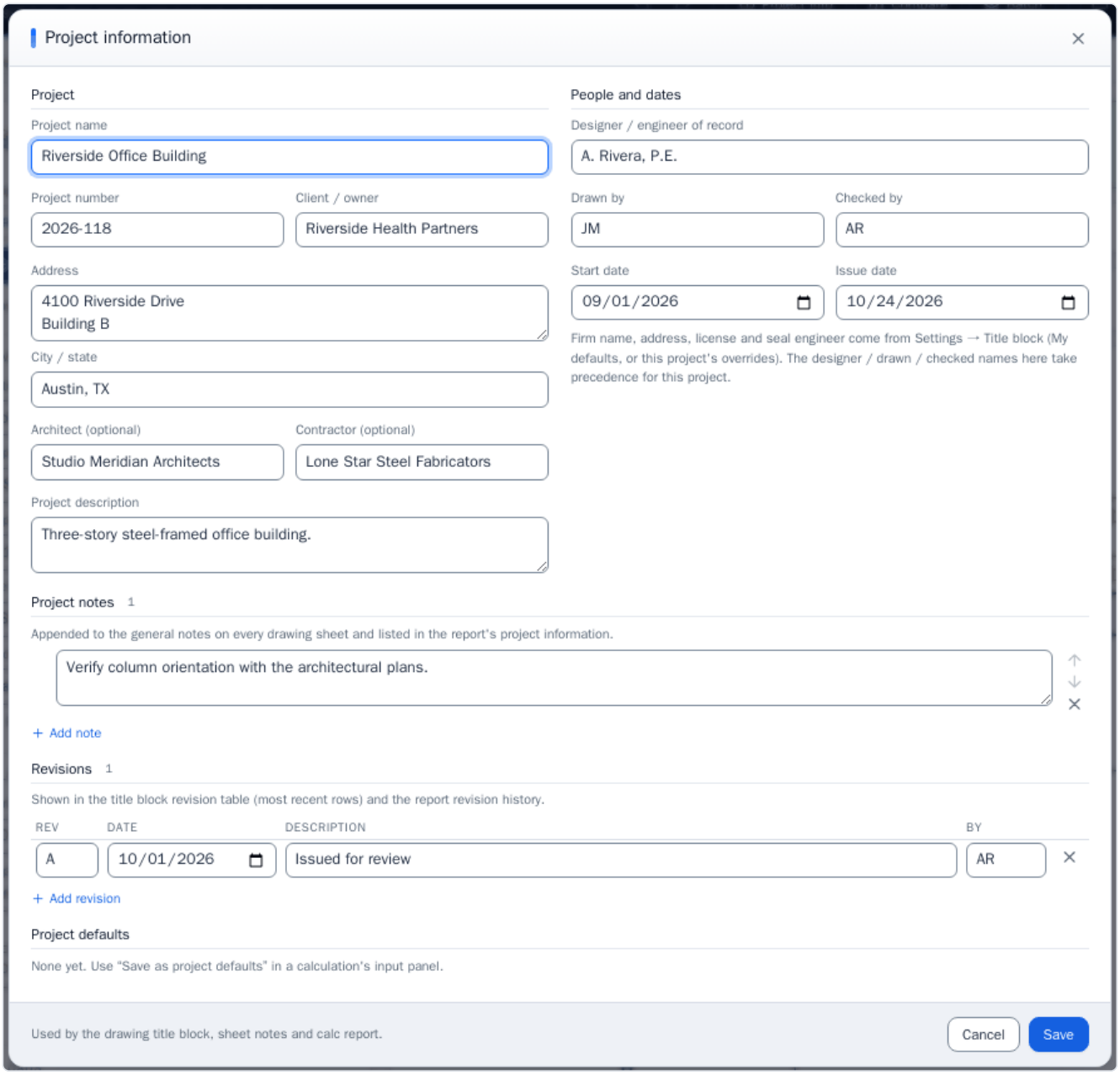
Projects and files

The project panel

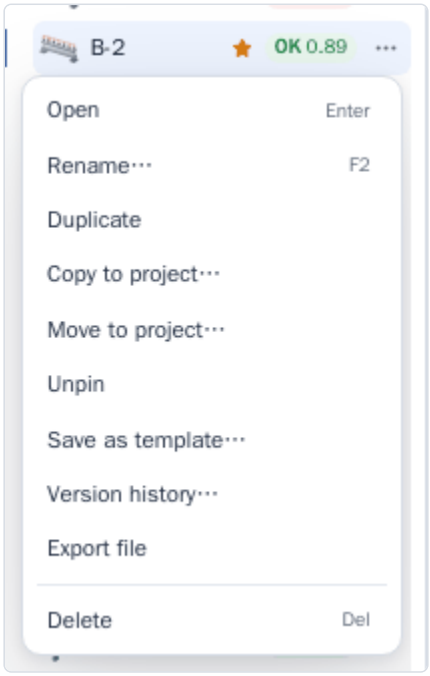
Projects hold calculations. The panel on the left lists your projects and their calculations with each one's status and governing ratio. A walk-through is in Training 2.



- **Search** finds projects and calculations by mark, project name or client.
- **Recently** and **By mark** change the order; **Issues** shows only the calculations that are not adequate, not feasible or have input errors.
- Right-click a calculation (or open its menu) to rename, duplicate, move, delete or pin it, save it as a template, or see its version history. Drag a calculation onto another project to move it (Ctrl+drop copies).
- Deleted projects and calculations go to the **Trash**, where you can restore them.
- **Export** saves a project as a file; **Import** reads it back.
- **Project info** holds what the title block and the report print: name, number, client, location, architect, contractor, engineer, dates, notes and revisions.



The menu of a calculation:



Batch

Batch designs a whole list of the same calculation type from one table: each row changes a few inputs (for example the shape and the length) of a base calculation. Paste rows from a spreadsheet, review the results, and save the rows you want as calculations in the project.

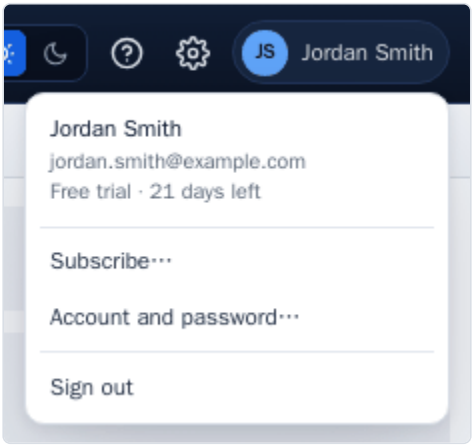
CHAPTER 4

Training 1: Getting started

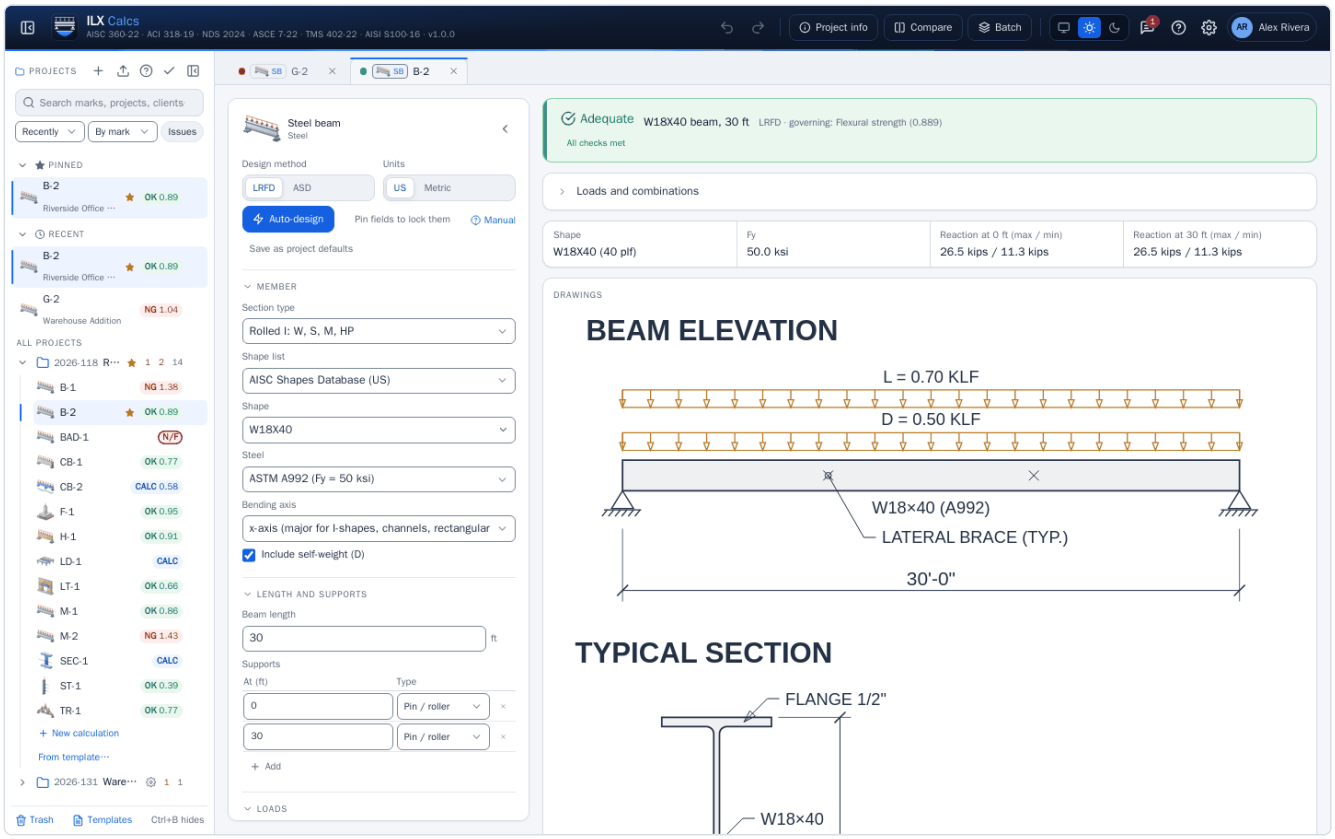
Fifteen minutes, one steel beam, start to report. Every number in the training chapters comes from running that exact calculation in the app. A calculation of yours with the same inputs gives the same results, unless a later version changes a default or a method; the chapters are tested against the app for that.

Sign in

Sign in with your ILX account. During the 30-day free trial the account menu shows the days left.



The screen



- **Left:** the project panel. Projects hold calculations, each with its status and governing ratio.
- **Middle:** the inputs of the open calculation. Results update as you type, and every change is saved.
- **Right:** the status, key results, drawings, the output buttons and the checks.
- **Header:** undo and redo, Project info, Compare, Batch, the theme, Support, this manual (**?** or F1), Settings and your account.

Your first calculation

1. Under a project, select **New calculation**. The **Calculation type** step shows a card with a picture for every type. Select **Steel beam**, then **Next** (or press Enter).
2. The **Configure** step opens. Keep the cards **W shape**, **Simple span** and **Continuous**. On the right, the preview already designs the beam.
3. Set the mark to `B-1` and select **Create**.

Make sure these inputs are set. They are today's defaults, stated here so this walk-through stays true if a default changes:

Input	Value
Section type	Rolled I: W, S, M, HP
Shape, Steel	W21X50, ASTM A992 (Fy = 50 ksi)
Beam length	35 ft, pin / roller at 0 and 35 ft

Input	Value
Line loads	D = 0.45 kip/ft, L = 0.75 kip/ft, whole length
Self-weight	Included
Compression flange braced	Continuously
Deflection limits	Live load span / 360, total load span / 240
Design method	LRFD

What you get

The status bar reads **Adequate**, *W21X50 beam, 35 ft*, governing **Total load deflection** at a ratio of **0.845**.

Check	Ratio
Flexural strength (AISC 360-22 F2)	0.668
Total load deflection	0.845

The reaction at each end is 31.5 kips at most. Scroll down the results for the shear, moment and deflection diagrams.

Change the live load to 1.0 kip/ft, and the calculation and every drawing follow. Select undo (Ctrl+Z) to go back.

Print it

Select **Calc report (PDF)** for the report and **Drawing sheet** for the sheet. The next chapter sets what they print.

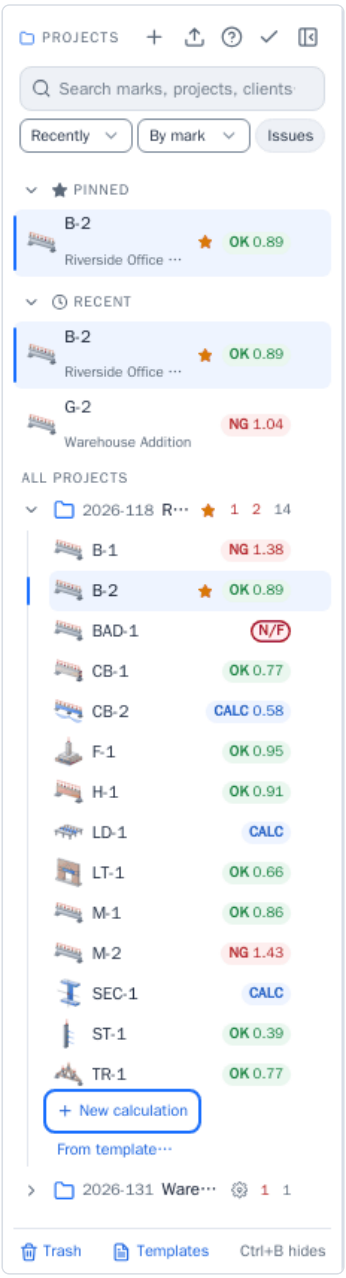
Next: Projects, settings, title block and output.

CHAPTER 5

Training 2: Projects, settings, title block and output

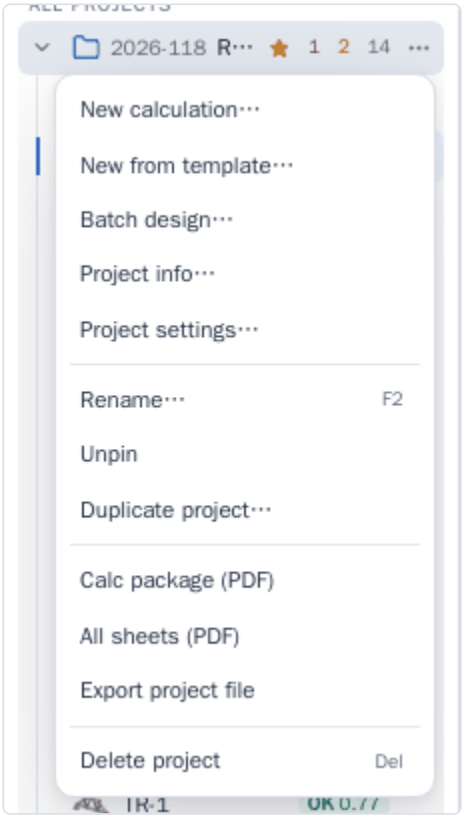
Set this up once and your reports and sheets carry your firm. The rule: **your defaults apply to all your projects, and a project can override any field.** Nothing is global to the server, so a colleague's settings never change yours.

Projects



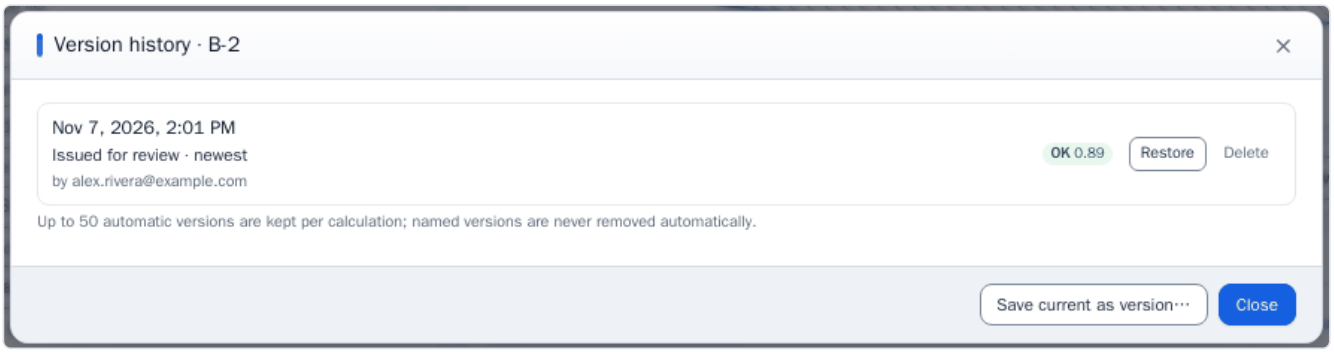
- **+** creates a project. **Project info** in the header (or the project's menu) holds the name, number, client, location, architect, contractor, engineer, dates, notes and revisions. These go on the title block and the report.
- **Recently**, **By mark** and **Issues** in the panel order and filter the list. **Issues** shows only what is not adequate or not feasible.
- The status in the list is the calculation's: `OK 0.89`, `NG 1.38`, `N/F` for not feasible, `CALC` for tools with no pass or fail.

Right-click a project for its menu:



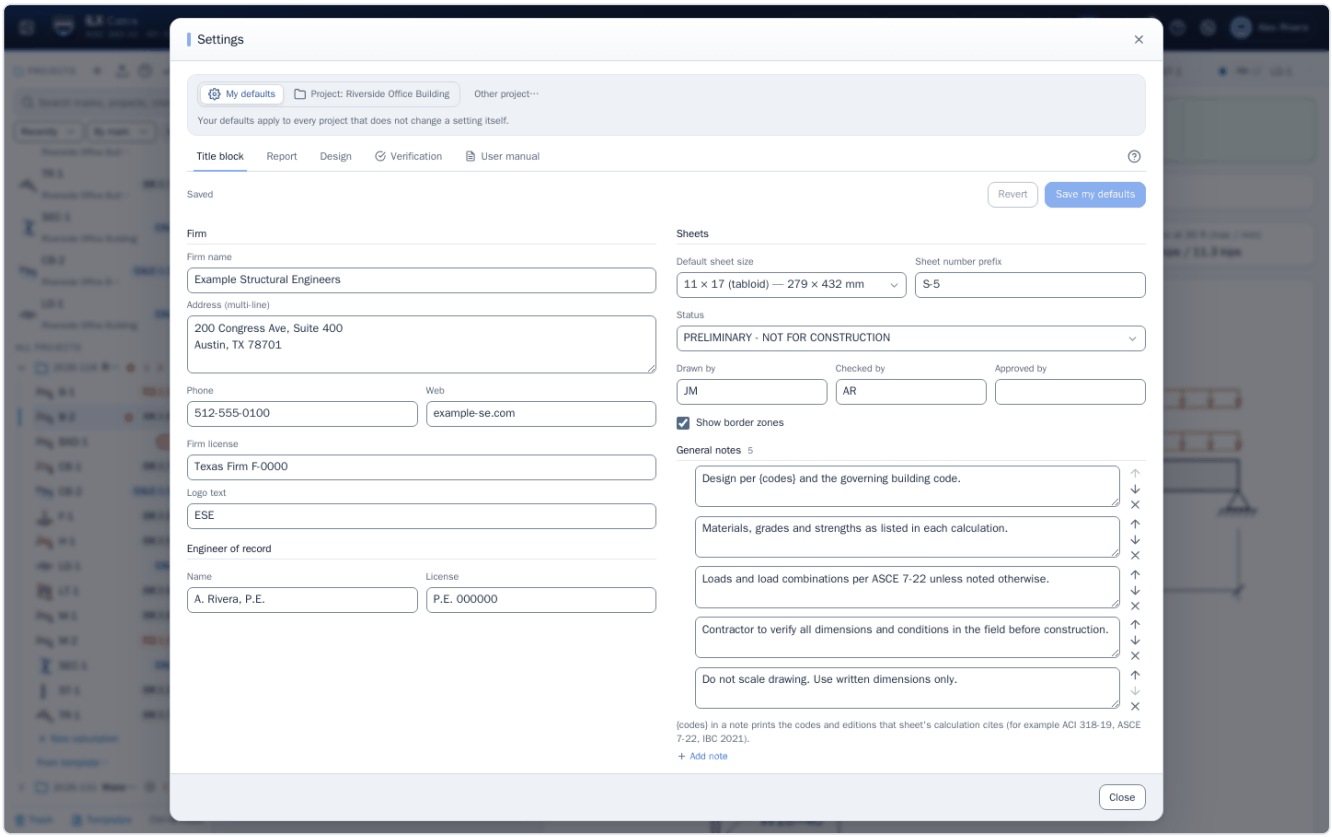
Calc package (PDF) puts the project's calculations in one report; **All sheets (PDF)** does the same for the drawing sheets. **Export project file** saves the project as a file, and the import button at the top of the panel reads it back. Deleted items go to **Trash** at the bottom of the panel.

Right-click a calculation for its menu. **Version history** saves named versions and restores them; **Save as template** starts new calculations from this one.



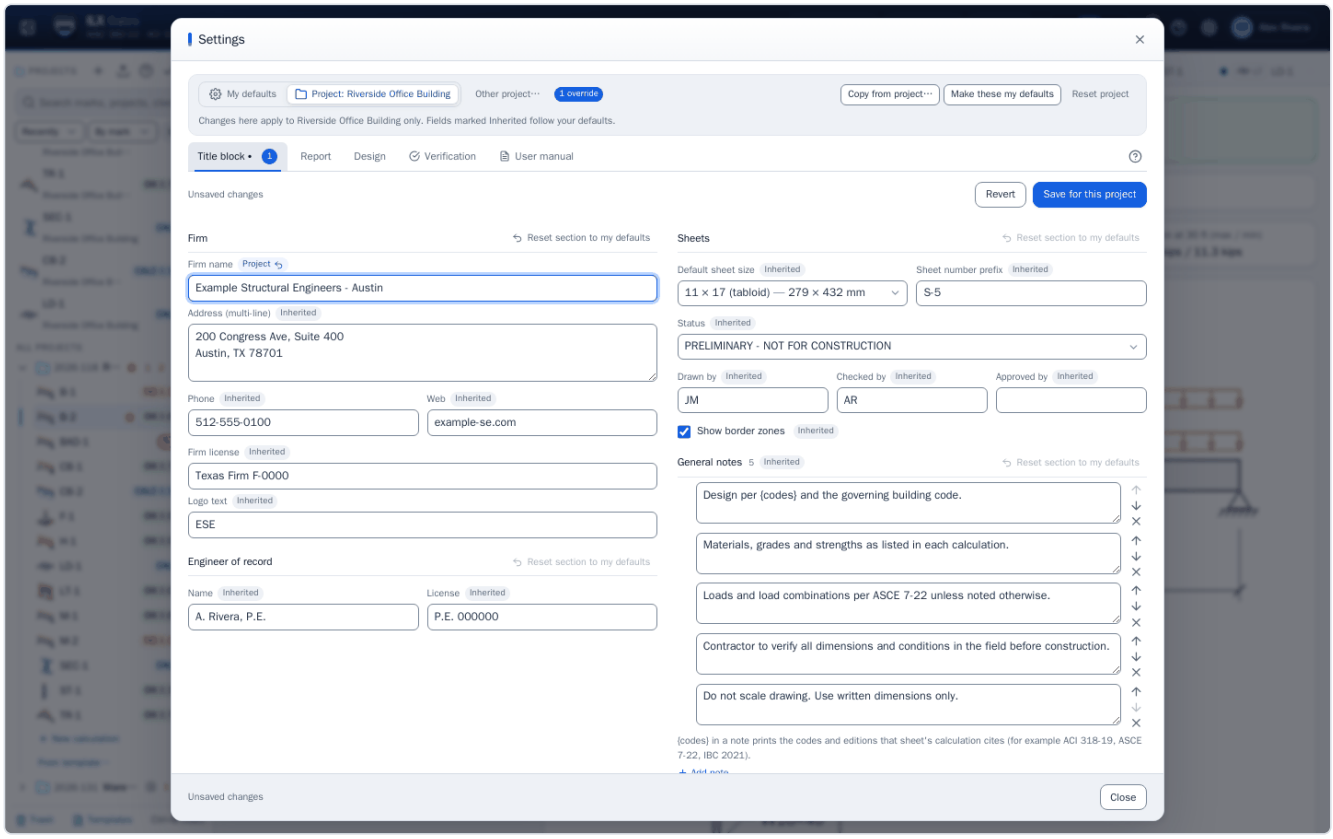
Title block: my defaults, then the project

Open **Settings** (the gear) with **My defaults** selected:



Enter the firm name, address, phone, web, license, logo text, engineer of record, drawn and checked by, the default sheet size, the sheet number prefix, the status line and the general notes. `{codes}` in a note prints the codes and editions that the sheet's calculation cites. Select **Save my defaults**.

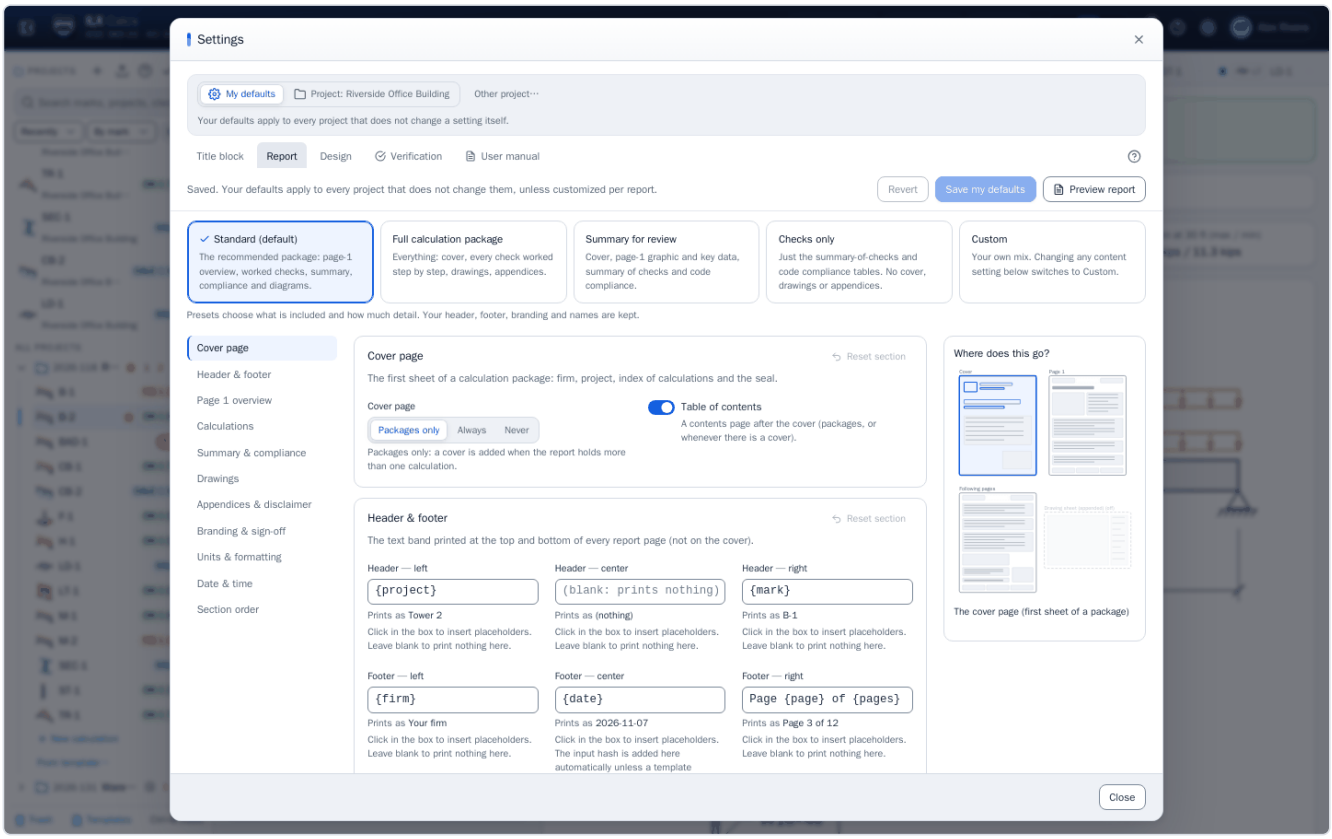
To change something for one job, select the project in the bar at the top. Each field shows **Inherited** (it follows your defaults) or **Project** (it is overridden, with a button to take it back):



Copy from project takes another project's overrides. **Make these my defaults** promotes them. **Reset project** removes them all.

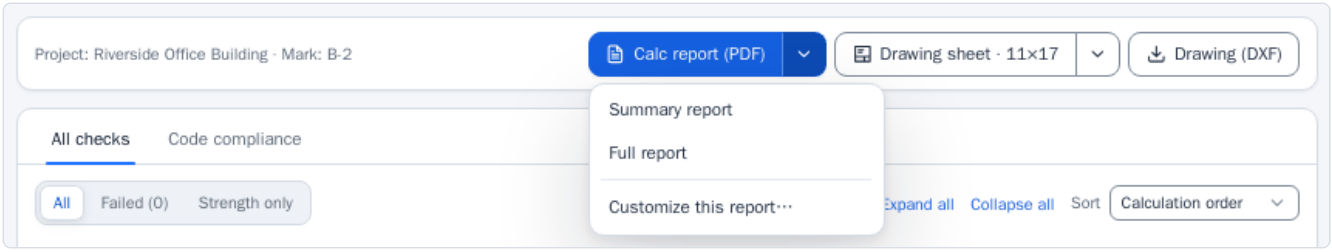
Save as project defaults in the editor is a different thing: it keeps the materials, methods and limits of this calculation as the starting values for new calculations of the same type in this project.

What the report prints



The **Report** tab sets the sections and their order, the level of detail, the units, the headers and footers, the signature and the disclaimer. The diagram at the right shows what prints where. **Preview report** shows the result before you save.

Output of one calculation



- **Calc report (PDF)** prints with the saved settings. The arrow beside it offers **Summary report**, **Full report** and **Customize this report**, which changes this one report only.
- **Drawing sheet** makes the sheet PDF, with the sizes in its arrow. **Drawing (DXF)** gives the same sheet for your CAD.

A report starts with the status, the inputs and the required strength, then every check with its equations:

Steel Beam Design

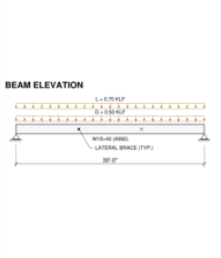
W18X40, 30 ft, AISC 360-22

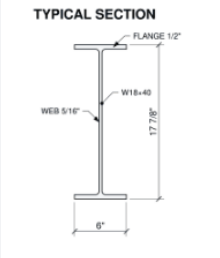
Mark	**B-2**
Date	2026-11-07
Method	LRFD
Units	US customary
Input	#ba39665a0a

ADEQUATE — governing: Flexural strength (ratio 0.889)

Project Information

Project	—
Issue date	2026-11-07





Shape	W18X40
Steel	ASTM A992 (Fy = 50 ksi)
Include self-weight (D)	Yes
Beam length	30.00 ft
Required	Mu, Vu
Governing	Flexural strength (0.889)

Input Summary

Shape	W18X40
Steel	ASTM A992 (Fy = 50 ksi)
Include self-weight (D)	Yes
Beam length	30.00 ft
Compression flange braced	At a spacing
Brace spacing	10.00 ft
Cb	Computed (Eq. F1-1)
Live load: span /	360
Total load: span /	240
Supports	Pin / roller at 0.00 ft, Pin / roller at 30.00 ft
Line load D	0.500 kip/ft, 0.00 ft to 30.00 ft
Line load L	0.700 kip/ft, 0.00 ft to 30.00 ft

Required Strength

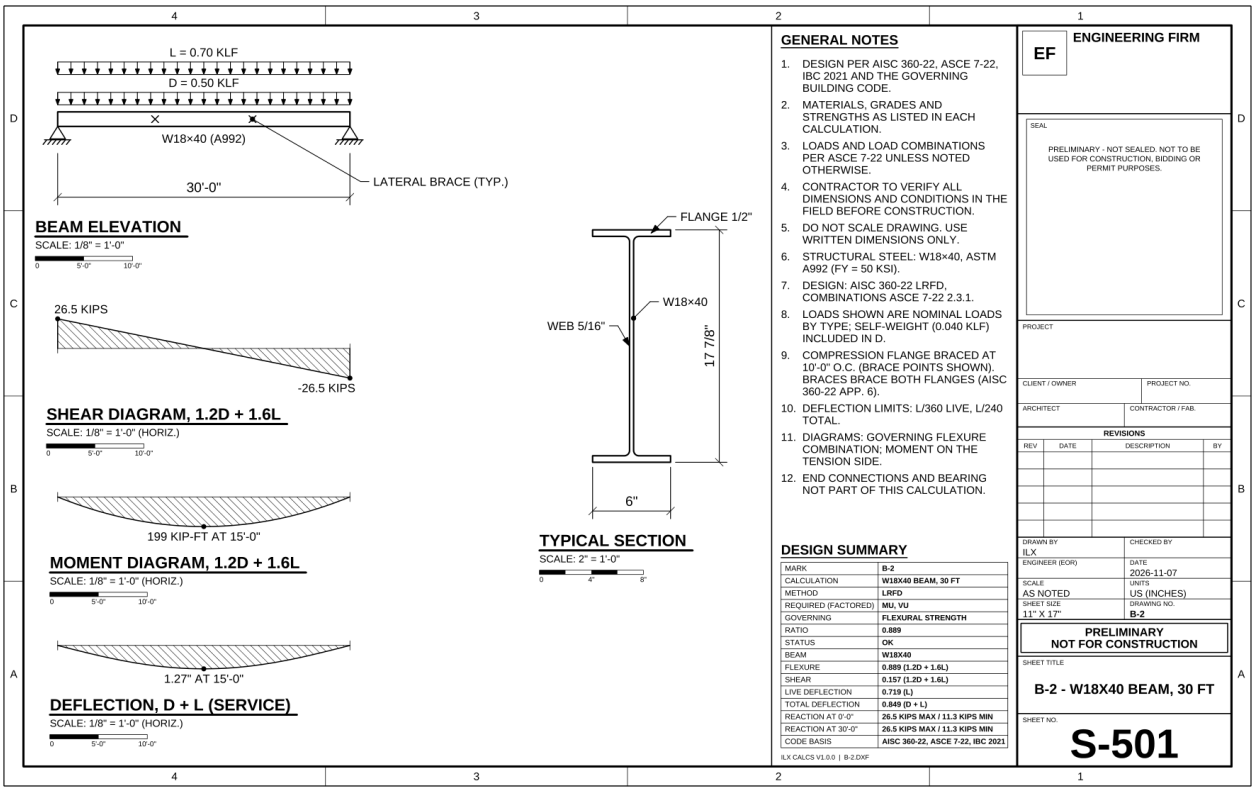
Load combinations: ASCE 7-22 2.3.1, 2 evaluated

```
Load combinations: ASCE 7-22 2.3.1, 2 evaluated
```

W and E act in both directions; Ev, Ω0 and the 0.5L option are not modeled.

Self-weight (D) = `0.040 kip/ft`

The sheet carries the elevation, section, diagrams, general notes, design summary and your title block:

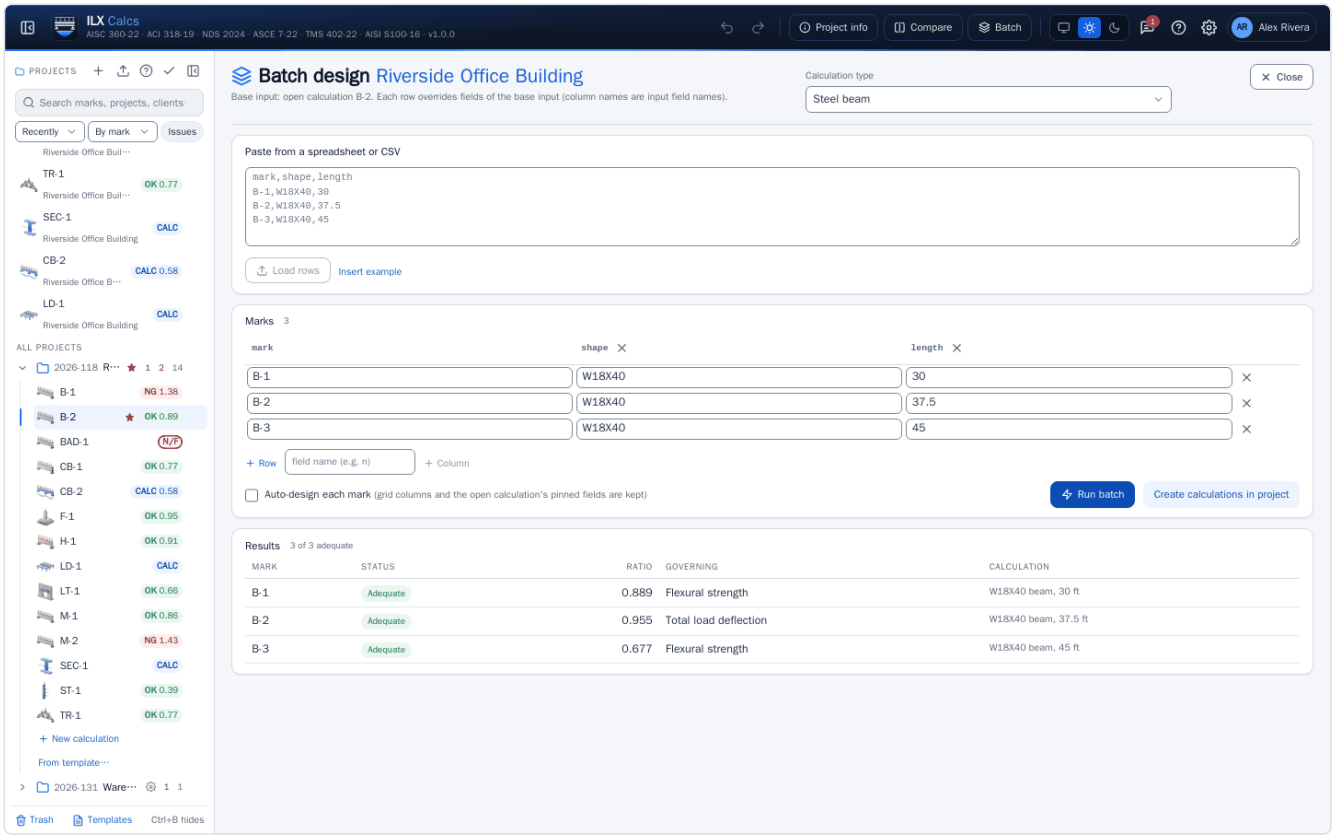


The sheet's status line comes from your title block, *Preliminary, not for construction* by default. Sealing and issuing are yours.

Four types make no sheet, because they have no geometry to draw: load chasedown, load combinations, dead load build-up and the unit converter.

Many marks at once

Batch in the header designs many marks of one type from a table: paste rows from a spreadsheet, optionally auto-design each mark, review, and create the calculations in the project.



Next: Reading checks, steps and clauses.

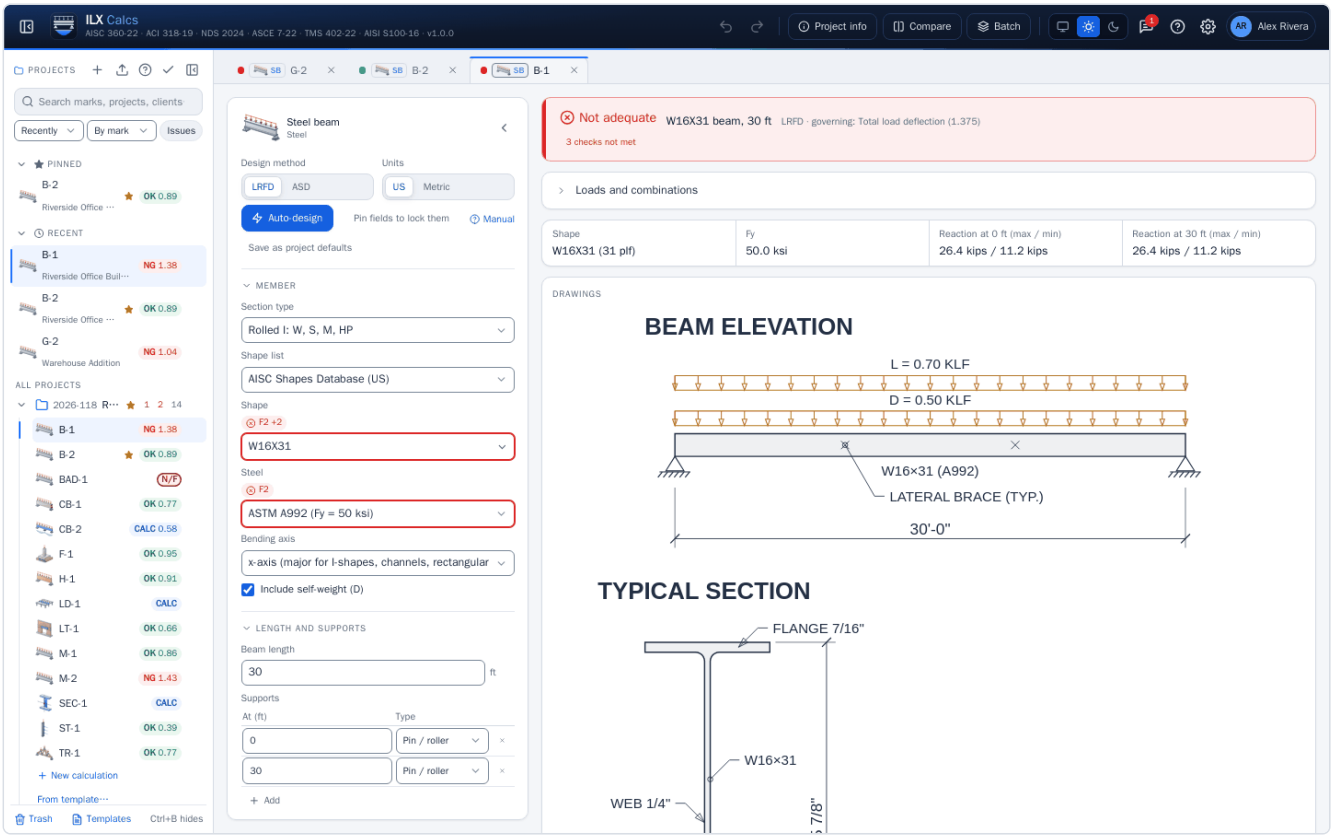
CHAPTER 6

Training 3: Reading checks, steps and clauses

A calculation is only as useful as your check of it. This chapter reads one that fails.

Set it up. New steel beam, mark `B-1`: W16X31, A992, 30 ft simple span, D = 0.5 kip/ft, L = 0.7 kip/ft, self-weight on, compression flange braced at a spacing of 10 ft, LRFD, deflection limits span / 360 and span / 240.

The status bar



The bar reads **Not adequate**, *W16X31 beam, 30 ft*, governing **Total load deflection (1.375)**, *3 checks not met*. The inputs that matter to a failing check carry a red mark with the clause, here **F2** on the shape and the steel.

There are three states, and tools have a fourth:

- **Adequate:** every check is met.
- **Not adequate:** at least one is not. The results are still calculated, so you can see by how much.
- **Not feasible:** the inputs describe something that cannot be calculated, and there are no results to trust. See below.

- **Calculated:** a tool with no pass or fail, such as the load takeoff or the section builder.

The checks table

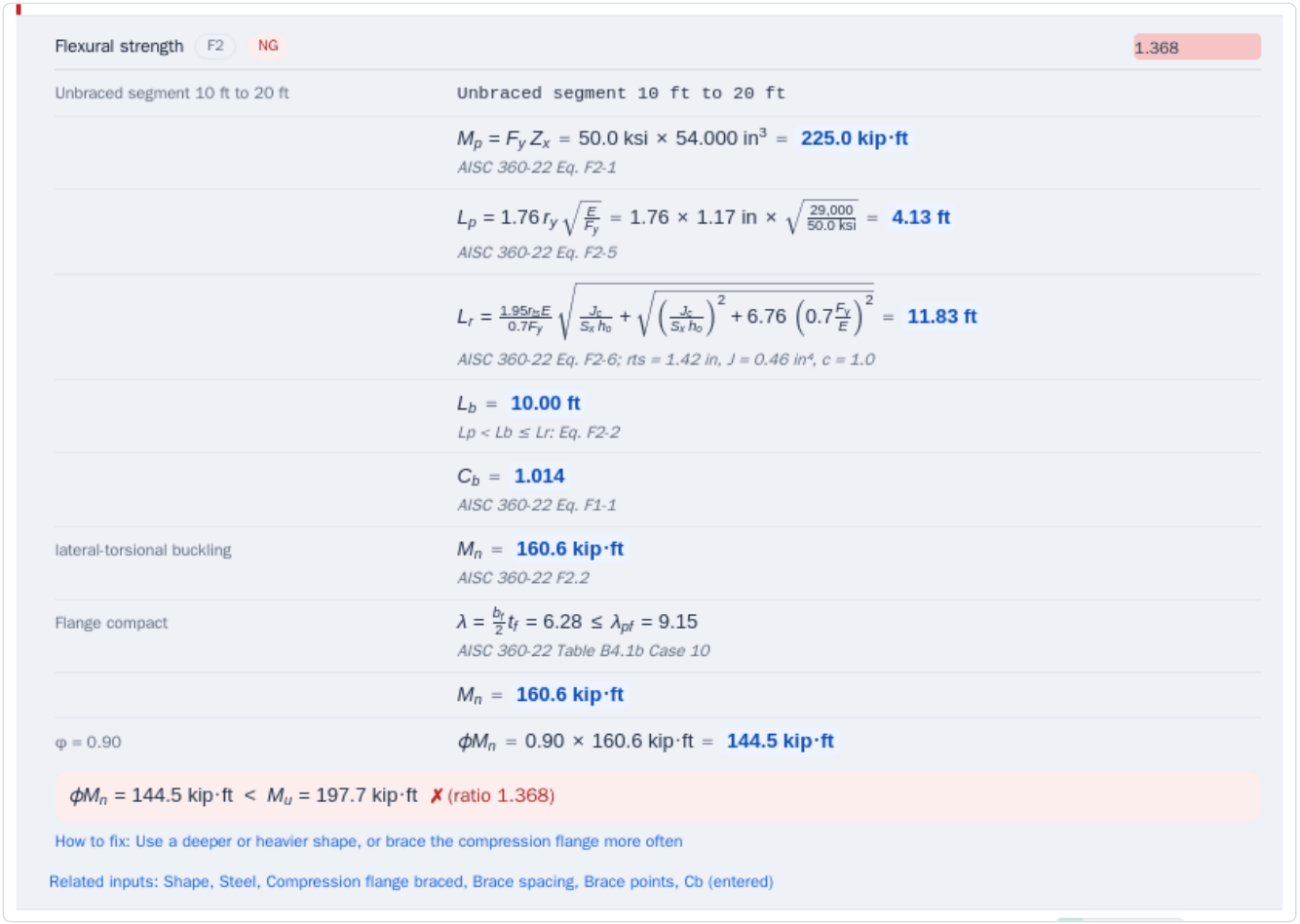
Below the drawings, **All checks** lists every check. Each row gives the required value, the available value, the ratio and the load combination that governs it.

Check	Clause	Required	Available	Ratio	Governs
Flexural strength	AISC 360-22 F2	197.7 kip-ft	144.5 kip-ft	1.368	1.2D + 1.6L
Live load deflection	IBC 2021 Table 1604.3 (limit entered)	1.173 in	1 in	1.173	L
Total load deflection	IBC 2021 Table 1604.3 (limit entered)	2.063 in	1.5 in	1.375	D + L

The buttons above the table show **All**, **Failed** or **Strength only**. **Code compliance**, the next tab, lists the limits and recommendations that are not strength checks. A clause marked `(to confirm against the code text)` is one this version has not yet confirmed against the code text; read that clause yourself.

Open a check: the steps

Select a row. Every equation appears with its values, its result and its clause:



Read them as you would a hand calculation. Here, in the governing segment *Unbraced segment 10 ft to 20 ft*:

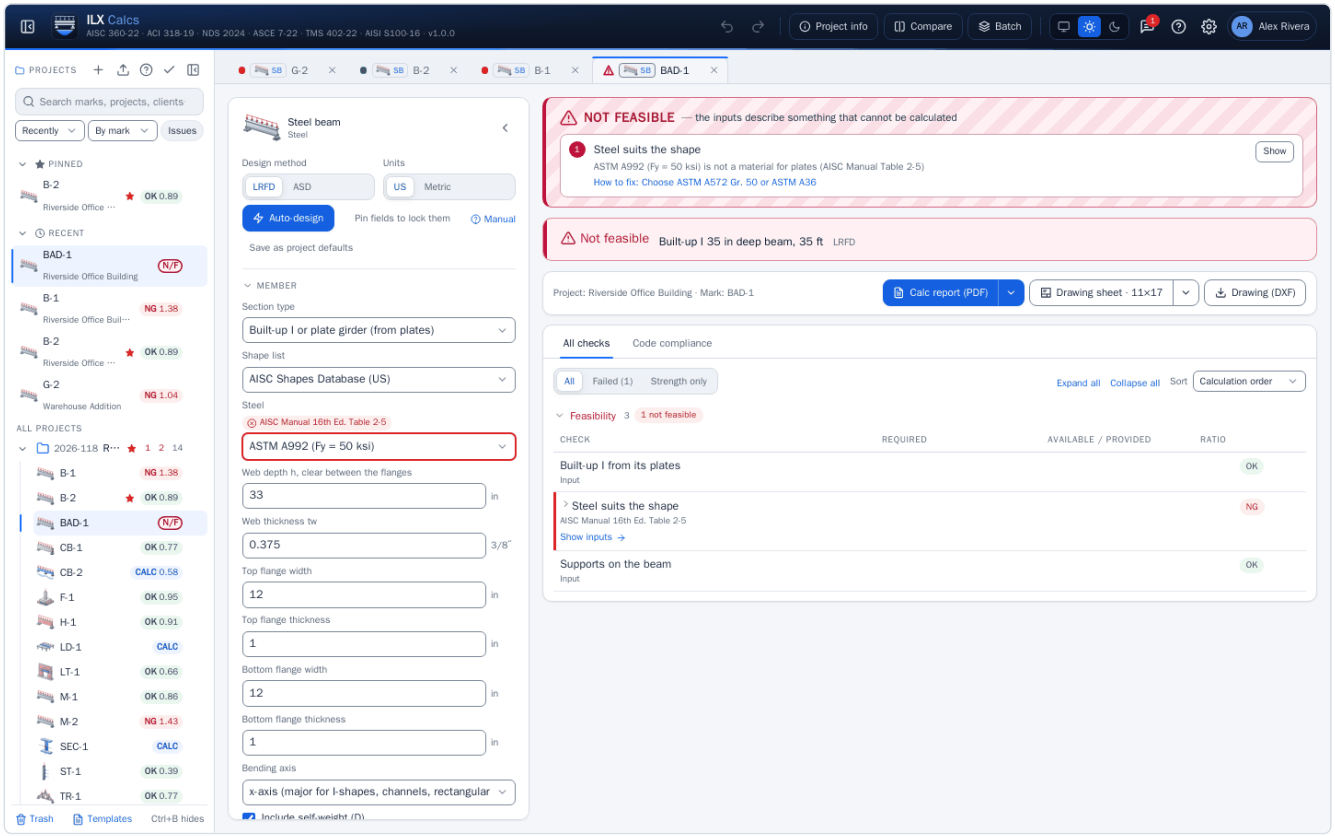
Step	Result	Clause
Mp = Fy Zx	225.0 kip-ft	AISC 360-22 Eq. F2-1
Lp = 1.76 ry √(E/Fy)	4.13 ft	Eq. F2-5
Lr	11.83 ft	Eq. F2-6
Lb, between Lp and Lr	10.00 ft	Eq. F2-2 applies
Cb	1.014	Eq. F1-1
Mn	160.6 kip-ft	F2.2
φMn = 0.90 × 160.6	144.5 kip-ft	

The end of the steps states the comparison, gives **How to fix** (here: *Use a deeper or heavier shape, or brace the compression flange more often*) and links the **Related inputs**. **Show inputs** takes you to them.

Loads and combinations

Open **Loads and combinations** above the results. It lists what was evaluated. Enter service loads, each with its type (D, L, Lr, S, R, W, E): the program applies the ASCE 7-22 combinations of those types and reports the governing one per check. Self-weight is added to D when selected, which is why the moment above is 197.7 kip-ft and not the 193.5 kip-ft of the entered loads alone (1.72 kip/ft on 30 ft). See Loads and combinations for what is and is not modeled.

Not feasible



Here a plate girder was left on A992, which AISC Manual Table 2-5 does not list for plates. The banner names the problem, its reference and the fix; **Show** takes you to the input. Choose **ASTM A572 Gr. 50 or ASTM A36** and the calculation runs.

What the ratio does not tell you

A ratio under 1.00 means the checks this type performs are met for the inputs you gave. It says nothing about checks the type does not perform. Read each chapter's **Limitations**, and see Limits and your responsibility.

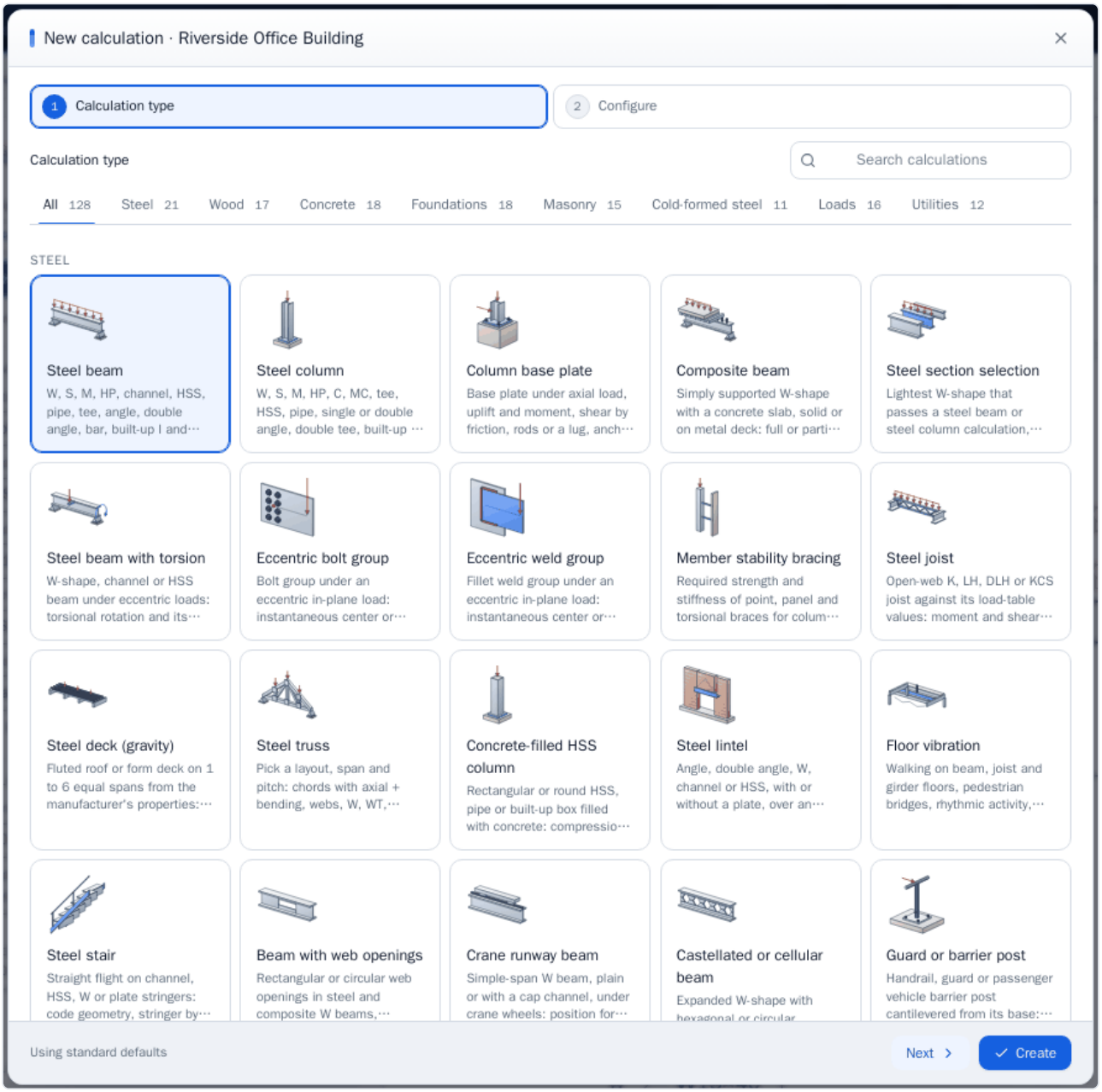
Next: The Wizard and auto-design.

CHAPTER 7

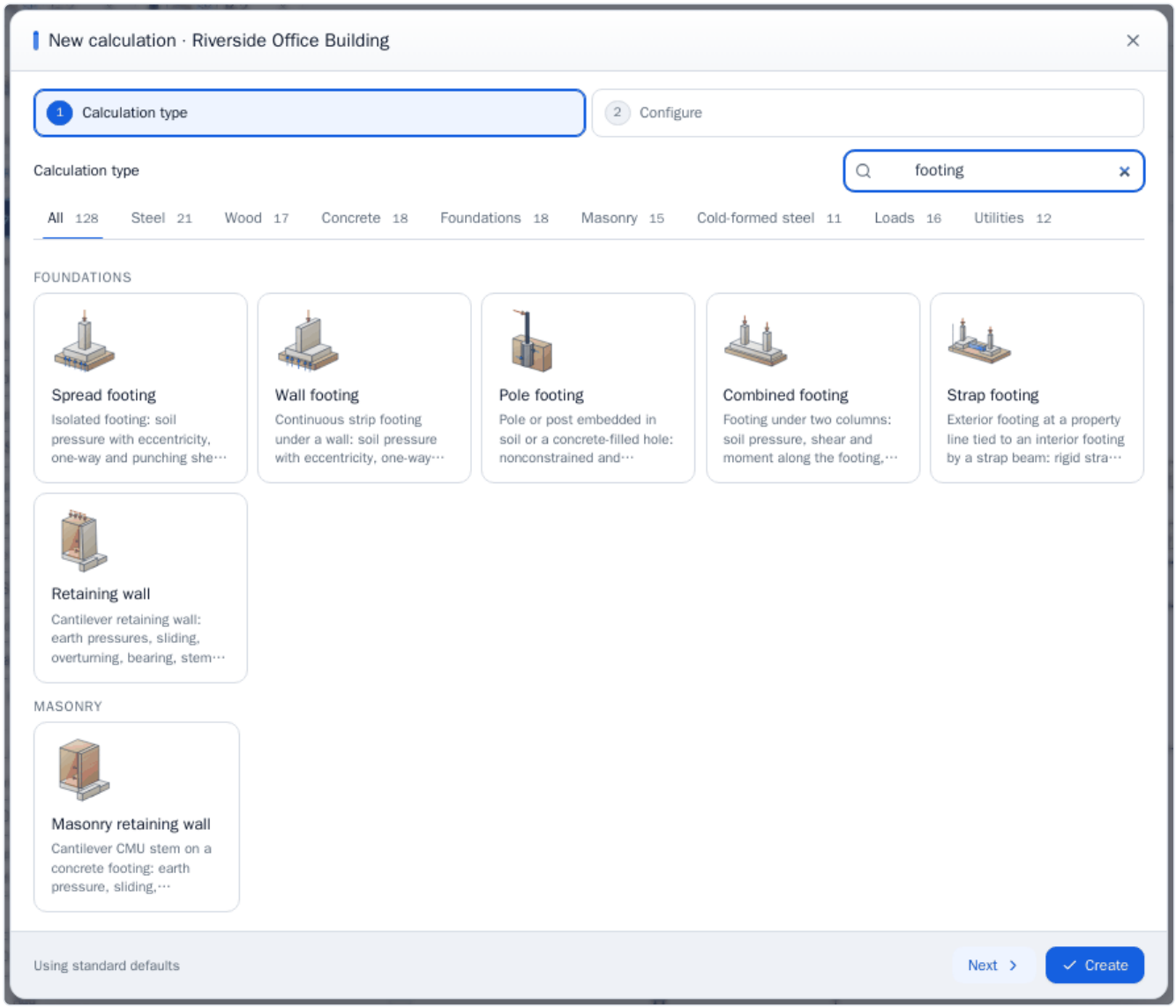
Training 4: The Wizard and auto-design

The Wizard

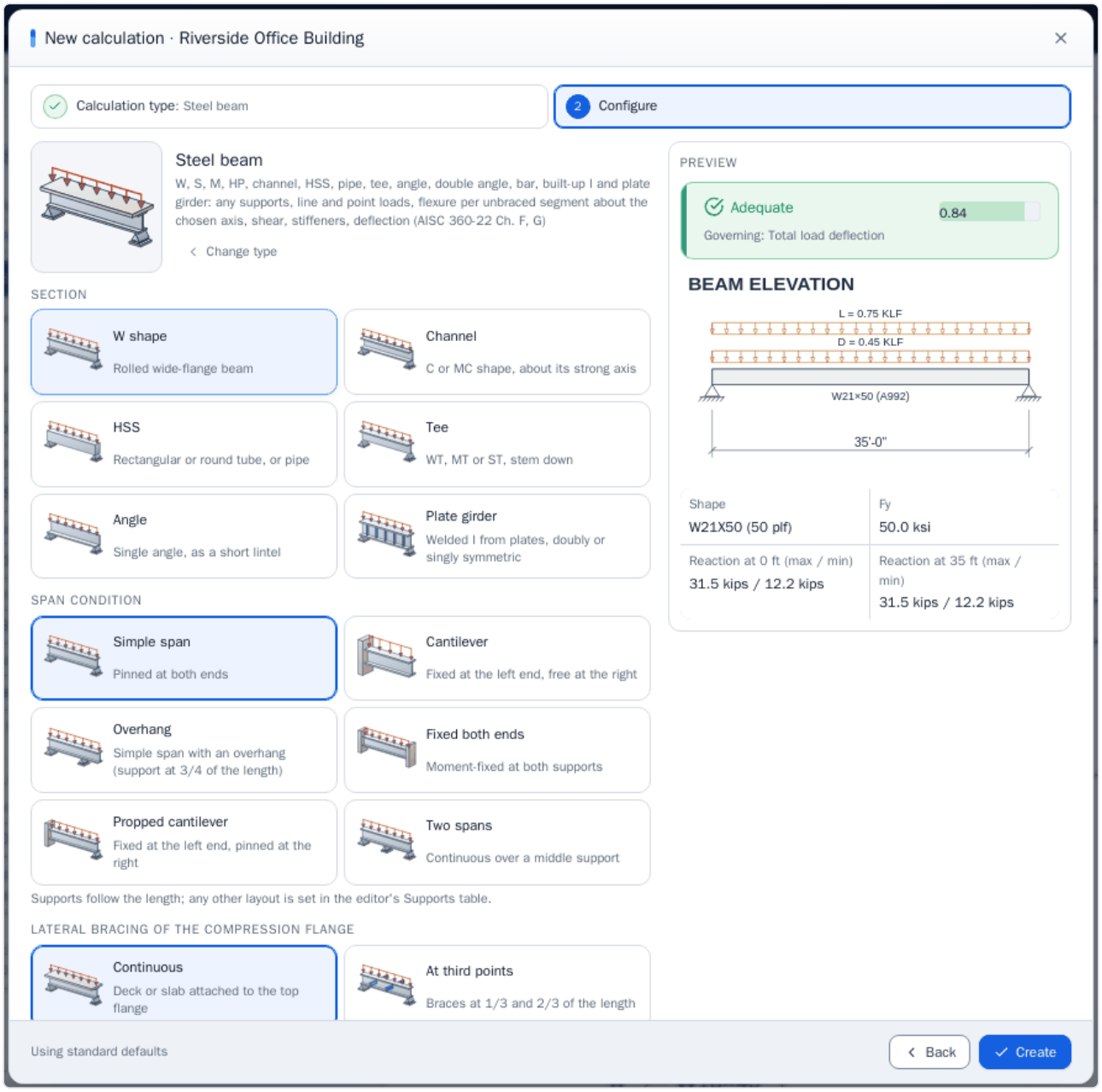
New calculation opens the Wizard. The first step is every type as a picture card, with tabs for the material groups and a search box:



Type a word, such as `footing`, and the cards narrow:



Configure shows the type's main choices as cards, with the design beside them, updating as you choose:



For a steel beam: **Section** (W shape, Channel, HSS, Tee, Angle, Plate girder), **Span condition** (Simple span, Cantilever, Overhang, Fixed both ends, Propped cantilever, Two spans) and **Lateral bracing**. Each card sets several inputs. An overhang card, for example, puts the support at 3/4 of the length, and the Supports table in the editor takes any other layout. Below the cards are the key inputs, the mark, the units and the method.

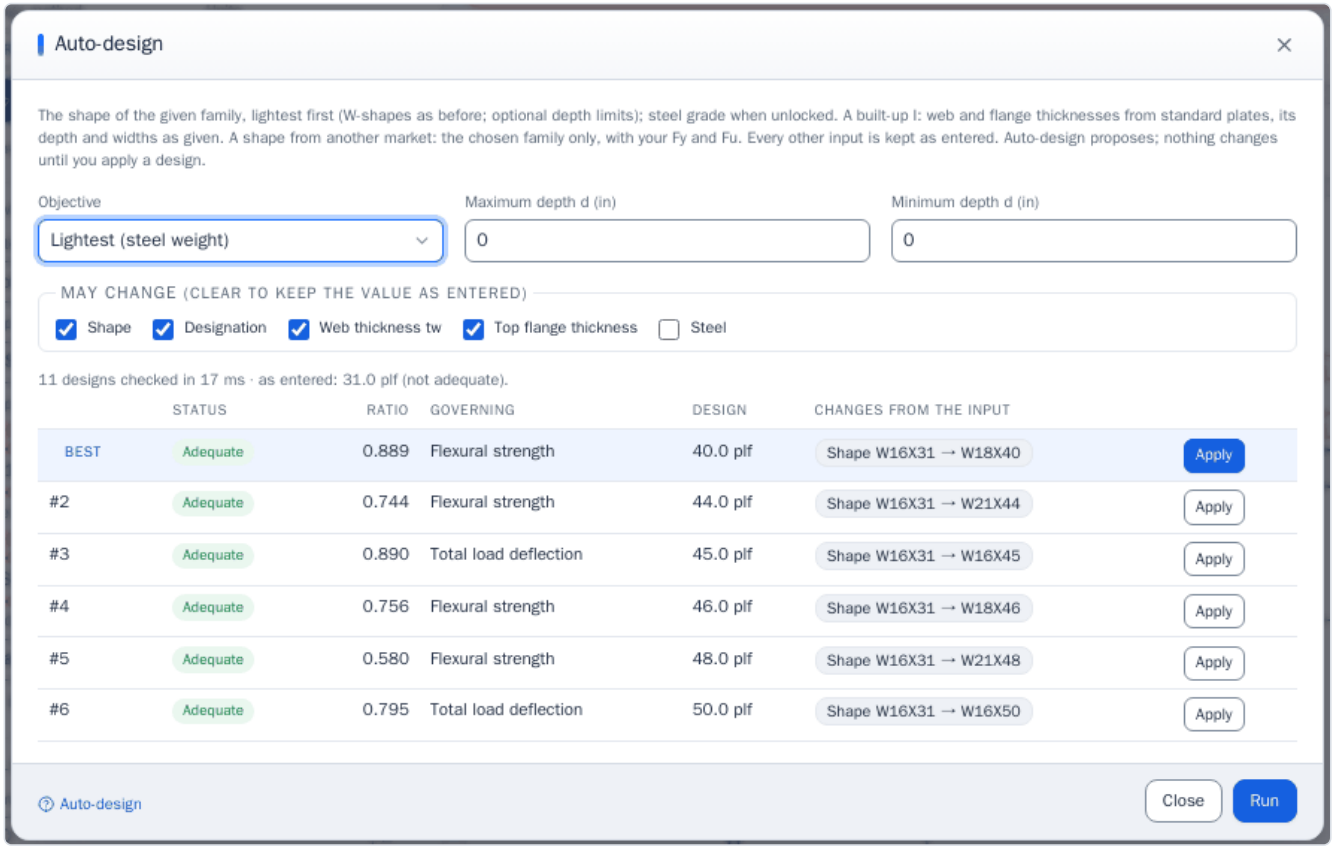
Every other input starts from its default, or your project's defaults, and is set in the editor. A card sets a starting point, not a design: check it.

Enter and the arrow keys work on the cards. **Escape** closes the Wizard and creates nothing.

Letting auto-design search

Auto-design in the editor searches for the lightest or cheapest input that passes every one of the calculation's own checks. It proposes; nothing changes until you apply a design.

Take the failing beam of the last chapter: W16X31, 30 ft, D = 0.5 and L = 0.7 kip/ft, braced at 10 ft. Select **Auto-design**:



	Status	Ratio	Governing	Change
Best	Adequate	0.889	Flexural strength	W16X31 to W18X40, 40 plf
#2	Adequate	0.744	Flexural strength	to W21X44, 44 plf
#3	Adequate	0.890	Total load deflection	to W16X45, 45 plf
#4	Adequate	0.756	Flexural strength	to W18X46, 46 plf

Things to do with this table:

- **Objective.** Here *Lightest (steel weight)*; steel beams also offer the shallowest. Other types offer the least concrete or a cost index, which ranks the alternatives and does not estimate a job.
- **Limits. Maximum depth** and **Minimum depth** narrow the search: 0 means no limit. A ceiling that allows 18 in. removes W21X44 from the list.
- **What may change.** Clear a box to keep that input as entered. Steel starts cleared: select it to let the search choose the grade.
- **Pins.** The pin next to an input's label locks it, so a search you run later keeps it.
- **Apply** puts a row's values into the form. Ctrl+Z takes it back.

Every other input (loads, supports, bracing, code options) stays as entered. Auto-design stops after the best design and five alternatives or at its time limit, a few seconds. When nothing passes, it shows the closest one and what it does not meet.

After **Apply**, read the checks as for any calculation: the steel chapter does that with W18X40. Auto-design does not know about the framing, the fabricator or the depth of the beam next door.

Next: Units.

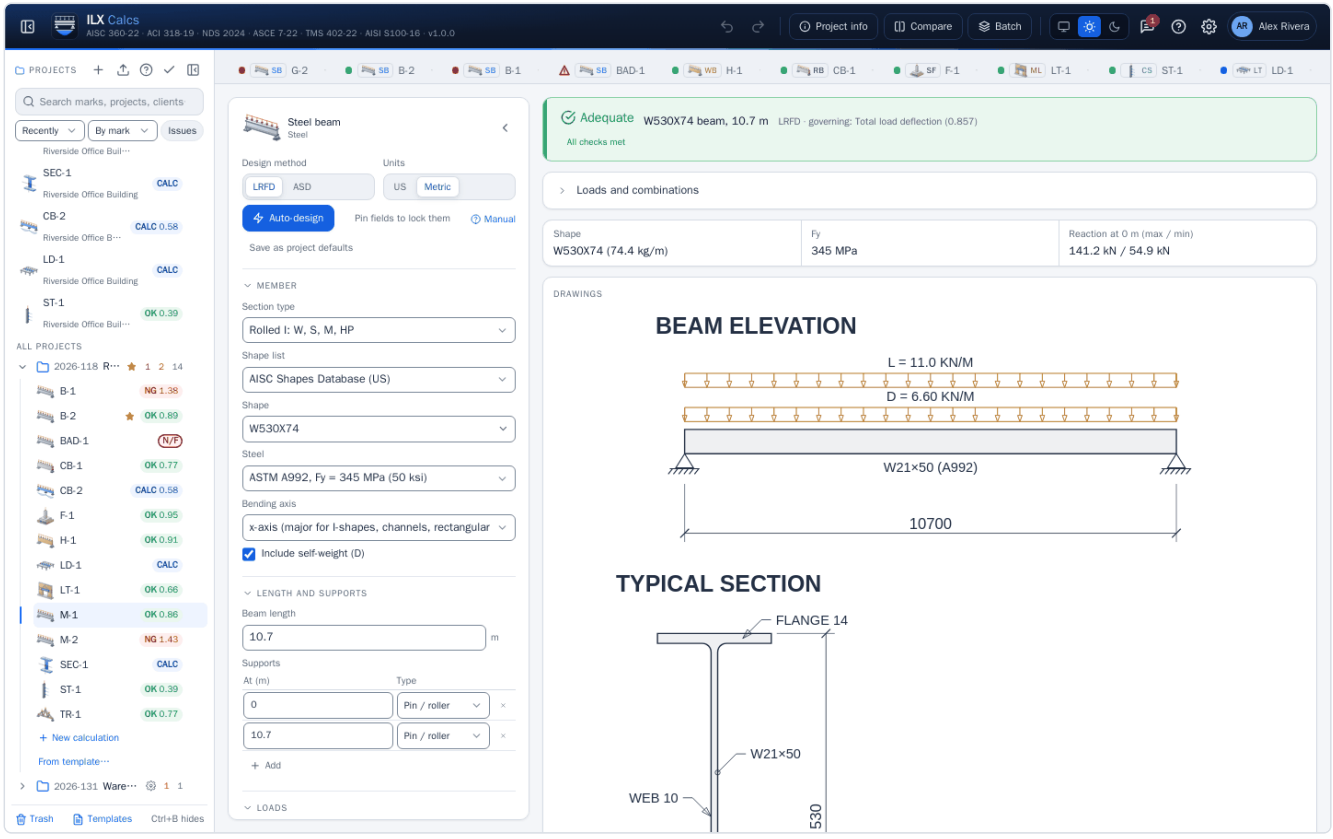
CHAPTER 8

Training 5: Units, and shapes from other markets

Each calculation has its own **Units** switch: **US** or **Metric**. The report and the sheet follow it. Switching converts every value, and nothing is lost going back and forth.

A new calculation in Metric

Choose **Metric** in the Wizard. A new calculation starts from round metric values, not from converted ones: 345 MPa, 10.7 m. The steel beam's default in Metric:



The result is *W530X74 beam, 10.7 m*, on A992 at 345 MPa, with D = 6.6 and L = 11 kN/m.

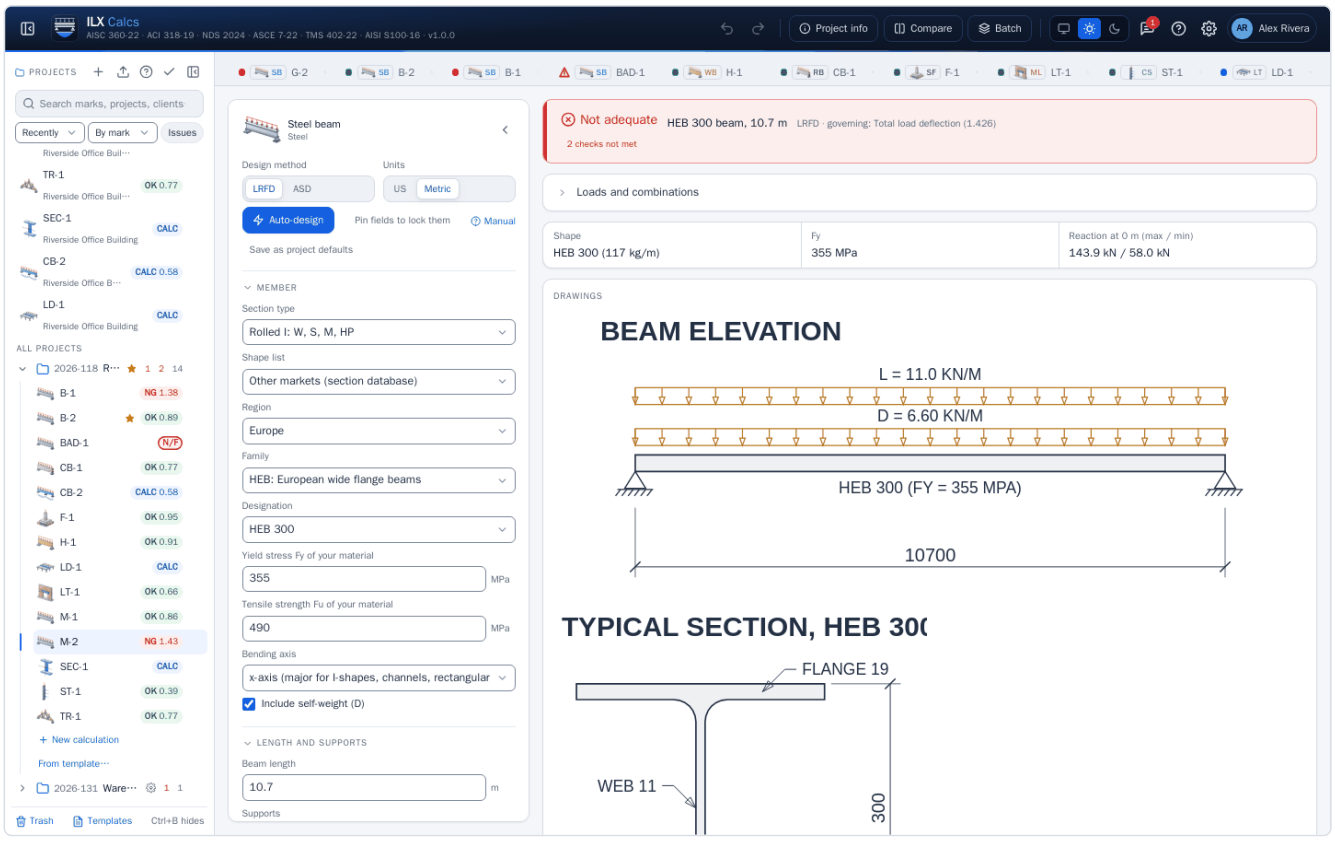
Check	Required	Available	Ratio
Flexural strength	377.8 kN-m	559.3 kN-m	0.675
Live load deflection	22.9 mm	29.7 mm	0.771
Total load deflection	38.2 mm	44.6 mm	0.857

The shape list shows the AISC metric designations. A W530X74 is the W21X50 of Training 1. The design code is still AISC 360 and ASCE 7, in metric units.

Shapes from other markets

On the steel beam and the steel column, the **Shape list** can change from *AISC Shapes Database (US)* to *Other markets (section database)*. Then choose **Region**, **Family** and **Designation**.

For this example: Region *Europe*, Family *HEB*, Designation *HEB 300*, **Fy = 355 MPa** and **Fu = 490 MPa**, with the loads and span above.



HEB 300 beam, 10.7 m at 355 MPa is not adequate. Flexure is fine, 597.0 kN-m and a ratio of 0.645, but the total load deflection is 63.6 mm against 44.6 mm, a ratio of 1.426. **Auto-design** stays in the family you chose: it proposes **HEB 340**, ratio 0.988.

Three things to keep in mind:

- **The code stays AISC 360.** A shape from another market is checked to AISC 360 with your material, not to its own national code, and the sheet says so.
- **Fy and Fu are yours.** There is no default and no assumed grade. Whether that material suits AISC 360 (Section A3.1) is your judgment.
- **No published example applies AISC 360 to a non-US shape,** so this path is checked by hand calculations and by running US shapes through the same database. See Steel beam, verification.

Next: the worked calculations, starting with Steel.

CHAPTER 9

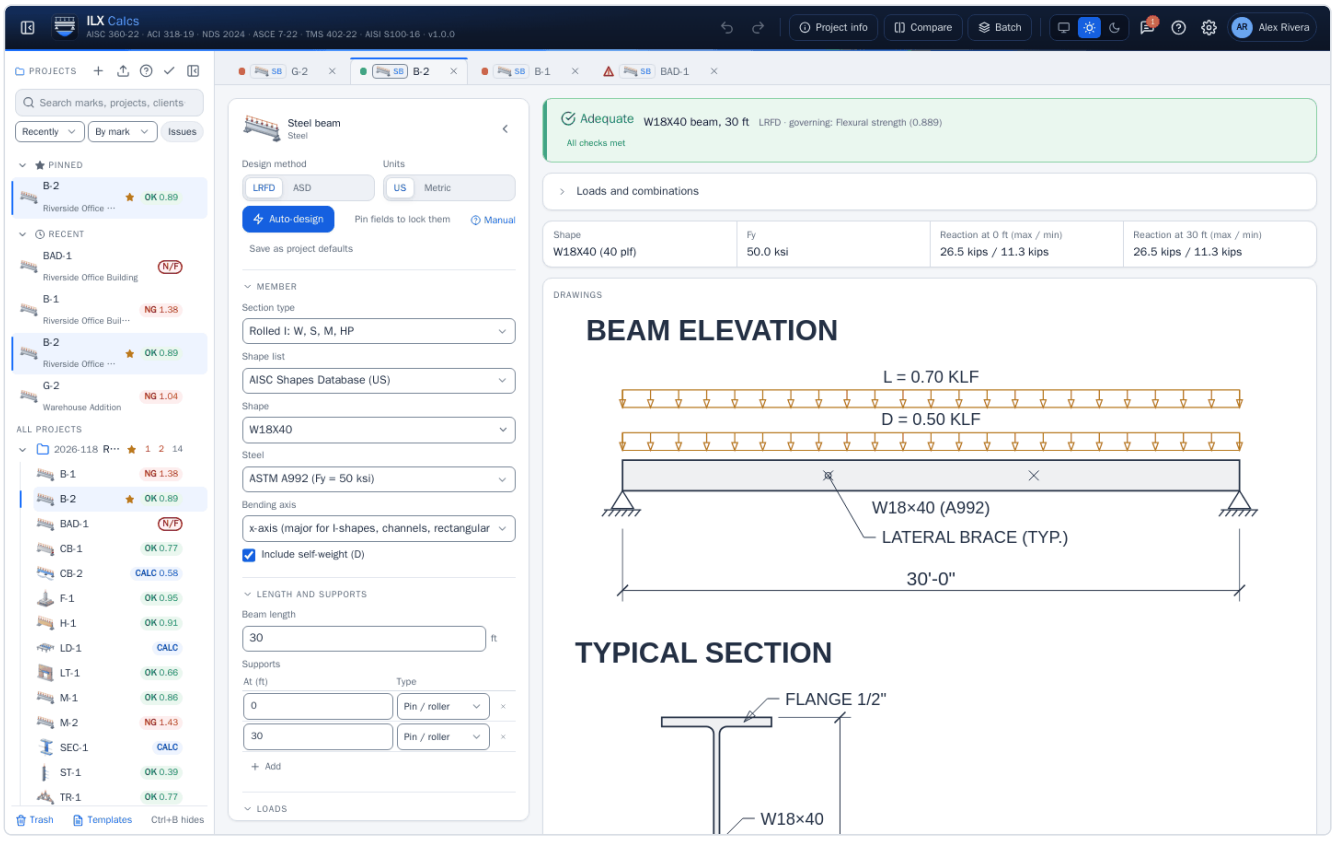
Training 6: Worked calculation, steel

A floor beam, the one auto-design proposed in Training 4.

Inputs (Steel beam, LRFD):

Input	Value
Shape, Steel	W18X40, ASTM A992
Beam length, supports	30 ft, pin / roller at both ends
Line loads	D = 0.5 kip/ft, L = 0.7 kip/ft
Self-weight	Included
Compression flange braced	At a spacing of 10 ft
Deflection limits	Span / 360 live, span / 240 total

Results: *W18X40 beam, 30 ft*, **Adequate**, governing flexure at 0.889.



Check	Clause	Required	Available	Ratio	Governs
Flexural strength	AISC 360-22 F2	198.9 kip-ft	223.7 kip-ft	0.889	1.2D + 1.6L

Check	Clause	Required	Available	Ratio	Governs
Shear strength	AISC 360-22 G2.1	26.5 kips	169.2 kips	0.157	1.2D + 1.6L
Live load deflection	IBC 2021 Table 1604.3 (limit entered)	0.719 in	1 in	0.719	L
Total load deflection	IBC 2021 Table 1604.3 (limit entered)	1.273 in	1.5 in	0.849	D + L

Read it. The factored reaction is 26.5 kips, also the shear. Braces at the third points give the 10 ft unbraced length: try **Continuous** and the flexural strength rises while the deflections stay as they are.

What it does not do. The end connections, bearing, concentrated force checks and the strength and stiffness of the braces are not part of this calculation. See Steel beam, limitations.

Other steel types work the same way. Columns, base plates, composite beams, joists and the rest have their own chapters, listed by subject in the overview.

CHAPTER 10

Training 7: Worked calculation, wood

A built-up header over a 12 ft opening. New calculation, **Sawn lumber beam**.

Inputs:

Input	Value
Species, grade	Douglas Fir-Larch, No. 2
Size, plies	2x12, 2 plies
Fastening between plies	Nails
Length, supports	12 ft, pin / roller at both ends
Line loads	D = 150 plf, L = 300 plf

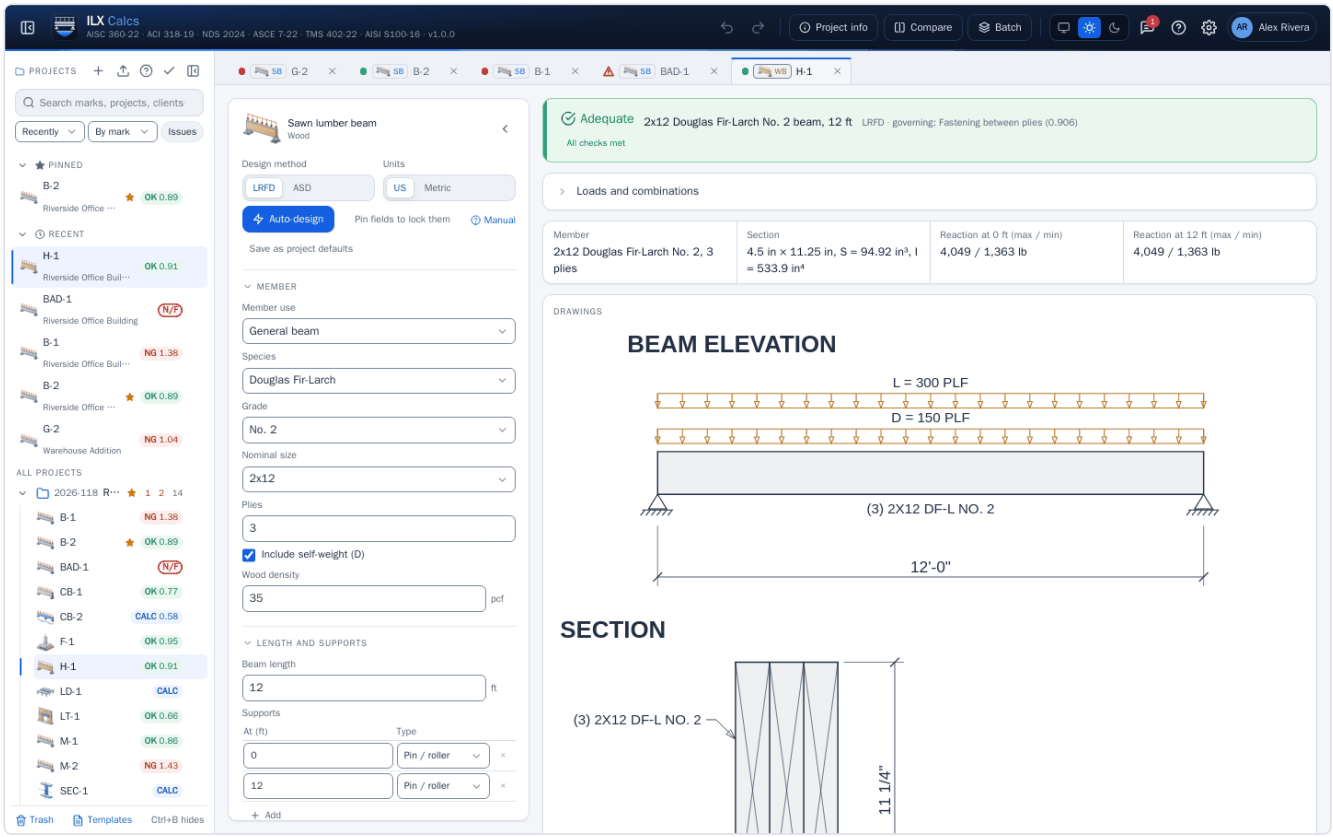
Everything else stays at its default: LRFD, dry service, normal temperature, 3 in. bearing, span / 360 and span / 240.

Two plies

Not adequate. Bending stress is 2,286 psi against 1,788 psi, a ratio of 1.279, under 1.2D + 1.6L. Everything else passes.

Three plies

Change **plies** to 3.



Check	Clause	Required	Available	Ratio
Bending stress	NDS 2024 3.3	1,535 psi	1,788 psi	0.859
Shear stress	NDS 2024 3.4	99 psi	311 psi	0.317
Live load deflection	IBC 2021 Table 1604.3 (limit entered); NDS 2024 3.5	0.164 in	0.4 in	0.41
Total load deflection	IBC 2021 Table 1604.3 (limit entered); NDS 2024 3.5	0.297 in	0.6 in	0.495
Fastening between plies	NDS 2024 12.3, Table 11.3.1	440 plf	486 plf	0.906

Read it. The governing check is now the **Fastening between plies** at 0.906, not bending. A third ply adds a joint: that is why the nailing check exists and why you should open it. The largest reaction is 4,049 lb.

A word on the reference values. The NDS 2024 Supplement is not on file: the wood tables here are 2018 values marked for review, as the chapter says. Enter your own if yours differ. The inputs are in the editor.

CHAPTER 11

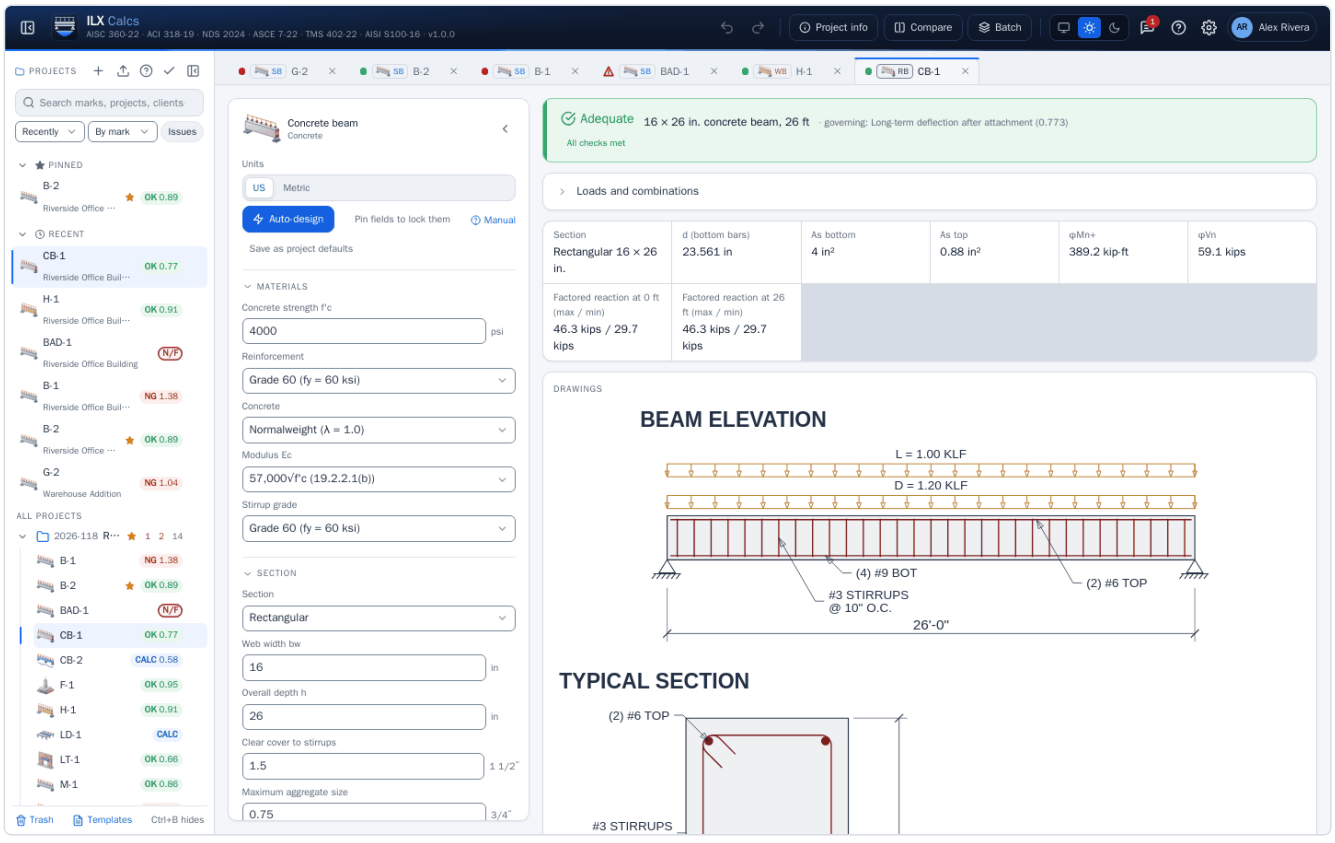
Training 8: Worked calculation, concrete

A simply supported beam. New calculation, **Concrete beam**.

Inputs:

Input	Value
Section	Rectangular, 16 in. wide, 26 in. deep
Materials	f'c = 4,000 psi, Grade 60
Bottom bars	4 #9
Stirrups	#3 at 10 in.
Span	26 ft, pin / roller at both ends
Line loads	D = 1.2 kip/ft, L = 1.0 kip/ft, self-weight on

Results: *16 × 26 in. concrete beam, 26 ft*, **Adequate**. The effective depth to the bottom bars is 23.561 in.



Check	Clause	Required	Available	Ratio
Positive moment strength	ACI 318-19 22.2, 22.3	300.8 kip-ft	389.2 kip-ft	0.773

Check	Clause	Required	Available	Ratio
Shear strength	ACI 318-19 22.5	37.5 kips	59.1 kips	0.635
Long-term deflection after attachment	ACI 318-19 24.2.4	1.005 in	1.3 in	0.773
Stirrup spacing along the beam	ACI 318-19 Table 9.7.6.2.2	10 in	11.78 in	0.849

Read it. The status bar names the long-term deflection as governing, 0.773, the same ratio as the moment. The highest ratio in the table is a detailing row, the stirrup spacing at 0.849. Do not stop at the status bar: read the rows.

Try it. Change the spacing to 12 in. The status turns **Not adequate**: 12 in. is past the 11.78 in. limit, though the shear strength still passes, 37.5 kips against 55.2 kips. That is what the separate rows are for.

Where the numbers come from. The concrete beam is checked against StructurePoint examples written to ACI 318-14, with deviations where 318-19 differs. See the chapter.

CHAPTER 12

Training 9: Worked calculation, foundations

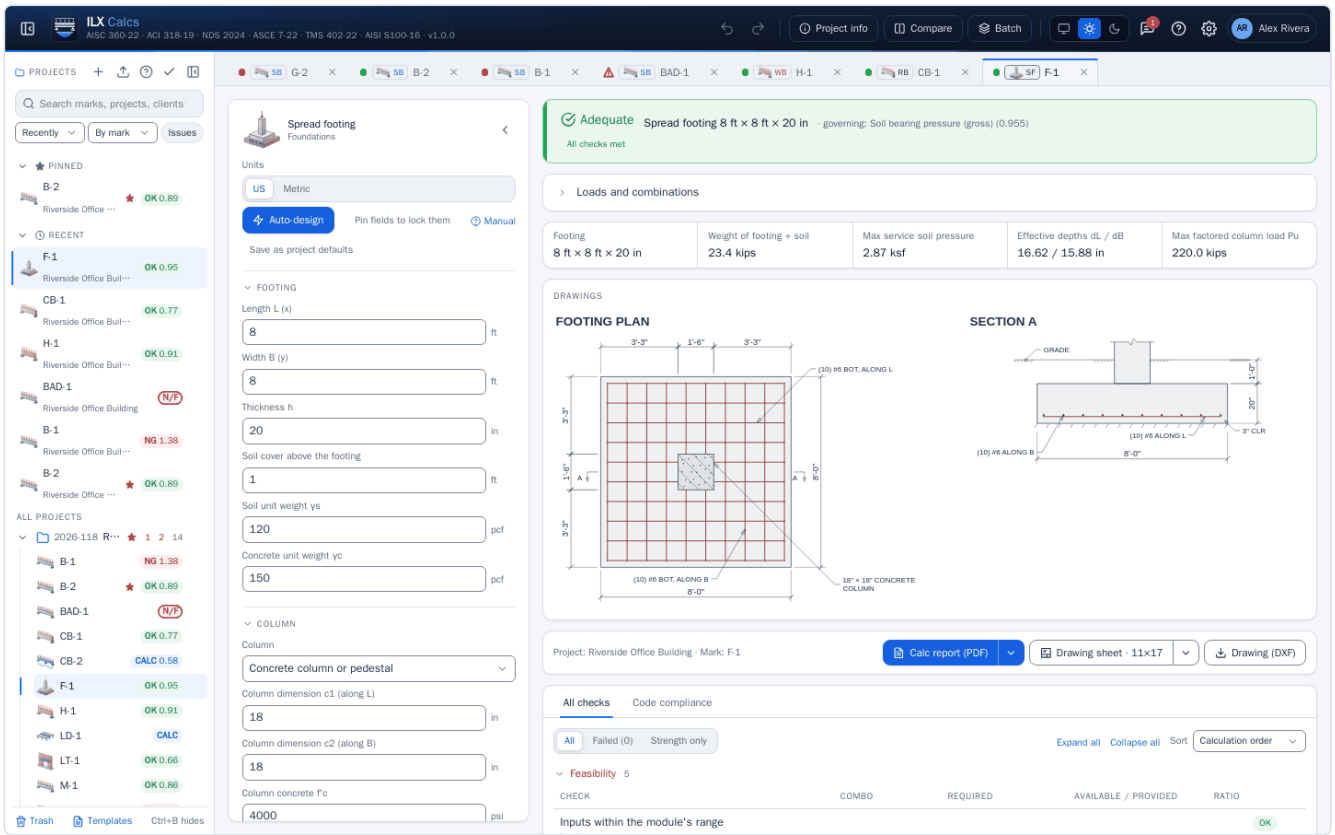
A square spread footing under an 18 in. concrete column. New calculation, **Spread footing**.

Inputs:

Input	Value
Footing	8 ft × 8 ft × 20 in.
Column loads, service	D = 90 kips, L = 70 kips
Allowable soil pressure	3.0 ksf, gross
Bars	10 #6 each way

The rest stays at its default: 1 ft of soil cover at 120 pcf, concrete at 150 pcf, f'c = 4,000 psi, Grade 60, 3 in. cover.

Results: *Spread footing 8 ft × 8 ft × 20 in*, **Adequate**, governing soil bearing.



Check	Clause	Required	Available	Ratio	Governs
Soil bearing pressure (gross)	ACI 318-19 13.3.1.1 (allowable pressure entered)	2.87 ksf	3.00 ksf	0.955	D + L
Minimum flexural reinforcement	ACI 318-19 8.6.1.1 (13.3.3.1)	3.456 in²	4.4 in²	0.785	
One-way shear, span along B	ACI 318-19 22.5.5.1	53.0 kips	82.3 kips	0.644	1.2D + 1.6L

The footing and soil weigh 23.4 kips, included in the bearing check. The factored column load is 220.0 kips.

Read it. Bearing uses service loads with the combination D + L; concrete design uses factored ones. The allowable pressure is your geotechnical engineer's. The program takes the value you enter and states so in the clause.

Let auto-design try. From the default 9 ft footing with the same loads and soil, **Auto-design** with *Lowest cost index* proposes an 8 ft × 8 ft × 18 in. footing with 16 #4 each way, at a bearing ratio of 0.947. A shallower footing with more small bars ranks cheaper on its relative unit costs: check whether you would really build it.

CHAPTER 13

Training 10: Worked calculation, masonry

A reinforced CMU lintel over an 8 ft opening. New calculation, **Masonry lintel and beam**.

Inputs:

Input	Value
Wall	8 in. CMU, f'm = 2,000 psi, Type M or S mortar (the defaults)
Lintel	2 courses
Opening	8 ft, bearing 8 in. each side
Bottom bars	2 #5, embedment past the opening 32 in.
Applied loads	D = 400 plf, Lr = 300 plf, plus the wall above and self-weight (defaults)

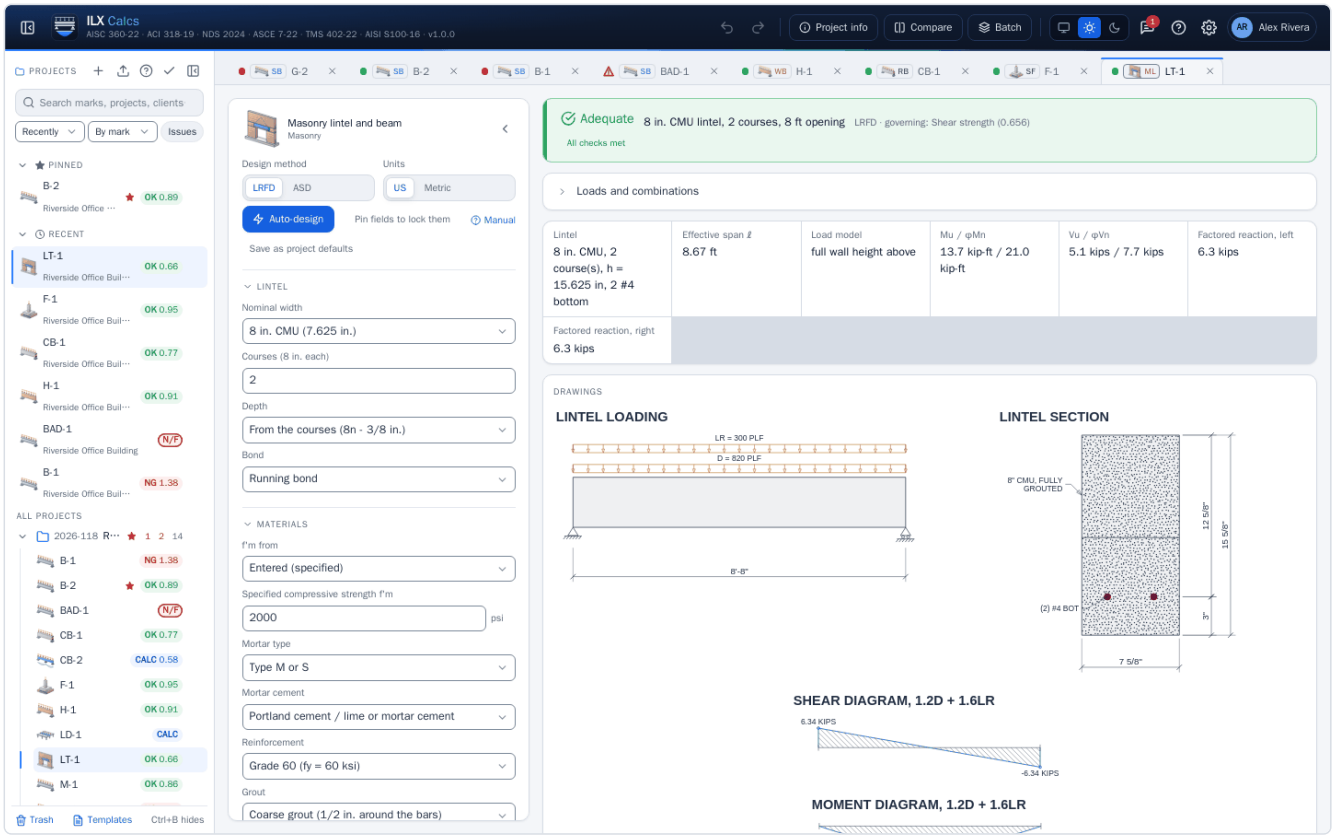
2 #5: not adequate, and not for strength

The moment strength is ample, 31.0 kip-ft, yet the status is **Not adequate**. The failing check is **Bottom bar embedment ≥ ld** (TMS 402-22 6.1.6.3.1): 38.932 in. needed and 32 in. provided, a ratio of 1.217.

This is why you read the rows and not only the governing strength check.

2 #4: adequate

Change the bars to 2 #4.



8 in. CMU lintel, 2 courses, 8 ft opening, **Adequate**. The effective span is 8.67 ft.

Check	Clause	Required	Available	Ratio
Flexural strength (positive moment)	TMS 402-22 9.3.2	13.7 kip-ft	21.0 kip-ft	0.656
Shear strength	TMS 402-22 9.3.3.1.2	5.1 kips	7.7 kips	0.656
Deflection (service)	TMS 402-22 4.6, 5.3.1.6	0.069 in	0.173 in	0.397

The factored reaction at each end is 6.3 kips. Take that to the bearing below.

A plain word on the code basis. TMS 402-22 itself is not on hand: the masonry types are built on TMS 402-13 wording and the TMS cross-reference, as the chapter records. References marked `(to confirm against the code text)` in the checks are clauses not yet confirmed against the 2022 text. Read those yourself until they are cleared.

CHAPTER 14

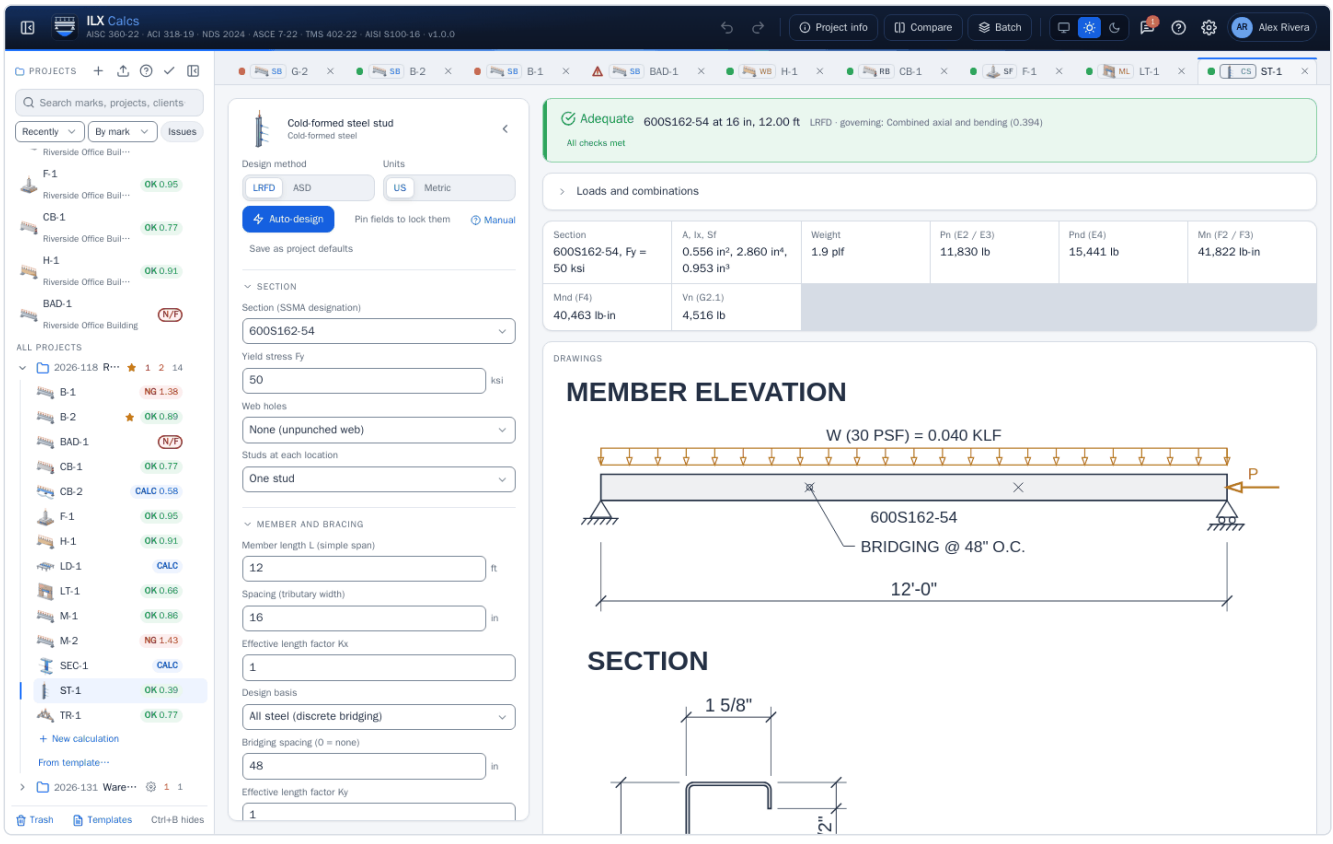
Training 11: Worked calculation, cold-formed steel

A wall stud, axial load and wind. New calculation, **Cold-formed steel stud**.

Inputs. State the **Web holes** setting yourself, here and on your projects: a standard SSMA stud is punched, and the code does not choose for you.

Input	Value
Section	600S162-54, Fy = 50 ksi
Length, spacing	12 ft, at 16 in.
Axial loads	D = 400 lb, L = 1,000 lb
Wind on the wall	W = 30 psf
Web holes	**None (unpunched web)**, then **Punched web**

Unpunched web



600S162-54 at 16 in, 12.00 ft, **Adequate**, governing **Combined axial and bending**, 0.394, under 1.2D + 1.0W + L. The nominal axial strength Pn (E2 / E3) is 11,830 lb, and the shear strength Vn is 4,516 lb. The factored axial load is 2,080 lb under 1.2D + 1.6L.

Punched web

Change **Web holes** to **Punched web**.

	None	Punched
Pn (E2 / E3)	11,830 lb	10,181 lb
Vn (G2.1)	4,516 lb	3,115 lb
Available axial strength, 1.2D + 1.6L	10,055 lb	8,654 lb
Combined axial and bending	0.394	0.419

The stud still passes, with less to spare. Check what your supplier's stud has, and set the input to match.

On the section tables. The listed sections are the SSMA designations. See the chapter for what the module was checked against.

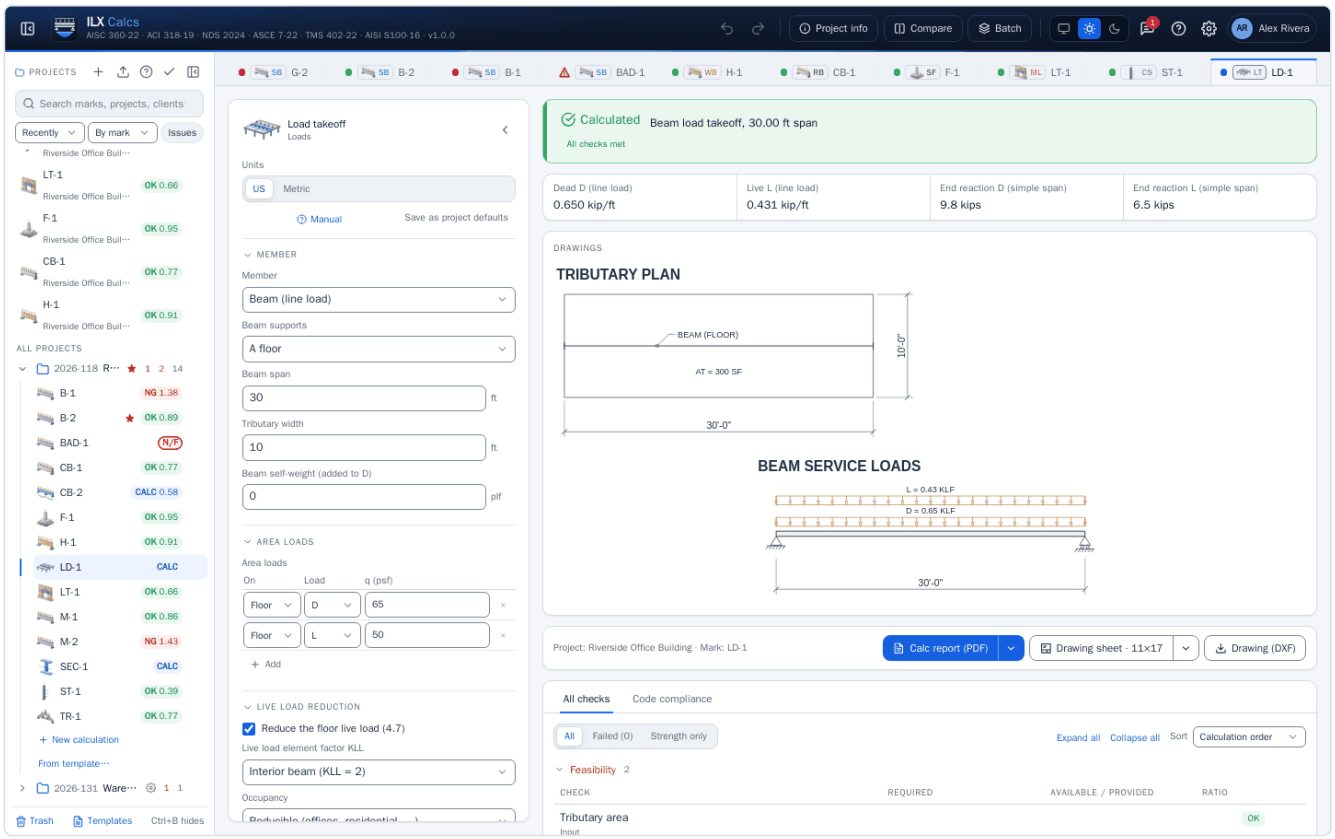
CHAPTER 15

Training 12: Worked calculation, loads

The types in the Loads group give the loads you enter in other calculations. They do not pass or fail: their status is **Calculated**.

A floor beam takeoff

New calculation, **Load takeoff**. A floor beam, 30 ft span, 10 ft tributary width, with 65 psf dead and 50 psf live, live load reduction on (interior beam).



Result	Value
Dead D (line load)	0.650 kip/ft
Live L (line load)	0.431 kip/ft
End reaction D (simple span)	9.8 kips
End reaction L (simple span)	6.5 kips

The tributary area is 300 ft², so ASCE 7-22 Eq. 4.7-1 reduces 50 psf to 43.1 psf. The check row confirms that it stays above the 25.0 psf limit of 4.7.2.

Take these line loads to the beam: they are service values with a load type, exactly what the steel, wood and concrete beams ask for. Training 14 does that.

Combining effects

New calculation, **Load combinations**. Three effects, by load type:

Load type	P	V	M
D	100	5	40
L	60	3	25
W	20	12	90

Result, LRFD	Value	Combination
P, largest	216.000	2a: 1.2D + 1.6L
P, smallest	70.000	5a: 0.9D − W
M, largest	163.000	4a: 1.2D + W + L
M, smallest	-54.000	5a: 0.9D − W

The table lists every combination. Use it to check another program's output, or to size something this app does not design.

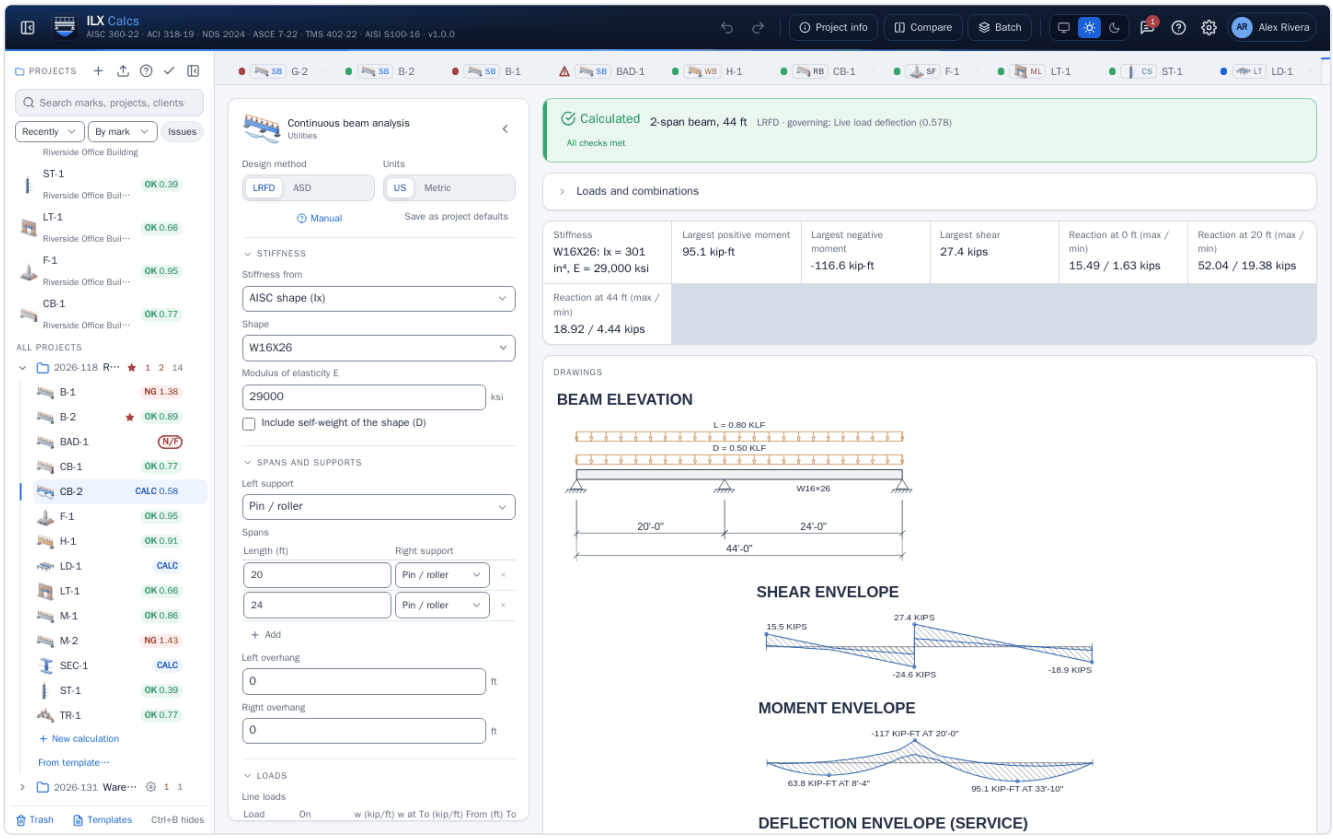
Other load types. Wind, snow, seismic, rain, flood and tornado have their own chapters, each stating what it computes and what it leaves to you. They are tools: site parameters and classifications remain your input.

CHAPTER 16

Training 13: Worked calculation, utilities

Continuous beam analysis

New calculation, **Continuous beam analysis**. Two spans of 20 ft and 24 ft, pin supports, a W16X26 for the stiffness, D = 0.5 and L = 0.8 kip/ft, with live load patterned on the spans.



2-span beam, 44 ft.

Result	Value
Largest positive moment	95.1 kip-ft
Largest negative moment	-116.6 kip-ft
Largest shear	27.4 kips
Reaction at 0 ft (max / min)	15.49 / 1.63 kips
Reaction at 20 ft (max / min)	52.04 / 19.38 kips
Reaction at 44 ft (max / min)	18.92 / 4.44 kips
Live load deflection	0.463 in, ratio 0.578, with L [L on span 2]

Read it. The combination label says which pattern governs. Live load on one span alone gives the largest deflection, which is why the patterns are on. This type is analysis only: it does not design the member, though it does check deflection against your limits.

Converting a value

New calculation, **Unit converter**. Choose *Moment*, 100 kip-ft: 135.582 kN-m, with every other unit of moment listed. It uses the exact factors of NIST SP 811.

Other utilities. Section properties has its own training chapter. See the overview for the rest.

CHAPTER 17

Training 14: A small project

One interior column line of a three-story office building, from the floor load to the footing. Bays are 30 ft × 30 ft, with beams at 10 ft. Floors carry 65 psf dead and 50 psf live; the roof carries 20 psf dead and 20 psf roof live.

Calculations do not link to each other. You carry each result to the next by hand, which is one more moment to think about it. Create one project, and one calculation per step.

1. Floor beam takeoff (`LD-1`, Load takeoff)

Beam, floor level, span 30 ft, tributary width 10 ft, live load reduction on. **D = 0.650 kip/ft, L = 0.431 kip/ft**, as in Training 12.

2. Floor beam (`B-1`, Steel beam)

W18X40, 30 ft simple span, D = 0.65 and L = 0.431 kip/ft, self-weight on, deck bracing the top flange (**Continuous**).

Check	Required	Available	Ratio
Flexural strength	170.7 kip-ft	294.0 kip-ft	0.581
Live load deflection	0.443 in	1 in	0.443
Total load deflection	1.151 in	1.5 in	0.767

The factored reaction is 22.8 kips. The beam's end connections are not part of this calculation.

3. Column takeoff (`LD-2`, Load takeoff)

Column, 3 floors and a roof, 900 ft² each, live load reduction on. The reduced floor live load is 22.7 psf.

Result	Value
Dead D (point load)	193.5 kips
Live L (point load)	61.3 kips
Roof live Lr (point load)	10.8 kips

4. Column (`C-1`, Steel column)

W12X65, A992, 13 ft, K = 1.0 both ways, the three point loads with no moments. **Adequate**: compression demand 335.7 kips under 1.2D + 1.6L + 0.5Lr, against 707.2 kips available, a ratio of 0.475. You could go lighter, and **Auto-design** will say how far.

5. Base plate (`BP-1`, Column base plate)

Column W12X65, plate 18 × 18 × 1.25 in. on a 24 × 24 in. pier, f'c = 3,000 psi, the same three loads. **Adequate**: concrete bearing strength 716.0 kips at a ratio of 0.469, and plate bending of 9.14 kip-in/in against 17.58 kip-in/in, a ratio of 0.52.

6. Footing (`F-1`, Spread footing)

A steel column on an 18 × 18 in. plate, with the same three loads, allowable soil pressure 3.50 ksf gross, 9 ft × 9 ft × 24 in., 11 #6 each way.

Not adequate: 3.57 ksf against 3.50 ksf, a ratio of 1.019. Make it 9.5 ft × 9.5 ft with 12 #6 each way: **Adequate**, 3.24 ksf and a ratio of 0.927. Then flexure along L is 282.7 kip-ft against 480.3 kip-ft, and two-way shear is 96 psi against 190 psi.

Issue the set

In the project's menu, **Calc package (PDF)** gives one report of all six and **All sheets (PDF)** the sheets of those that have them.

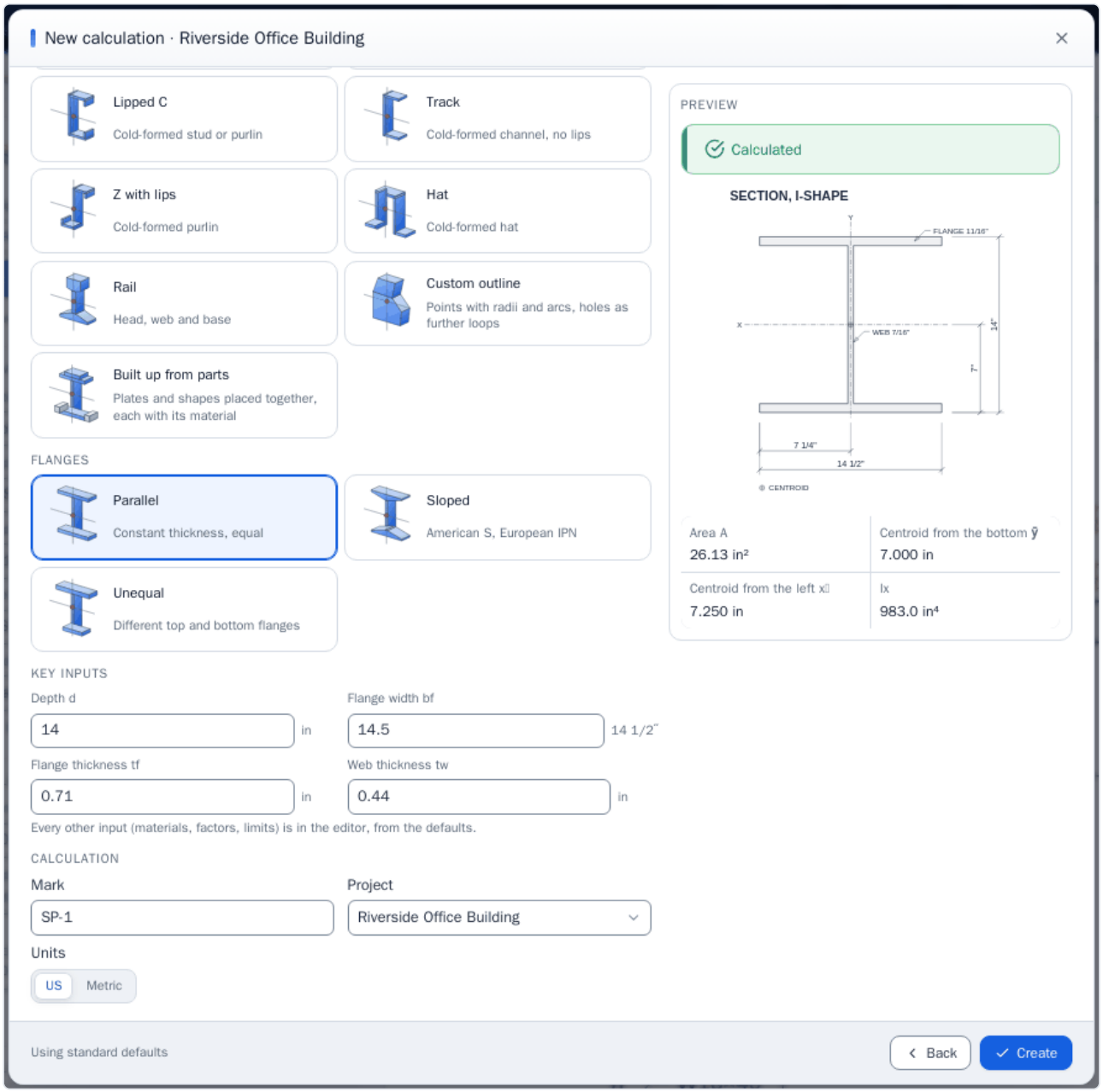
What this chain leaves to you

- The load path itself: which beam frames where, the tributary areas, the girder between the beam and the column. The girder is not in this example.
- Lateral loads, and the column moments they cause.
- Keeping the chain current. If the roof load changes, nothing updates downstream: you must revise each step.

CHAPTER 18

Training 15: The section builder

Section properties computes every geometric property of a cross-section. It has no design code and no LRFD or ASD: its status is **Calculated**. Find it in the Wizard under Utilities.



Four ways to describe a section: a **Standard shape** from the database, a **parametric shape**, a **Custom outline** of points, or a section **Built up from parts**.

Start from a standard shape

Shape *Standard shape (database)*, Region *USA*, Family *W*, Designation *W18X35*.

Property	Value
Area A	10.29 in²
Ix	509.8 in⁴
Sx	57.61 in³
Zx	66.50 in³
J (finite elements, on the true outline)	0.5060 in⁴
J thin-walled	0.4625 in⁴
Cw (finite elements)	1,136 in⁶
Cw thin-walled	1,145 in⁶
Weight	35.03 plf

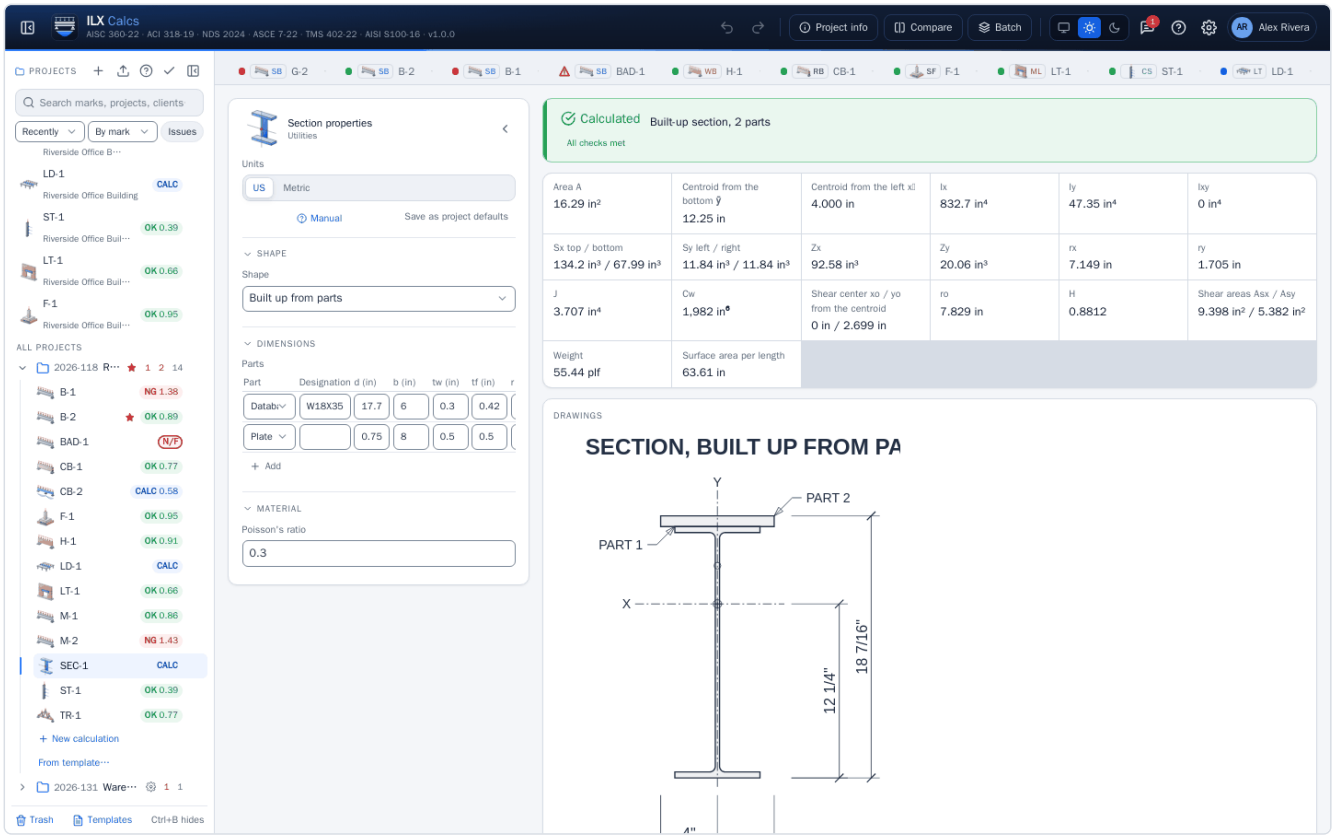
Two values of J and Cw appear because they answer different questions. The finite element value is the true one for the outline, fillets included. The thin-walled value is the classical one on the center line, the basis of AISC's tables and of equations that use them. Use the basis your equation assumes.

Add a cover plate

Shape *Built up from parts*. In the **Parts** table:

Part	Designation	d (in.)	b (in.)	tw, tf	dy (in.)
Database shape	W18X35	17.7	6	0.3, 0.425 (not used for a named shape)	0
Plate		0.75	8	0.5, 0.5 (not used for a plate)	9.225

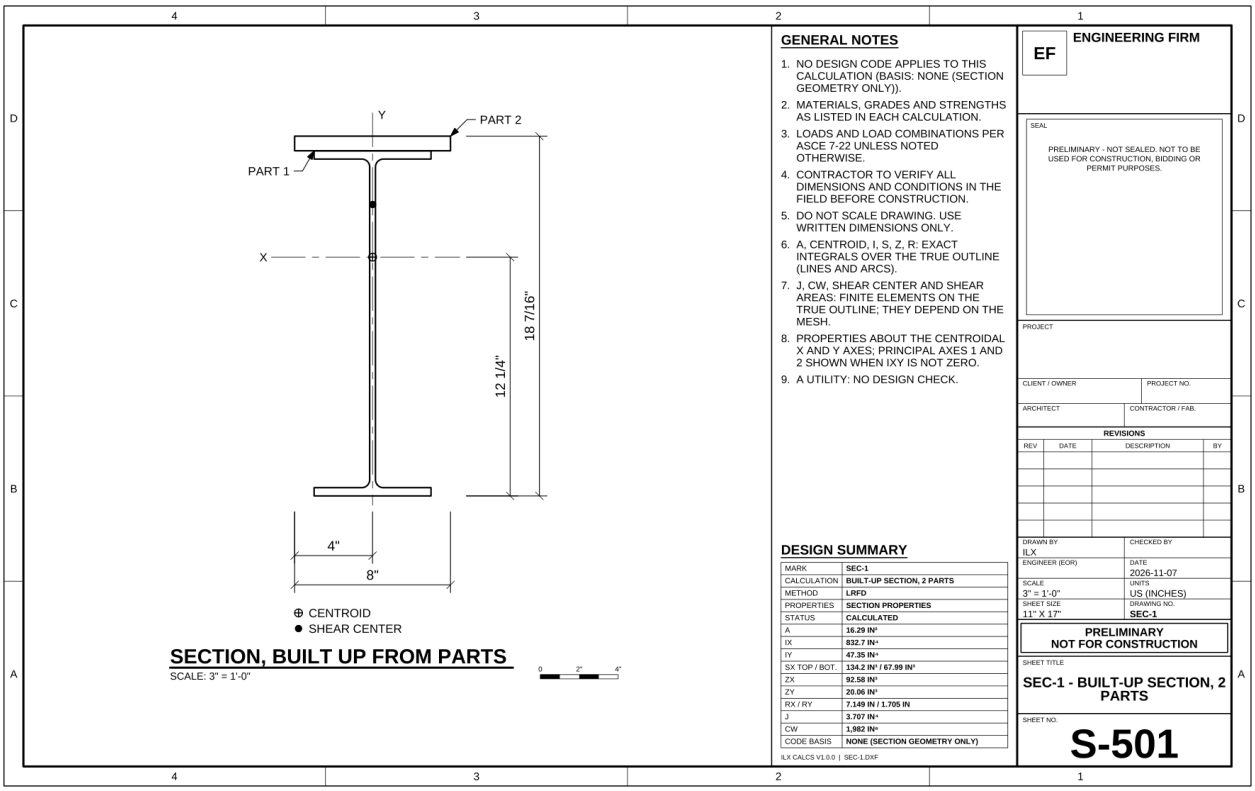
Each part is placed from its own origin: a plate and an I at their centers. The plate sits on the top flange: half the 17.7 in. depth plus half the 0.75 in. plate is 9.225 in. Parts may touch, which welds them into one section, but may not overlap.



Built-up section, 2 parts:

Property	Value
Area A	16.29 in²
Centroid from the bottom ȳ	12.25 in
Ix	832.7 in⁴
Sx top / bottom	134.2 in³ / 67.99 in³
Zx	92.58 in³
J	3.707 in⁴
Cw	1,982 in⁶
Shear center yo from the centroid	2.699 in
H	0.8812
Weight	55.44 plf

Read it. The section is no longer symmetric, so Sx has two values and the shear center moved. The **Mesh check** row compares the finite element area with the exact one: here 0.00000 %. Beyond 0.5 % the mesh is not trusted and the result is Not feasible.



What to do with the numbers

The properties do not flow into another calculation by themselves: enter them where a type asks. The builder has no design code, so it checks neither the parts nor the welds that join them.

See Section properties for every shape and method, and for which results depend on the mesh.

CHAPTER 19

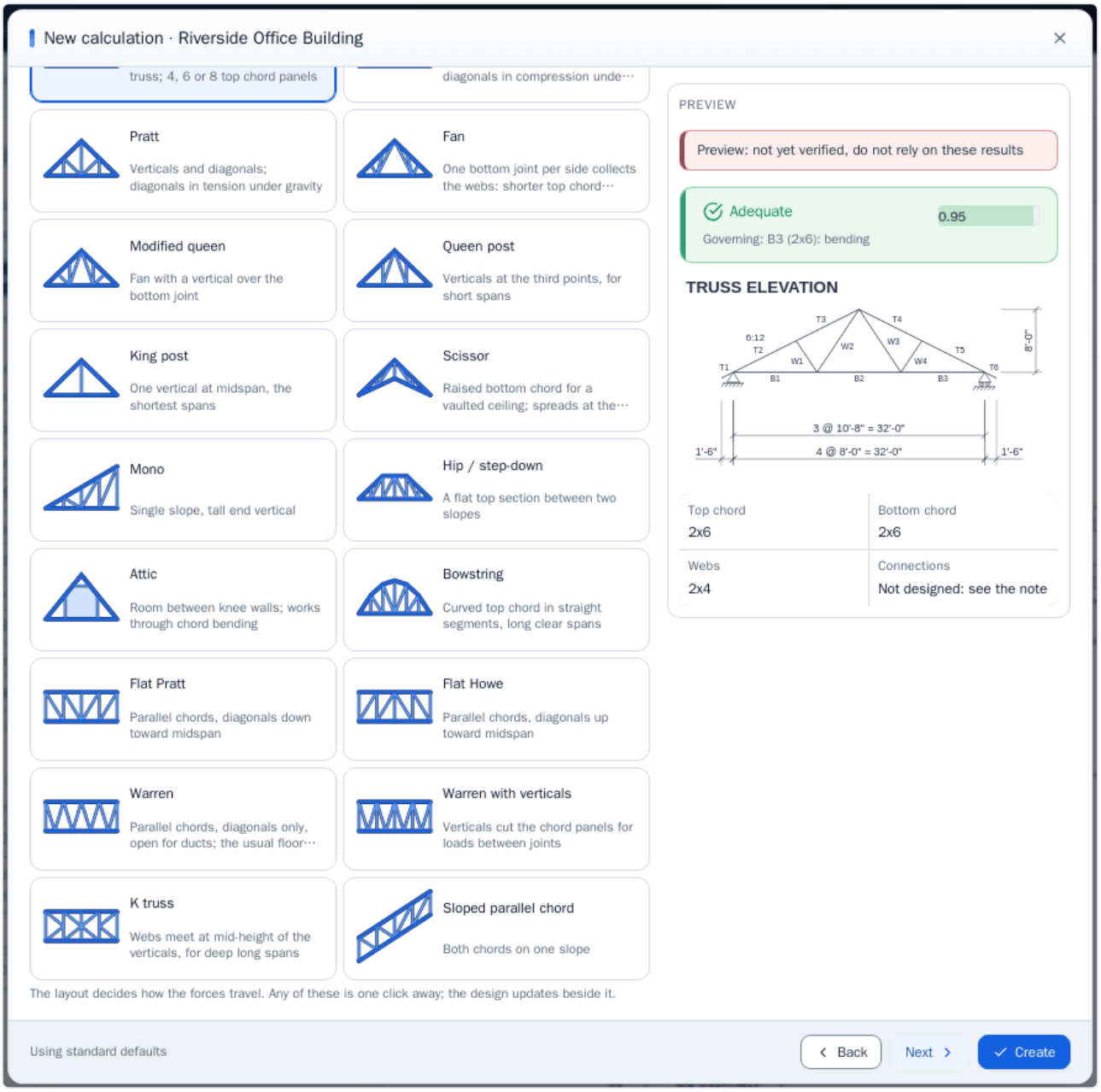
Training 16: The truss designer

Three types design a whole truss: **Wood truss**, **Steel truss** and **Cold-formed steel truss**. They work the same way.

All three carry a label on the screen, the report and the sheet: *Preview: not yet verified, do not rely on these results.* The label is there because no published complete truss example is on file for any of the three: see the verification section of each chapter. Treat a result as a starting point and check it by hand or with your truss supplier's design.

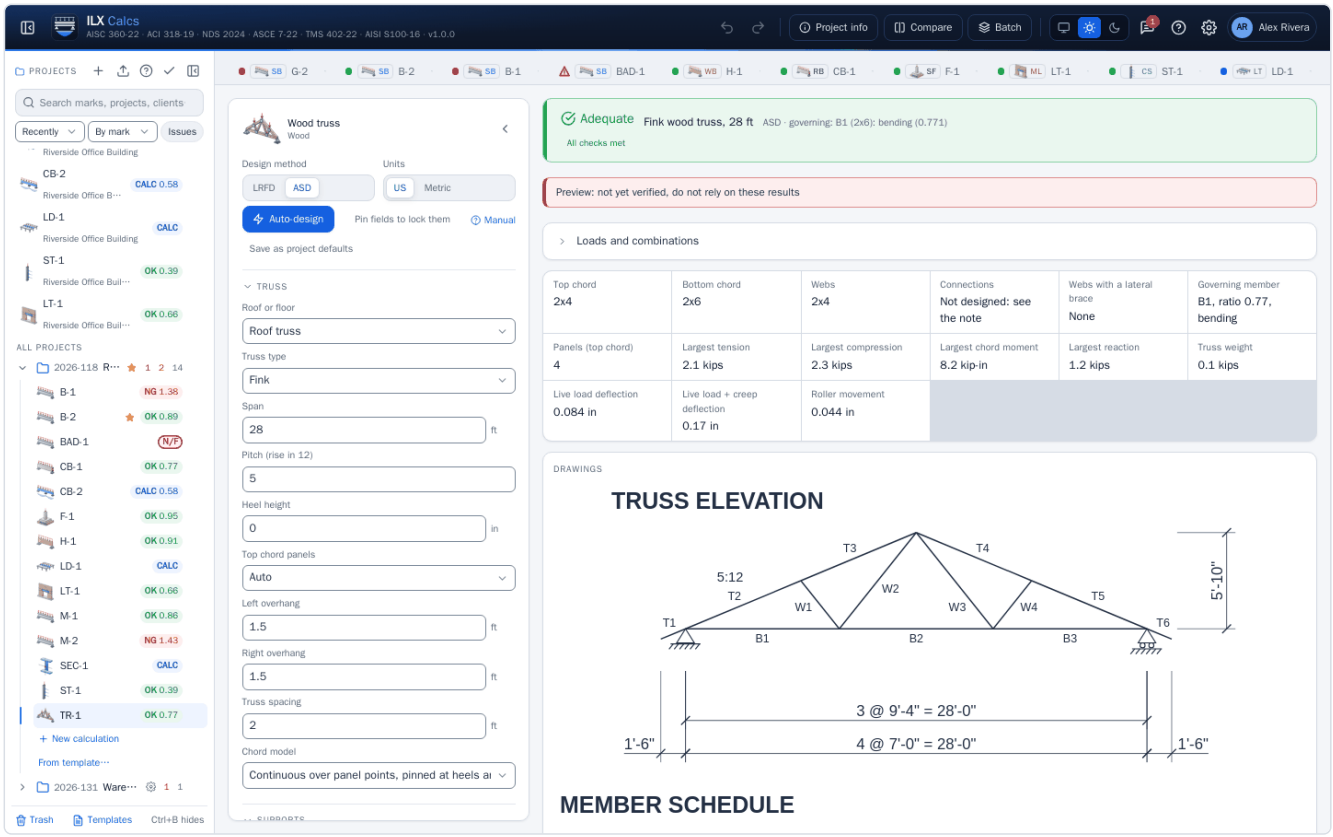
In the Wizard

The truss spreads over pages: **Truss** (roof or floor, and the layout by picture), **Geometry**, **Loads**, **Members and bracing**. The design updates beside every page, and **Create** works from any page.



Worked example: a Fink roof truss in wood

Input	Value
Use, layout	Roof truss, Fink
Span, pitch	28 ft, 5 in 12
Overhangs, spacing	1.5 ft each end (default), trusses at 2 ft
Loads	Top chord dead 10 psf, roof live 20 psf, bottom chord dead 7 psf, no wind
Material	Douglas Fir-Larch, No. 1
Size the members automatically	On

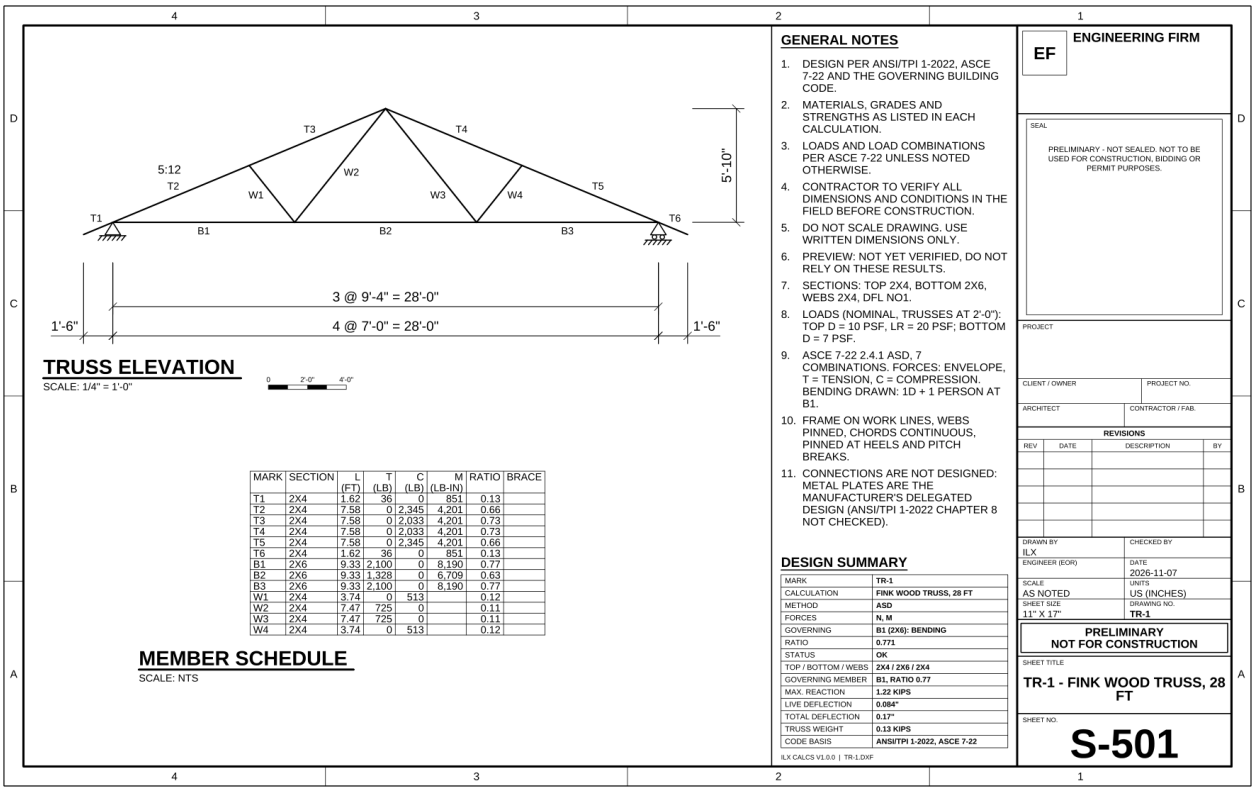


Fink wood truss, 28 ft, **Adequate** by ASD, with the preview label. Automatic sizing chose a 2x4 top chord, a 2x6 bottom chord and 2x4 webs.

Check	Clause	Required	Available	Ratio	Governs
B1 (2x6): bending	ANSI/TPI 1-2022 7.3.3	1,083 psi	1,405 psi	0.771	1D + 1 person at B1
T3 (2x4): axial + bending	ANSI/TPI 1-2022 7.3.5	0.733	1	0.733	D + Lr

Read it. Member marks (T for top chord, B for bottom chord, W for webs) match the elevation and the schedule. The largest tension is 2.1 kips, compression 2.3 kips, chord moment 8.2 kip-in and reaction 1.2 kips. Live load deflection is 0.084 in, and the roller moves 0.044 in.

The bottom chord governs on the person load on an unsheathed chord panel: an input you can review, not an obvious one.



MARK	SECTION	L (FT)	T (LB)	C (LB)	M (LB-IN)	RATIO	BRACE
T1	2X4	1.62	36	0	851	0.13	
T2	2X4	7.58	0	2,345	4,201	0.66	
T3	2X4	7.58	0	2,033	4,201	0.73	
T4	2X4	7.58	0	2,033	4,201	0.73	
T5	2X4	7.58	0	2,345	4,201	0.66	
T6	2X4	1.62	36	0	851	0.13	
B1	2X6	9.33	2,100	0	8,190	0.77	
B2	2X6	9.33	1,328	0	6,709	0.63	
B3	2X6	9.33	2,100	0	8,190	0.77	
W1	2X4	3.74	0	513		0.12	
W2	2X4	7.47	725	0		0.11	
W3	2X4	7.47	725	0		0.11	
W4	2X4	3.74	0	513		0.12	

MARK	TR-1
CALCULATION	FINK WOOD TRUSS, 28 FT
METHOD	ASD
FORCES	N, M
GOVERNING	B1 (2X6): BENDING
RATIO	0.771
STATUS	OK
TOP / BOTTOM / WEBS	2X4 / 2X6 / 2X4
GOVERNING MEMBER	B1, RATIO 0.77
MAX. REACTION	1.22 KIPS
LIVE DEFLECTION	0.084"
TOTAL DEFLECTION	0.17"
TRUSS WEIGHT	0.13 KIPS
CODE BASIS	ANSI/TPI 1-2022, ASCE 7-22

Taking control

- **Size the members automatically** off: enter the sections, and they are checked as they are.
- **Members with their own section:** one heavy web or chord panel, by the mark in the schedule. The rest keep their group's section.
- **Bracing:** what holds each chord and the webs out of plane. *As needed* adds a lateral brace at mid-length of a compression web that fails unbraced, before anything is upsized, and the results name it. These are your judgment, and the braces themselves are your design.
- **Auto-design** varies the three section groups and fixes the result, which turns automatic sizing off. It never changes the geometry, loads, bracing or code options.

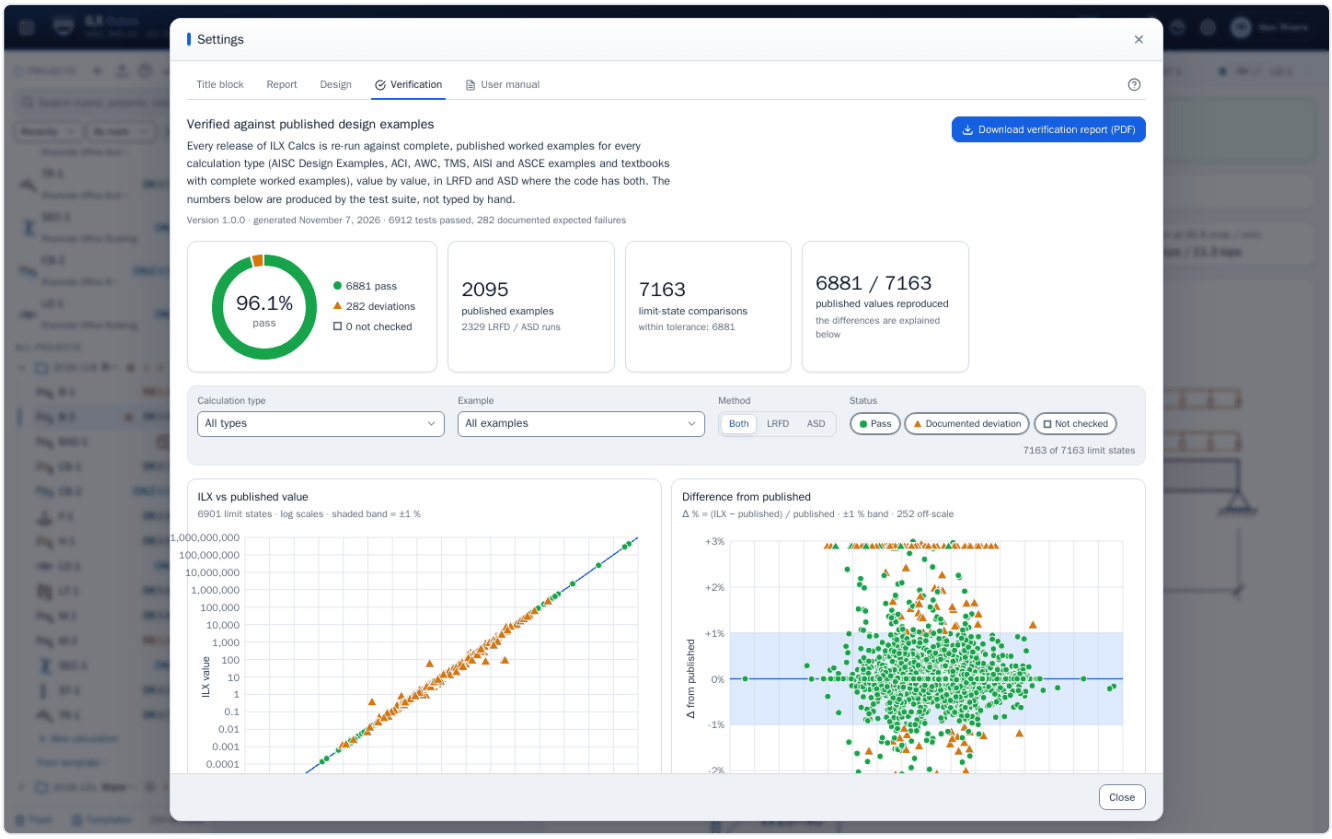
What it does not do

Connections: Not designed: see the note. No plates, gussets, screws or heels. Read the chapter's Limitations: wood, steel, cold-formed steel.

CHAPTER 20

Training 17: The verification report and deviations

Before relying on a calculation type, find out what it was checked against. Everything below is in the app, under **Settings > Verification**, and on the public site.



The numbers, read carefully

This release: 9,654 values compared in 2,841 examples, 2,143 of them published and 698 hand calculations from the cited equations: 9,254 within tolerance, 400 recorded deviations.

Those are counts of tests, not a grade for the program. Weigh them this way:

- **A published example** is someone else's worked calculation. Agreement is real evidence, for the case that example covers. Most are single cases in one set of conditions, many written to an older edition.
- **A hand calculation** is ours, from the cited equations. It catches arithmetic and programming errors. It does not catch a misreading of the code, because the same person read it twice.
- **A table cell** read back (a load table, a shape table) checks that one value.

- **The percentage** on the page mostly reflects how many easy comparisons exist for a type. Read the examples, not the percentage.

The statement in each chapter

Every calculation chapter opens its **Verification examples** section with a status line. For the steel beam:

Verification status: 168 values compared in 37 examples (37 published, 0 hand calculations from the cited equations): 164 within tolerance, 4 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Code basis says how the module was written: *read* (the provisions it uses were read in the target edition), *partial*, *older edition*, *from knowledge* (no document on hand) or *not determined* (no record). ILX Studio plans to verify every module against the purchased references once the module list is complete.

Types with little behind them

No published example, hand calculations only (51): AAC masonry wall, Atmospheric ice loads, Beam grid (grillage), Bond beam chord and collector, Built-up cold-formed steel member, Cold-formed steel bypass and infill framing, Cold-formed steel ceiling framing, Cold-formed steel clip angle, Cold-formed steel floor vibration, Cold-formed steel girt, Cold-formed steel jamb studs, Cold-formed steel parapet, Cold-formed steel shear wall chord studs, Cold-formed steel sloped and mansard wall, Cold-formed steel soffit framing, Cold-formed steel special bolted moment frame, Cold-formed steel storage rack, Cold-formed steel track, Cold-formed steel truss bracing, Concrete stair, Corbelled masonry, Curved and tapered glulam, Dapped-end beam, Fall-protection anchorage, Fluid and hydrostatic pressure, Freestanding masonry wall, Gabion retaining wall, Gravity retaining wall, Helical pile, Joist and waffle slab, Laterally loaded pile (p-y), Ledge and inverted-tee beam, Liquefaction triggering, Liquid-containing tank wall, Masonry arch, Masonry bearing, Masonry chimney, Masonry parapet, Micropile, Modal response spectrum, Movement joints, Notched and tapered member ends, Pile capacity from CPT, Prestressed concrete beam, Roof geometry, Roof ponding, Segmental retaining wall, Seismic member checks, Special moment frame members, Wood shrinkage and moisture, Wood stair.

Thin, 2 published examples or fewer (36): Basement (restrained) retaining wall, Beam with varying stiffness, Braced excavation, Built-up and spaced columns, Cable and catenary, Cold-formed steel track anchorage, Combined footing, Concrete deflection, Continuous concrete beam, Corbel, Coupling beam, Glulam and SCL beam, Ground improvement, Guard or barrier post, Gust-effect factor, Impact and machinery loads, Influence lines, Ledger to masonry wall, Load chasedown, Masonry beam and deep beam, Masonry partition wall, Mat foundation, Monorail beam, One-way slab, Plate girder, Post-tensioned slab strip, Prestressed masonry wall, Seismic site parameters and spectrum, Sheathing-braced cold-formed steel stud, Slope stability, Soldier pile wall, Steel side and knife plate connections, Strap footing, Stress transformation, Unreinforced masonry wall, Wood floor vibration.

Being on this list does not make a type wrong. It means less stands behind it, so check its results more closely.

Reading a deviation

A deviation is a published value the program does not reproduce within tolerance, kept on the record with the reason. They are not all alike. On the Verification page, select the status **Documented deviation** and read each one. Ask of each:

Who differs, and which way? Real examples from this release's record:

- **Steel beam, F.8B.** The example takes a simpler, conservative section; the program follows the specification's wording and reads a little higher: 46.3 against 45.4 kip-ft. A difference of method, stated.
- **Steel column, E.14A.** The example rounds Lc/r up to 127 before finding the stress. Unrounded it is 126.6, so the program reads 52.8 against 52.3 kips. Harmless.
- **Strap footing.** The one free example is a software output with unstated soil weights. None of its four pressures matches within tolerance: differences of up to 6 % that nobody can trace. That is a **weak** check, and the chapter says so.
- **Sheet pile wall, Fig. 4-43.** The program's cantilever moment is 8 % below the printed value. It is on the unsafe side. It comes from equilibrium of the stated pressures, and no line-by-line example is on hand. If this case governs you, check it yourself: see the chapter.

The last one is the kind to look for: **a program value on the unsafe side of a published one.**

What verification does not cover

Inputs outside the examples, combinations of options no example uses, and whatever a type's Limitations leave out. Passing every example does not make a calculation right for your structure. See Limits and your responsibility.

The whole method, with the tolerances, is in Verification.

CHAPTER 21

Training 18: Limits and your responsibility

You are the engineer of record. ILX Calcs is a tool for a licensed engineer who already knows how to do the calculation and can judge the answer.

What the program does

- It applies the code provisions as written, and it cites each one, so you can read it.
- Where a provision leaves a choice to the engineer (a deflection limit, a bracing assumption, a material), the choice is an input with its usual value prefilled, never a hidden assumption.
- It says what it does not know: **Not feasible** with the reason, `(to confirm against the code text)` on a clause not yet confirmed, a **Preview** label on the truss designers, and the verification status of each type.

What stays with you

- **The structural system and the load path.** The program checks one member or element at a time, with what you tell it.
- **The inputs.** Soil values, site parameters, classifications, tributary areas, the end conditions you assumed. A correct calculation of the wrong input is wrong.
- **Whatever a type leaves out.** Every chapter has a **Limitations** section. Read it the first time you use a type. Connections, bearing, stability of the whole, constructability and field conditions are mostly not checked, unless a chapter says they are.
- **The governing code.** The app's editions are stated on each calculation (the header shows the main ones). Your jurisdiction may have adopted others, or amended them.
- **Checking.** Compare with a hand check or a second program, the more so for a type with weak verification (Training 17).
- **Sealing.** Nothing is sealed or approved by the program. Sheets carry the status line of your title block, *Preliminary, not for construction* until you change it.

A checklist before you issue

1. Have I read the type's Limitations?
2. Are the governing checks the ones I expected? If not, why?
3. Did I read the failing and nearly failing rows, not only the status?
4. Is the input summary in the report what I meant? Self-weight, supports and units included.
5. Is any clause marked `(to confirm against the code text)`, or any preview label on? Did I read those clauses?
6. How was this type verified, and does my case resemble those examples?
7. Would I sign this if the program had not been involved?

Help

Use **Support** in the header for questions and problems about the program; include the calculation so we can see what you see. Write to services@ilxstudio.com for support, or to info@ilxstudio.com for anything else.

CHAPTER 22

Steel beam

A beam of any section AISC 360-22 Chapters F and G cover, in bending and shear about one axis, with ASCE 7-22 load combinations: rolled I-shapes (W, S, M, HP), channels (C, MC), rectangular and round HSS, pipe, tees (WT, MT, ST), single and double angles, rectangular bars, and welded built-up I-shapes and plate girders with transverse stiffeners. One model covers simple spans, cantilevers, overhangs, propped and fixed beams and continuous beams. Biaxial bending stays in the steel column.

Inputs of the steel beam

- **Section type:** the family. The Shape list shows that family's AISC shapes (AISC Shapes Database v16.0). A built-up I takes its plates: web depth h between the flanges and thickness tw, width and thickness of each flange (the two flanges may differ). A bar takes its depth and thickness.
- **Steel:** the grade must suit the shape, as the AISC Manual lists them (Tables 2-4 and 2-5): rolled shapes A992, A572 Gr. 50, A913 Gr. 65, A36; HSS A500 Gr. C or B, A1085; pipe A53 Gr. B; plates and bars A36 (up to 8 in.), A572 Gr. 50 (up to 4 in.). A steel that does not suit reads NOT FEASIBLE with the choices. A992 is still the default, so a new built-up I or HSS starts by asking you to choose.
- **Bending axis:** x or y, and for a single angle also w (major principal) or z (minor principal). The load acts perpendicular to the axis chosen.
- **Stem or web legs (tees, double angles):** pointing down or up. **Toe of the leg (single angles):** pointing up or down; for the w-axis this is the long leg's toe, for the z-axis both toes. Positive moment puts the top of the section in compression, and a section whose strength depends on the sign is checked at each sign, as F1 asks in reverse curvature.
- **Lateral-torsional restraint (single angles):** none between the braces, or at the point of maximum moment only (F10.2). This is your judgment. Continuous restraint is the bracing choice "continuous".
- **Include self-weight:** added to the dead load. A built-up I counts its flanges and web, not its stiffeners.
- **Length and supports:** the beam length and any number of supports, each a pin (roller) or a fixed end, at any position. Two pins at the ends make a simple span; a fixed end alone makes a cantilever; supports inside the ends make overhangs; three or more make a continuous beam.
- **Loads:** line loads (uniform, partial or linearly varying; *From* = *To* = 0 loads the whole length; leave *w at To* blank for a uniform load) and point loads, each with its load type (D, L, Lr, S, R, W, E). See Loads and combinations.
- **Lateral bracing:** the compression flange is braced continuously, at the supports only, at a spacing, or at given points. Cb is computed by Eq. F1-1 from each combination's moment diagram, or entered. An unbraced cantilever segment uses Cb = 1.0. Brace points are taken to brace both flanges. Tees and double angles do not use Cb (F9.2).
- **Web and stiffeners (built-up I):** no transverse stiffeners, a spacing (with one at each end of the beam) or given positions. A panel counts as stiffened only with a stiffener on both sides, so include the beam ends if they have one; a panel that is not, or with a/h above 3, uses kv = 5.34. a is the spacing, center to center. Stiffener width and thickness, pairs or one side. Tension field action in interior panels is your choice (G2.2 is permitted, not required) and is off by default.

- **Deflection limits:** span / n for live load and total load; IBC 2021 Table 1604.3 floor values (360 and 240) are prefilled and 0 turns a check off. For a cantilever the span is twice the overhang (Table 1604.3 note h).

Auto-design (editor toolbar) varies the shape of the given family, lightest first (W-shapes as before, optional depth limits; an HSS stays rectangular or round; a double angle keeps its leg arrangement and gap); steel grade when unlocked, among those that suit the family. For a built-up I it varies the web and flange thickness over standard plates, keeping the depth h and the plate widths as you entered them, the bottom flange following the top flange's change; it does not search the depth. A bar is not sized. Objectives: lightest (steel weight); shallowest, then lightest. Kept unless you select them: steel. Every other input is kept; see Auto-design.

Shapes from other markets (extra choices). The Shape list can change from the AISC Shapes Database to the section database: region, family and designation, as the section builder picks them. In: rolled I-shapes with parallel or tapered flanges (HE, IPE, IPN, UB, UC, H, ISMB and similar) with a compact web; channels (UPE, UPN, PFC and similar) under the channel section type; single angles under the angle section type; tees under the tee section type. Double angles, hollow sections and cold-formed sections are not offered on this type, and a saved one reads NOT FEASIBLE. US shapes stay on the AISC list, with their data and results unchanged.

The design code stays AISC 360. Fy and Fu are your inputs, with no assumed grade and no default: a shape from another market is checked to AISC 360 with your material, not to its own national code, and the sheet says so. Whether that material suits AISC 360 (Section A3.1) is your judgment. Properties are the database's, calculated from nominal dimensions; Cw, ro and H are thin-walled and J by finite elements on the true outline, the basis AISC 360's equations assume. The designation shows as the market prints it, in US and metric. Without a shape database on the server the choice does not appear. **Auto-design** stays on the AISC shapes unless you chose another family; then it searches that family only, lightest first, with your Fy and Fu.

Checks performed

Section	Flexure, each unbraced segment and combination	Shear
W, S, M, HP, x-axis	F2 yielding and lateral-torsional buckling (Eqs. F2-1 to F2-6); F3 flange local buckling for noncompact or slender flanges (Eqs. F3-1, F3-2)	G2.1: φv = 1.00 and Cv1 = 1.0 when h/tw ≤ 2.24√(E/Fy), else φv = 0.90 with Cv1 by Eqs. G2-3 / G2-4 (kv = 5.34)
I-shapes, y-axis	F6	G6, both flanges
C, MC, x-axis	F2 with c of Eq. F2-8b	G2.1(b), φv = 0.90
C, MC, y-axis	F6	G6
Rectangular HSS, x or y	F7 yielding, flange and web local buckling (a slender flange by its effective width), lateral-torsional buckling when bent about the major axis of a rectangle	G4
Round HSS, pipe	F8	G5, with Lv from the shear diagram
WT, MT, ST, x-axis	F9: yielding (stem in tension or in compression), lateral-torsional buckling (a stem in compression anywhere along the length sets it), flange and stem local buckling	G3 on the stem
WT, MT, ST, y-axis	F12: Fy times the smaller Sy, the flange's stress of Eq. F6-4 when it is slender, taken as the local buckling stress by analysis	G6, the one flange
Double angle, x-axis	F9 with F10.3 and Eqs. F10-2, F10-3 where F9 sends you there; J is twice the single angle's	G3, one per angle, on the web legs

Section	Flexure, each unbraced segment and combination	Shear
Single angle, geometric axis	F10: 1.5 My, F10-5a or F10-5b with the reductions of F10.2 for an equal-leg angle, none with continuous restraint; F10.3 leg local buckling	G3 on the leg in the plane of bending
Single angle, w-axis	F10-1, F10-4 with βw of Commentary Table C-F10.1 and Cb at most 1.5, F10.3	G3 on the short leg (see below)
Single angle, z-axis	F10-1, no lateral-torsional buckling, F10.3 for both toes	G3 on the short leg
Rectangular bar, x or y	F11	G3 with kv = 1.2
Built-up I, x-axis	Compact web, doubly symmetric: F2 and F3 with the flange limits of Table B4.1b Case 11 and kc. Noncompact web or singly symmetric: F4 (Rpc, Rpt, FL, rt, tension flange yielding). Slender web: F5 (Rpg). F13.2 proportioning limits	G2.1(b) panel by panel over the combinations, with G2.2 in interior panels if you ask for it; the larger of the two
Built-up I, y-axis	F6, the wider flange ratio and the smaller Sy	G6, both flanges
Transverse stiffeners		G2.4: width-to-thickness (Eq. G2-16) and moment of inertia (Eqs. G2-17 to G2-19), the governing panel's ρw

Live load deflection takes each of L, Lr, S and R alone (service), and total load deflection the ASD gravity combinations (service), both against span / n, with the moment of inertia of the axis chosen. A single angle bent about a geometric axis without continuous restraint uses the moment of inertia divided by 1.82, as Commentary F10 says for its total deflection.

The reactions at each support (largest and smallest over the combinations) are listed with the key results. The diagrams show the shear and moment of the combination that governs flexure and the deflection of the governing service combination.

Limitations

- Bending about one axis. Biaxial bending stays in the steel column; so does the H2 stress ratio of a single angle with moments about both principal axes (Example F.11C's last step).
- No axial load, torsion, web openings, concentrated-force checks (J10), bearing or holes in the tension flange (F13.1). A channel loaded in the plane of its web is eccentric to the shear center: torsion is not checked here (see the steel beam with torsion).
- Reads NOT FEASIBLE, with the reason: a rolled I-shape or channel with a web or a channel flange that is not compact (F4 and F5 are built for built-up I only; none of the AISC shapes offered reaches this at the grades offered); a rectangular HSS with a slender web (F7.3(c)); a round section with D/t of 0.45 E/Fy or more; a built-up I outside F13.2; an unequal-leg angle on a geometric axis without continuous restraint; a double angle about its y-axis; a tee about its y-axis with Iy above Ix and without continuous bracing (F12 would need a buckling stress by analysis).
- Built-up I: welded, one grade, no cover plates, no hybrid girders. Flange-to-web welds, stiffener welds and their cutoffs (G2.4(b), (c)) and bearing stiffeners at reactions (J10.8) are not part of this calculation.
- **End panels stay on G2.1.** G2.3 permits a tension field in an end panel too; it is not built. Design Examples G.8A and G.8B do the same. In an end panel the value is on the safe side.
- Solid round bars are not offered: Chapter G has no section for their shear.

- Shear of a single angle bent about a principal axis divides between the legs; the program takes the short leg alone, as a conservative reading of G3. No published example.
- Lateral-torsional buckling uses the unbraced segments between brace points; bracing strength and stiffness (Appendix 6) are the engineer's responsibility.

Verification examples

Verification status: 168 values compared in 37 examples (37 published, 0 hand calculations from the cited equations): 164 within tolerance, 4 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Examples v16.0, in LRFD and ASD:

Example	What it checks	Published	ILX Calcs
F.1-1A	W18×50, 35 ft, continuously braced	φMn = 379 kip-ft, Mn/Ω = 252 kip-ft	378.8, 252.0
F.1-2B	Braced at the third points	Cb = 1.01, φMn = 304 kip-ft	1.014, 306.5
F.1-3B	Braced at midspan	Cb = 1.30, φMn = 288 kip-ft	1.299, 287.6
F.3B	W21×48, noncompact flanges	φMn = 398 kip-ft, Mn/Ω = 265 kip-ft	397.9, 264.7
G.1B	W24×62 shear	φVn = 306 kips, Vn/Ω = 204 kips	305.7, 203.8
F.2-1B	C15×33.9, continuously braced	φMn = 191, Mn/Ω = 127 kip-ft	190.5, 126.7
F.2-2B	C15×33.9, Lb = 5 ft, Cb = 1.0	173, 115	172.3, 114.7
F.5	W12×58 about the y-axis	122, 81.4	121.9, 81.1
F.6	Square HSS (Manual table value)	7.20, 4.79	7.20, 4.79
F.7B	HSS10×6×3/16, noncompact flange	59.7, 39.7	59.7, 39.7
F.8B	HSS8×8×3/16, slender flange	45.4, 30.2 (be = 6.33 in.)	46.3, 30.8 (be = 6.33 in.), a recorded deviation
F.9B	Pipe 8 x-Strong	81.4, 54.1	81.4, 54.1
F.10	WT5×6, stem in tension	7.32, 4.87	7.32, 4.87
F.11A	L4×4×1/4, no restraint	3.69, 2.46	3.69, 2.46
F.11B	L4×4×1/4, restrained at midspan	3.98, 2.65; lateral-torsional buckling Mn = 5.53	4.97, 3.31, a recorded deviation; 5.53
F.11C	L4×4×1/4, the strengths about z and w	4.13, 2.75; 7.49, 4.98	4.13, 2.75; 7.48, 4.98
F.12	Bar 5 × 3 in., Lb = 6 ft	70.4, 46.8	70.3, 46.8
F.14	Plate girder, noncompact web	7,880, 5,250; Lp = 98.3, Lr = 369 in.; φVn = 440, Vn/Ω = 293 kips; Δ = 0.944 in.	7,896, 5,257; 98.4, 369; 439.9, 292.9; 0.944
G.2B	C15×33.9 shear	φVn = 162, Vn/Ω = 108 kips	162.0, 107.8
G.3	L5×3×1/4 shear	33.8, 22.5	33.8, 22.5
G.4	HSS6×4×3/8 shear	93.6, 62.3	93.3, 62.1
G.5	HSS16.000×0.375 shear	232, 155	232.2, 154.5
G.6	W21×48 minor-axis shear	189, 126	189.0, 125.8

Example	What it checks	Published	ILX Calcs
G.7	C9×20 minor-axis shear	59.1, 39.3	59.1, 39.3
G.8A	Built-up, no stiffeners	176, 117	176.1, 117.2
G.8B	End panel a = 42 in.; second panel a = 90 in. with tension field	217, 144; 185, 123	216.7, 144.2; 185.7, 123.6

AISC Steel Construction Manual, 16th Ed., Table 3-2 (p. 3-25), each shape continuously braced and as a 2Lr span braced at midspan with Cb = 1.0 (Lb = Lr), LRFD and ASD:

Shape	φbMpx	φbMrx	Lp / Lr (ft)	φvVnx	ILX Calcs
W21×55	473	289	6.11 / 17.4	234	472.5, 287.8, 6.11 / 17.38, 234.0
W18×40	294	180	4.49 / 13.1	169	294.0, 179.6, 4.49 / 13.10, 169.2
W16×40	274	170	5.55 / 15.9	146	273.8, 169.5, 5.55 / 15.89, 146.4
W14×43	261	164	6.68 / 20.0	125	261.0, 164.5, 6.68 / 20.03, 125.4
W12×65 (noncompact flange)	356	231	— / 35.1	142	356.2, 231.0, — / 35.15, 141.6

Moments in kip-ft, shears in kips; the ASD values (Mpx/Ωb, Mrx/Ωb, Vnx/Ωv) agree in the same way. For W12×65 the Manual lists Lp′ (where lateral-torsional buckling reaches the flange local buckling strength, Manual Eq. 3-2), not the Eq. F2-5 value ILX reports, so Lp is not compared.

The F.1-2B value is 0.8 % above the Example because the Example rounds Cb to 1.01 before using it.

Two published values are recorded as deviations, each with its reason on the Verification page. **F.8B:** the Example takes a simpler, conservative section (the ineffective width off both flanges), the program follows F7.2(c) (off the compression flange, the neutral axis moving): 46.3 against 45.4 kip-ft; the effective width agrees. **F.11B:** the Example reuses a leg local buckling strength with the 0.80 of an unrestrained angle and says that is conservative; the program applies 0.80 only without restraint, so lateral-torsional buckling governs: 4.97 against 3.98 kip-ft, and the lateral-torsional buckling value agrees (5.53).

By hand only, from the equations, because no published example applies: S, M and HP shapes (F2, F3, G2.1); tee stems and double angle web legs in compression, and lateral-torsional buckling of tees and double angles (F9); unequal-leg angles on the principal axes (F10-4 with βw, F10.3 for each toe); bending about the y-axis of a tee (F12); singly symmetric built-up I (F4, tension flange yielding, Case 16) and doubly symmetric with a slender web (F5); transverse stiffeners (G2.4); the section values of a built-up I against closed forms. Example F.13 (a round bar) is out of scope.

Shapes from other markets: no published example applies AISC 360 to a non-US shape, so nothing here is checked against one. Hand calculations from the plain dimensions of HEB 300, SHS 200x200x10, CHS 273x10 and UPE 200; and the database path run on US shapes against the AISC path (W14X90, W21X50, W8X31, W30X90 within 1.5 %; HSS8X8X1/2, HSS12X6X1/4, HSS6.625X0.280 within 3 %; C15X33.9, MC12X35, WT8X25, MT6X5, L4X4X1/4, L6X4X1/2 within 3 %, 6 % where lateral-torsional buckling is involved, because J differs).

CHAPTER 23

Steel column

A steel member in axial compression or tension with bending and shear, to AISC 360-22 Chapters D, E, F, G and H, with ASCE 7-22 load combinations and the approximate second-order analysis of Appendix 8. Families: W, S, M, HP, C, MC, WT, MT, ST, rectangular or square HSS, round HSS, pipe, single angles, double angles, two tees flange to flange, built-up I from plates (equal or different flanges) and a single plate or bar, with pin and eyebar ends.

Inputs of the steel column

- **Member:** first what the member is: one rolled shape, HSS or pipe; a single angle; a double angle; two tees flange to flange; a built-up I from plates; or a plate or bar. Then the shape (AISC Shapes Database v16.0) or the plates, and the steel. W, S, M, HP, C, MC, L and tees take A992, A572 Gr. 50, A913 Gr. 65 or A36 (Manual Table 2-4; a tee takes its parent's grades); HSS take A500 Gr. C (Fy = 50 ksi), A500 Gr. B (46 ksi) or A1085 (50 ksi); pipe takes A53 Gr. B (35 ksi); plates take A572 Gr. 50 or A36 (Table 2-5). A steel that does not suit the section is reported as not feasible, so a built-up member needs its grade changed from the A992 default.
- **Single angle:** the design basis. E5(a) for an individual member or a web of a planar truss, E5(b) for a web of a box or space truss, with the connected leg (long or short), and your statement that requirements (1) to (3) of E5 are met: loaded at the ends through one leg, welded or at least two bolts, no transverse load between. The other two requirements are computed: Lc/r up to 200, and a long leg less than 1.7 times the short. If any of them fails the calculation does not go on: E5 sends the member to Chapter H. **Chapter H** is the third basis: the eccentricities ew and ez of the load about the principal axes, signs as in Example E.14B (positive ew compresses the toe of the long leg, positive ez the heel), and moments Mx and My of the loads, read as moments about w and z.
- **Double angles and double tees:** the number of intermediate connectors (a = L/(n + 1)), whether they are welded or pretensioned (Eq. E6-2) or bolted snug-tight (Eq. E6-1), and your statement that the end connection is welded or pretensioned, which E6.1 requires. For two tees, also the gap between the flanges. The separation of a double angle is in its name.
- **Built-up I:** the width and thickness of each flange, the web depth and thickness. Different flanges make it singly symmetric.
- **Plate or bar:** width and thickness, and the end: plain, pin-connected (pin, hole, distance a, edge distance) or eyebar (pin, hole, edge distance, head diameter, radius of transition).
- **Lengths and bracing:** the member length L and, for each buckling axis, the unbraced length and the effective length factor (Kx Lx, Ky Ly, and Kz Lz for torsional buckling of W-shapes). K is the engineer's input (Commentary Table C-A-7.1 or Appendix 7); with the direct analysis method K = 1.0 (Section C3). Lb is the unbraced length of the compression flange for lateral-torsional buckling.
- **Loads:** either service loads by load type (D, L, Lr, S, R, W, E), combined by ASCE 7-22 2.3.1 / 2.4.1 (W and E in both directions), or required strengths entered per case (factored for LRFD). Each row has the axial force P (compression positive, tension negative), the moments at the top and the bottom as moment-diagram ordinates (the same sign at both ends is single curvature), the mid-height moment of loads applied along the member ($wL^2/8$, added to the end-moment line), and optionally the shears Vx and Vy (blank: taken from the moments as $|M_{top} - M_{bot}|/L + 4|M_{mid}|/L$).

- **Second-order effects:** either the moments come from a second-order analysis, or the approximate method of Appendix 8 is applied: B1 = Cm/(1 − αPr/Pe1) ≥ 1 on the no-translation moments, with Cm from the end moments (Eq. A-8-4) or 1.0 when there is transverse loading, Pe1 = π²EI*/L² (K1 = 1.0) with EI* = 0.8τbEI for the direct analysis method or EI for the effective length method. B2 is entered from the story analysis and multiplies the axial force and moments of the W and E load types (taken as the lateral-translation effects Plt and Mlt).
- **Flexure of a tee or double angle, and of a singly symmetric built-up I:** whether the stem or web legs are in tension, in compression or reverse along the unbraced length under Mx (F9); which flange is in compression under Mx (F4, F5). They are your reading of the moment: with a reversal on a tee the lower of the two cases is used.
- **Flexure:** Cb by Eq. F1-1 from the moment diagram along the member (when Lb is the member length; for a shorter Lb, Cb = 1.0 unless entered), or entered. The H1.2 increase of Cb by √(1 + αPr/Pey) under axial tension can be switched off.
- **Tension end connection:** no holes and all elements connected (Ae = Ag), or the net area and the shear lag factor U (Table D3.1) entered. The net area is either entered or taken from up to 20 failure paths (B4.3b): holes on the path, Σ s²/4g of its diagonals, thickness, with one hole diameter. U is yours: the end connection decides it, and for open sections D3 lets it stop at the connected area over the gross area.

Auto-design (editor toolbar) varies the shape of the given family (W, S, M, HP, C, MC, tee, angle, double angle, rectangular HSS, round HSS or pipe), lightest first, optional depth limit; steel grade when unlocked. A double angle stays on its legs back to back and separation; a member from plates keeps its plates. Objectives: lightest (steel weight); smallest depth, then lightest. Kept unless you select them: steel. Every other input is kept; see Auto-design.

Shapes from other markets (extra choices). The Shape list can change from the AISC Shapes Database to the section database: region, family and designation, as the section builder picks them. In: rolled I-shapes (parallel or tapered flanges, a compact web), rectangular, square and circular hollow sections, hot-finished or cold-formed, channels of either flange form, tees, and single angles (with Member set to Single angle). A hollow wall is the nominal thickness of the table, without the 0.93 factor AISC applies to ASTM A500; a slender web of a rectangular section reads NOT FEASIBLE. Double shapes, plates and cold-formed sections are not on this list: no matching section of AISC 360 is built for them here. US shapes stay on the AISC list, with their data and results unchanged.

The design code stays AISC 360. Fy and Fu are your inputs, with no assumed grade and no default: a shape from another market is checked to AISC 360 with your material, not to its own national code, and the sheet says so. Whether that material suits AISC 360 (Section A3.1) is your judgment. Properties are the database's, calculated from nominal dimensions; Cw, ro and H are thin-walled and J by finite elements on the true outline, the basis AISC 360's equations assume. The designation shows as the market prints it, in US and metric. Without a shape database on the server the choice does not appear. **Auto-design** stays on the AISC shapes unless you chose another family; then it searches that family only, lightest first, with your Fy and Fu.

Checks performed

Check	Basis
Compression, W, S, M, HP, HSS, pipe	E3 flexural buckling about x and y (Eqs. E3-2 to E3-4); E4 torsional buckling of I-shapes (Eq. E4-2); E7 effective area for slender elements (E7.1 with Table E7.1 c1, c2; E7.2 for round HSS)
Compression, C, MC	E3 about y; E4-3 flexural-torsional about the axis of symmetry x; E7; Cw, ro, H from the shapes data
Compression, WT, MT, ST	E3 about x; E4-3 flexural-torsional about y; E7 (stem Table B4.1a Case 4, flange Case 1); Cw, ro, H from the shapes data

Check	Basis
Compression, single angle	E5 modified slenderness, Eqs. E5-1 to E5-4 with the short-leg term and floor; or Chapter H with flexural buckling about w and z; E4 by the cubic Eq. E4-4 when b/t exceeds 0.71 √(E/Fy); Fn by Eq. E3-2 or E3-3 on the lower elastic stress, with the area of E7
Compression, double angle	E6 modified slenderness about y (Eqs. E6-1, E6-2a, E6-2b, Ki = 0.50); E3 about x; E4-3; E7; J and Cw twice one angle's
Compression, two tees	E6 about x (Ki = 0.86); E3 about y; E4-2 with Cw = 0; E7
Compression, built-up I	E3; E4-2 (equal flanges) or E4-3 (different flanges); flanges Case 2 with kc, web Case 5; E7
Compression, plate	E3 only, as a solid rectangle (Table User Note E1.1)
Connectors of a built-up member	E6.2(a): a/ri at most 3/4 of the governing slenderness (a requirement); D4 User Note: a/ri up to 300 in tension (a recommendation); E6.1: the end connection
Tension	D2 yielding (Fy Ag, φ = 0.90) and rupture (Fu Ae, φ = 0.75); D3 with U as an input; B4.3b net area, entered or by paths
Pin-connected plate	D5.1: rupture (Eq. D5-1), shear rupture (Eq. D5-2), bearing (J7), yielding (D2); D5.2(c): plate width, extension
Eyebar	D6.1: D2 with the body width counted up to 8 t; D6.2 dimensions (thickness, transition radius, pin, hole, edge distance)
Major-axis flexure	W, S, M, HP: F2 / F3; C, MC: F2 (compact only); rectangular HSS: F7; round HSS and pipe: F8; tees and double angles: F9; built-up I: F2 / F3, F4 or F5 by web class and symmetry; plate: F11; single angle under Chapter H: F10 about w
Minor-axis flexure	W, S, M, HP, C, MC, built-up I: F6; rectangular HSS: F7; round: F8; tee: F12 (with an Iy above Ix and an unbraced length: not feasible); plate: F11; single angle under Chapter H: F10 about z
Shear, both axes	W, S, M, HP: G2.1 (major), G6 (minor); C, MC: G2.1(b), G6; rectangular HSS: G4; round HSS and pipe: G5 (Lv = L/2 with transverse load, else L); tee: G3, G6; double angle: G3 about x; built-up I: G2, G6; plate: G3
Combined force	H1.1 Eqs. H1-1a / H1-1b in compression, H1.2 in tension, per load combination; single angle under Chapter H: H2-1 at the heel and both toes, compression positive
Slenderness	Lc/r ≤ 200 (E2 User Note) and L/r ≤ 300 for tension (D1), as recommendations

The flexure and interaction checks report the combination that governs each; the diagrams show the first-order moments along the member, bottom to top.

Drawings

The drawing sheet (PDF / DXF, Drawing sheet button) shows the column elevation with the axial loads by type (or the governing required strength) on the top, the member length, the ends as braced points and a callout with the effective-length factors and unbraced lengths used in the design, the first-order moment diagrams about each axis for the governing combination drawn along the column from the base to the top, and the section of the family with its dimensions: S, M, HP as the W-shape, and outlines without the fillets for channels, tees, angles, double angles, two tees, built-up I and plates. Intermediate connectors are counted in the calculation, not drawn. End fixity, brace locations between the ends, the base plate and splices are not part of the calculation and are not drawn; the sheet notes say so.

Limitations

- A W-shape whose web is not compact for flexure (F4 / F5) is reported as not feasible; an S, M or HP-shape only when it carries Mx. A channel that is not compact (flange or web) carrying Mx is reported as not feasible: F2 covers compact channels only. The rolled channels at Fy up to 50 ksi do not reach it.
- These combinations read NOT FEASIBLE with a message, never a number: moment My on a double angle (F9 covers the plane of symmetry); any moment or shear on two tees flange to flange (Chapter F has no section for this cruciform); a moment on a single angle under E5 (change to Chapter H); shear on a single angle.
- Double channels, and members built up with lacing or perforated plates, are not covered. E6.2(b) to (e) (end welds, fastener spacing along the plates, cover plates, lacing) are detailing for you to check; only E6.2(a) is computed.
- Bracing offset from the shear center (E4(d) to (f)) is not covered. Under E5 the flexural-torsional check takes the lengths Kx Lx (about w), Ky Ly (about z) and Kz Lz as entered; the modified slenderness replaces the effective lengths for flexural buckling.
- Built-up sections ignore the fillet welds in their section values, as the Design Examples do, and a member from plates has no section choice in auto-design. D6.2(f) keeps the edge distance above 2/3 of the body width; a head is not otherwise checked.
- H1.3 (separate in-plane and out-of-plane checks) and H2 are not offered; H1.1 is used, which is always permitted. Torsion (H3) is not checked.
- The approximate second-order method takes Mr = B1 Mnt + B2 Mlt with the member's largest first-order moments (conservative). B2 is not computed (it is a story quantity, entered). In the direct entry mode all moments are treated as Mnt.
- The moment diagram between the ends is the end-moment line plus a parabola for the transverse load (Cb and the largest moment are taken from it); other load patterns can be entered as required strengths with Cb.
- Concentrated forces (J10), bearing, splices and connections are not checked.
- A500 Gr. B round HSS are taken with Fy = 46 ksi (ASTM A500-21); the Manual Table 2-4 text on disk was not legible for this row, so this value is still to be confirmed against the printed table.

Verification examples

Verification status: 217 values compared in 71 examples (71 published, 0 hand calculations from the cited equations): 216 within tolerance, 1 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Examples v16.0, LRFD and ASD (all within 1 %):

Example	What it checks	Published	ILX Calcs
E.1C	W14×132, Lc = 30 ft	φcPn = 892, Pn/Ωc = 598 kips	893.2, 594.3
E.1D	W14×90, Lcx = 30 ft, Lcy = 15 ft	927, 617 kips; Fex = 83.3 ksi	927.5, 617.1; 83.26
E.1E	W16×31 slender web, Lc = 5 / 10 / 15 ft	314 / 189 / 87.1 kips; Ae = 8.44 in.²	313.2 / 190.4 / 87.14; 8.435
E.9	HSS12×10×3/8, K = 0.8, 20 ft	556, 370 kips	555.6, 369.7
E.10	HSS12×8×3/16 slender walls, Lc = 18 / 24 / 40 ft	177 / 151 / 74.8 kips; Ae = 5.35 / 5.77 in.²	177.8 / 151.3 / 74.39; 5.355 / 5.773
E.11	Pipe 10 Std.	221, 147 kips	222.0, 147.7

Example	What it checks	Published	ILX Calcs
D.1	W8×21 in tension, An = 4.76 in.², U = 0.908	yielding 277 / 184, rupture 211 / 141 kips	277.2 / 184.4, 210.7 / 140.5
F.5	W12×58 minor-axis flexure	122, 81.4 kip-ft	121.9, 81.09
F.7B	HSS10×6×3/16 noncompact flange	59.7, 39.7 kip-ft	59.70, 39.72
F.8A	HSS8×8×3/16 slender flange (Manual Table 3-13)	46.3, 30.8 kip-ft	46.27, 30.79
F.9B	Pipe 8 x-Strong	81.4, 54.1 kip-ft	81.38, 54.14
G.1B, G.4, G.5, G.6	Shear: W24×62, HSS6×4×3/8, HSS16.000×0.375, W21×48 minor axis	306, 93.6, 232, 189 kips (LRFD)	305.7, 93.29, 232.2, 189.0
H.1B	W14×99, P + Mx + My, second-order moments given	1,130 kips, 642 / 311 kip-ft, ratio 0.929 (ASD 0.933)	1,127, 642.2 / 311.4, 0.929 (0.932)
H.3	W14×82 in tension and biaxial bending, Cb × √(1 + αPr/Pey)	1,080 kips, 492 / 168 kip-ft, ratio 0.873 (ASD 0.874)	1,080, 490.4 / 168.0, 0.875 (0.872)
H.4	W10×33, B1 by Appendix 8	B1 = 1.02 / 1.09, ratio 0.979 (ASD 0.982)	1.018 / 1.088, 0.981 (0.985)
D.4	HSS6×4×3/8 in tension, An = 5.79 in.², U = 0.919	yielding 278 / 185, rupture 248 / 165 kips	278.1 / 185.0, 247.4 / 165.0
D.5	HSS6.000×0.500 in tension, An = 7.57 in.², U = 0.991	yielding 364 / 242, rupture 349 / 233 kips	364.1 / 242.2, 348.8 / 232.6
E.1B	W14×90, Lc = 19 ft (Manual Table 4-1a)	903, 601 kips	903.4, 601.1
E.4B	W14×82 in a moment frame, Kx = 2.0, L = 14 ft	861, 573 kips	861.9, 573.5
F.6	HSS3½×3½×1/8 compact flanges	7.20, 4.79 kip-ft	7.20, 4.79

AISC Steel Construction Manual, 16th Ed., column tables (φcPn / Pn/Ωc in kips, pinned, Lc on both axes):

Table	Shape, Lc	Published	ILX Calcs
4-1a (p. 4-16)	W14×145, 20 ft / 40 ft	1,470 / 980; 663 / 441	1,473 / 980.0; 663.2 / 441.3
4-1a	W14×120, 10 ft / 30 ft	1,470 / 980; 807 / 537	1,473 / 980.3; 806.8 / 536.8
4-1a	W14×109, 16 ft; W14×99, 26 ft	1,190 / 789; 781 / 519	1,186 / 789.3; 780.8 / 519.5
4-4 (p. 4-73)	HSS8×8×1/4, 12 ft	274 / 182	274.2 / 182.5
4-4	HSS8×8×3/16 (slender), 10 ft / 20 ft	193 / 128; 156 / 104	192.4 / 128.0; 155.8 / 103.7
4-4	HSS8×8×1/8 (slender), 15 ft	87.2 / 58.0	86.99 / 57.88
4-4	HSS7×7×1/2, 16 ft / 30 ft	354 / 235; 140 / 93.1	353.5 / 235.2; 139.9 / 93.06
4-6 (p. 4-100)	Pipe 12 Std., 20 ft; Pipe 10 x-Strong, 14 ft; Pipe 8 x-Strong, 30 ft	370 / 246; 427 / 284; 169 / 113	370.3 / 246.4; 426.5 / 283.8; 169.4 / 112.7

E.1C reads Fn from Manual Table 4-14 and H.3 / H.4 take Cb = 1.14 from Manual Table 3-1 (ILX computes 1.136 by Eq. F1-1). For F.8 the Manual table value is compared: ILX computes Se with the compression flange reduced and the neutral axis shifted, as F7.2(c) states; Example F.8B removes the ineffective width from both flanges, which is conservative (45.4 kip-ft). Flexure and shear examples written for beams are run as a member with the same moment or shear entered and no axial force.

The other families, against AISC Design Examples v16.0, LRFD and ASD (all within 1 % unless said so):

Example	What it checks	Published	ILX Calcs
D.2	L4×4×1/2 in tension, one line of four bolts, U = 0.869	yielding 169 / 113, rupture 140 / 93.5 kips; An = 3.31 in.²	168.8 / 112.3, 140.3 / 93.5; 3.31
D.3	WT6×20 in tension, welded flange, U = 0.860	yielding 263 / 175, rupture 245 / 163 kips	262.8 / 174.9, 244.8 / 163.2
D.6	2L4×4×1/2 in tension, U = 0.944	yielding 338 / 225, rupture 305 / 204 kips; An = 6.63 in.²	337.5 / 224.6, 304.9 / 203.3; 6.63
D.7	Pin-connected plate: rupture, shear rupture, bearing, yielding	78.8 / 52.5, 80.3 / 53.5, 33.8 / 22.5, 96.3 / 64.1 kips	78.5 / 52.3, 80.4 / 53.6, 33.8 / 22.5, 95.6 / 63.6
D.8	Eyebar 3 × 5/8	84.6, 56.3 kips	84.4, 56.1
D.9	Plate with staggered bolts: worst of four paths	An = Ae = 5.75 in.²	5.77
E.2	Built-up I, slender web	500, 332 kips; Ae = 19.2 in.²; Fe,z = 91.9 ksi	498.0, 331.4; 19.15; 92.2
E.3	Built-up I, slender flanges, fully effective	318, 211 kips; Fe,z = 71.2 ksi	317.5, 211.2; 71.3
E.5	2L4×3 1/2×3/8 LLBB, two connectors	157, 104 kips; Fex 48.5, Fey 76.9, Fez 102, Fe 60.5 ksi	156.7, 104.3
E.6	2L5×3×1/4 LLBB, slender legs	73.8, 49.1 kips; Ae = 3.58 in.²; Fez 37.8, Fe 26.8 ksi	73.8, 49.1; 3.58
E.7	WT7×34	128, 85.0 kips; Fex 16.2, Fez 165, Fe 29.5 ksi	128.5, 85.5
E.8	WT7×15, slender stem	36.6, 24.4 kips; Fez 57.0, Fe 10.5 ksi	36.6, 24.4
E.12	Built-up I, different flanges	280, 186 kips; Fez 66.2, Fe 26.4 ksi	278.3, 185.2
E.13	Two WT9×20, one connector	297, 198 kips; (Lc/r)m = 46.6; Fe,z = 49.2 ksi; Ae = 11.2 in.²	297.7, 198.1
E.14A	L5×3×1/2 through the long leg (E5)	52.3, 34.8 kips; Lc/r = 127	52.8 (1.05 %), 35.2; 126.6
E.14B	L8×4×1/2 eccentric, Chapter H	168 / 112 kips; 375 / 250 and 123 / 82.0 kip-in.; interaction 1.00 at Pu = 85.9 / Pa = 56.4 kips	168.8 / 112.3; 375 / 249.5, 122.9 / 81.7; 0.999 / 0.999

Example E.14A rounds Lc/r up to 127 before finding the buckling stress; unrounded it is 126.6, so ILX reads 52.8 kips, 1.05 % above the printed value. This is recorded as a deviation. Example E.12 rounds ho to 11.3 in. and carries a rounded centroid; ILX keeps ho = 11.25 in. and is 0.6 % lower. Example D.9's plate is run with the pitch and gages of its figure.

Manual Table 4-11 (concentrically loaded single angles, Fy = 50 ksi, run on the Chapter H basis): L12×12×1 1/4, L12×12×1, L6×6×9/16 and L6×6×5/16, at 1 to 20 ft, all within 1 %. On a slender leg the flexural-torsional stress is taken with the effective area: this is what reproduces the Table, and it is the lower reading of E7's sentence on single angles. Read the other way, as the stress of E3 alone with the effective area and that of E4 on the gross area, the L6×6×5/16 would be up to 19 % higher at short lengths.

Checked by hand only, from the cited equations, because no published example is on disk: channels (E4-3, E7), S, M and HP (as rolled I-shapes), E5 through the short leg and E5(b), E6-1, E4-4 on unequal angles, net area by paths on a shape, beam-column cases of tees, double angles, channels and built-up I (through flexure checked in the steel beam's verification). F4, F5, F9, F10 and G3 are verified with the steel beam against Design Examples F.10, F.11, F.14 and G.3, G.8; E.14B runs F10 here.

Shapes from other markets: no published example applies AISC 360 to a non-US shape, so nothing here is checked against one. Hand calculations from the plain dimensions of HEB 300, SHS 200x200x10, CHS 273x10 and L 100x100x10; and the database path run on US shapes against the AISC path (W14X90, W21X50, W8X31, W30X90 within 1.5 %; HSS8X8X1/2, HSS12X6X1/4, HSS6.625X0.280, and C, WT and L within 3 %, where the database has a US row).

CHAPTER 24

Column base plate

An exposed column base: a W-shape, rectangular HSS, round HSS or pipe column on a steel base plate, bearing on a concrete pier or footing through a grout pad, with anchor rods. Concrete bearing to AISC 360-22 J8, plate bending by the AISC Manual Part 14 and AISC Design Guide 1, 3rd Ed. (2024) Chapter 4, anchor rods to AISC 360-22 J3 and the anchorage in the concrete to ACI 318-19 Chapter 17, with ASCE 7-22 load combinations.

Inputs of the base plate

- **Column and plate:** the column (AISC Shapes Database v16.0), the plate length N (along the column depth, the direction of the moment and the shear), width B, thickness tp and steel.
- **Concrete support:** the plan dimensions and depth of the pier or footing, concentric with the plate. They give A2 (the largest area geometrically similar to the plate, within a 1:2 frustum in the depth, J8) and the edge distances of the rods. f'c is the concrete strength (2,500 to 10,000 psi); the grout thickness enters the shear lug eccentricity and the ACI 0.80 grout-pad factor on the rod shear (17.7.1.2.1).
- **Loads:** service loads per load type (D, L, Lr, S, R, W, E: axial P with compression positive, moment M about the column major axis, shear V along N), combined by ASCE 7-22 2.3.1 or 2.4.1, or the required strengths entered directly for the chosen method. See Loads and combinations.
- **Anchor rods:** diameter, ASTM grade (F1554 Gr. 36, 55, 105, A449, A36), spacing between the two rod lines (along N) and of the outer rods along B, rods per line (1 to 4), headed (heavy hex nut, or a plate washer area Abrg) or hooked (hook length eh), and the embedment hef.
- **Shear transfer** (the engineer's choice): friction (μ, 0.4 prefilled per DG1 4.3.5), the anchor rods (the number of rods that take shear: all of them with welded plate washers or a setting plate; rod bending over half the washer-to-grout distance, DG1 4.3.4) or a shear lug (width, height below the plate, thickness, steel, fillet weld).
- **Code options:** whether the ACI 318 anchorage is checked, cracked concrete, supplementary reinforcement for tension and for shear (φ per Table 17.5.3(b)) and the edge reinforcement factor ψc,V.

Auto-design (editor toolbar) varies the plate length N and width B (1-in. steps), thickness tp (1/8-in. steps) and rod diameter; the rod spacing follows the plate size; plate grade when unlocked. Objectives: lightest (plate + rods steel weight); lowest cost index (steel). Kept unless you select them: plate steel. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Concrete bearing	J8: $P_p = 0.85 f'_c A_1 \sqrt{(A_2/A_1)} \le 1.7 f'_c A_1$, $\varphi_c = 0.65$, $\Omega_c = 2.31$
Bearing under axial load and moment	DG1 uniform bearing stress: e ≤ ecrit (Y = N − 2e, Eqs. 4-39 to 4-44) or e > ecrit with rod tension (Eqs. 4-55 to 4-59; no real solution = plate too small)
Plate bending, bearing side	Axial: l = max(m, n, λn'), Manual Eqs. 14-2 to 14-6; moment: m with Y (Eqs. 4-51, 4-52) and n; $M_n = F_y t_p^2/4$ (F11) and $t_{p,req}$

Check	Basis
Plate bending, tension side	Rods beyond the flanges: x = f − d/2 + tf/2, M = T x / B (Eqs. 4-61, 4-62); rods between W flanges: the web cantilever a over beff = min(2a, d − 2tf) (Example 4.7-3)
Rod tension (AISC)	DG1 Eq. 4-22: Rn = Fu Ase, φ = 0.75, Ω = 2.00
Rod shear and combined stresses (AISC)	J3.7: Fnv = 0.450 Fu; F'nt by Eqs. J3-3a/b with the rod bending stress (DG1 4.3.4)
Friction	DG1 Eq. 4-30: min(0.65 μ Pu, 0.75 × 0.2 f'c Ac, 0.75 × 800 psi Ac), LRFD combinations
Shear lug	ACI 318-19 17.11.2 bearing and 17.11.3 breakout (φ = 0.65); lug flexure (F11) and shear yielding (J4-3); fillet welds (J2.4, kds = 1.5); the lug moment V (G + hef,sl/2) enters the bearing analysis (17.11.1.1.2)
Anchorage in the concrete	ACI 318-19 Chapter 17 for the rod group (see Anchors in concrete): steel, breakout, pullout, side-face blowout, shear breakout, pryout, interaction

ACI 318 is strength design: the anchorage, friction and lug-concrete checks always use the LRFD combinations. With ASD and required strengths entered directly, those checks cannot be made and an advisory says so.

Drawings

The drawing sheet (screen, PDF and DXF) has two views: the **plan** (concrete support, base plate, column section, anchor rods, the shear lug hidden below the plate, the rod spacing and edge distances, and the plate and support sizes) and the **section along N** (column stub, plate, grout pad, the support hatched, the rods with their headed or hooked end at hef, the shear lug with its designed fillet welds, rod projection, embedment and thickness). Only what the calculation designs is drawn; the column-to-plate welds, hole and washer sizes, rod length, leveling hardware and the support reinforcement are not part of it, and the sheet notes say so. The nuts and plate washer are the standard hardware of the rod diameter. On 11 x 17 both views share one sheet; on letter sheets a larger plate takes a continuation sheet.

Limitations

- Moment about the column major axis only; biaxial moments and a minor-axis moment are not covered. The bearing is the DG1 uniform (rectangular) stress block; the triangular (elastic) distribution of DG1 Appendix B is not offered.
- Rods in two lines across N, symmetric about the plate centre. Rods inside the walls of an HSS or pipe are reported as not feasible. For HSS and pipe columns the tension-side lever uses the wall centre line (DG1 states Eq. 4-61 for W columns).
- Not checked: column-to-plate welds, plate washers and holes (Manual Table 14-1), the base plate local bending at the lug, anchor reinforcement (ACI 17.5.2.1), seismic anchorage (17.10; use the Anchors in concrete calculation for it).
- Combined perpendicular and parallel shear on one edge is not a code rule: shear acts along N only here.

Verification examples

Verification status: 133 values compared in 25 examples (25 published, 0 hand calculations from the cited equations): 129 within tolerance, 4 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Examples v16.0 (J.6, K.9) and AISC Design Guide 1, 3rd Ed. (2024):

Example	What it checks	Published	ILX Calcs
J.6	W12×96, 22 × 22 plate on a 24 × 24 pedestal	φcPp = 875, Pp/Ωc = 583 kips, tmin = 1.54 in.	875.2, 582.9, 1.541
K.9	HSS6×6×½ on a footing	Pp = 862, φcPp = 560, Pp/Ωc = 373 kips, tmin = 0.917 / 0.918 in.	861.9, 560.2, 373.1, 0.917 / 0.918
DG1 4.7-1	Axial, A2 = A1	φcPp = 729, X = 0.960, λn' = 3.11, tmin = 1.36 in.	729.3, 0.959, 3.112, 1.361
DG1 4.7-2	Axial with confinement	φcPp = 723, tmin = 1.21 in.	716.0, 1.211
DG1 4.7-3	Uplift, rods between the flanges	ru = 17.5, φRn = 20.1, tp,req = 0.882 in., φNcbg = 76.3 kips	17.5, 20.1, 0.882, 76.6
DG1 4.7-5	Shear lug	Y = 0.0967 in., T = 6.41, φVbrg = 70.9, φVcb = 26.1, ru = 7.91 kip/in.	0.097, 6.41, 70.7, 26.1, 7.91
DG1 4.7-6	Rods in tension, shear and bending	ft = 42.2, φF'nt = 26.5 ksi (1⅛ in.); 20.3, 32.6 ksi (1½ in.)	42.2, 26.5; 20.4, 32.6
DG1 4.7-8	Friction; A2 ≥ 4A1	φVn = 129, φcPp = 1,440 kips, tmin = 0.896 in.	129.0, 1,436.5, 0.897
DG1 4.7-9	Pure moment	Y = 0.886 in., T = 47.0 kips, tp,req = 1.06 / 1.31 in.	0.885, 47.0, 1.062 / 1.307
DG1 4.7-10	Small moment	ecrit = 5.02 in., Y = 14.0 in., tp,req = 1.16 in.	5.023, 14.0, 1.158
DG1 4.7-11	Large moment	Y = 8.60 in., tp,req = 1.92 / 0.755 in.	8.585, 1.918 / 0.749
DG1 4.7-12	Uplift with a large moment	qmax = 42.0 / 27.9 kips/in.; Y = 0.300 in., T = 113 / 68.4 kips	41.99 / 27.97; 0.274 / 0.247 in., 111.5 / 66.9 kips (deviation)
DG1 4.7-13	Uplift with a low moment, all rods in tension	largest rod tension 28.1 / 16.9 kips	28.12 / 16.88

Every value is within 1 % (LRFD and ASD, in `tests/test_verify_steel_base_plate.py`) except the DG1 4.7-12 bearing length and rod tension: the Example writes Y = 17.5 in. − 17.2 in. = 0.300 in., the difference of two rounded numbers; unrounded, Eq. 4-65 gives 0.274 in. (LRFD) and 0.247 in. (ASD), and the rod tension is 1.3 % and 2.2 % lower (documented deviation DG1-4712-ROUND). The 4.7-2 bearing strength is 1.0 % lower because ILX takes A2 geometrically similar to the plate as J8 defines it (518 in.²; the example uses 528 in.²). Where an example leaves out the plate size or the rod pattern, the value used is stated with the example in the test file.

CHAPTER 25

Composite beam

A simply supported rolled W-shape acting compositely with a concrete slab (solid, or on formed steel deck) through steel headed stud anchors, to AISC 360-22 Chapter I, with ASCE 7-22 load combinations. The calculation covers full and partial composite action, the anchors, the unshored construction stage, shear on the steel section and the deflections before and after composite action.

Inputs of the composite beam

- **Beam:** the W-shape, the steel, the span (simply supported) and whether the self-weight is added to the dead load applied before composite action.
- **Effective width:** the distance to the adjacent beam on each side, or the distance to the slab edge on side 2. Each side contributes the least of span/8, half the distance to the adjacent beam and the edge distance (I3.1a).
- **Slab:** solid slab, or formed deck with the ribs perpendicular or parallel to the beam; the concrete thickness above the deck tc (the total thickness for a solid slab), the rib height hr, the average rib width wr (not more than the clear width near the top of the deck), f′c, normal weight or lightweight concrete and the unit weight wc (for Ec = wc^1.5 √f′c). For perpendicular ribs the concrete below the top of the deck is neglected (I3.2c.2). For parallel ribs it is included (I3.2c.3) as the fraction wr / rib spacing of the rib zone; enter the rib spacing.
- **Anchors:** diameter, Fu (65 ksi for AWS D1.1 Type B anchors), installed length, position in the rib (weak: emid-ht < 2 in., Rp = 0.6; strong: Rp = 0.75; the weak position is the conservative choice when it is not controlled), the anchors between the point of maximum moment and the support as groups (number of anchors, anchors per rib or per transverse row, which sets Rg), the spacing along the beam, whether the anchors are welded over the web, and, when there are concentrated loads, whether the anchors are uniformly spaced (I8.2d(a)) or all lie between the support and the load.
- **Construction:** unshored or shored; the bracing of the compression flange during construction (continuous when the deck is attached, or an unbraced length) and the camber.
- **Loads:** line and point loads, each with a load type and a stage: *before composite* (slab, deck, beam; on the steel section and permanent), *construction only* (the construction live load of ASCE/SEI 37, for example 25 psf over the tributary width; on the steel section only) and *after composite* (superimposed dead and live loads). See Loads and combinations.
- **Deflection limits:** span / n before composite (net of camber; AISC Design Guide 3 suggests 360 and 1 in.), for live load and for total load (IBC 2021 Table 1604.3 floor values 360 and 240 prefilled); 0 turns a check off.

Auto-design (editor toolbar) varies the W-shape (lightest first, optional depth limit) and the number of steel headed stud anchors. Objectives: lightest steel, then fewest anchors; lowest cost index (steel + anchors). Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Construction stage flexure	I3.1b: the steel section alone under the loads before composite and the construction loads, F2 / F3 with the construction unbraced length and Cb (F1-1)
Composite flexure	I3.2a(a): plastic stress distribution; slab force C = min(0.85 f'c Ac, Fy As, ΣQn) (Eqs. I3-1a to I3-1c); PNA in the slab, top flange or web; Mn about the top of the steel (Commentary Eq. C-I3-10); φb = 0.90, Ωb = 1.67
Flexure at concentrated loads	I8.2c: the anchors between each concentrated load and the support develop the moment there
Anchor strength	I8.2a Eq. I8-1, Rg and Rp from the deck orientation, anchors per rib and position
Shear	I4.3 and G2.1 on the steel section
Pre-composite deflection	Loads before composite on Is, net of the camber (limit entered)
Live load deflection	Each of L, Lr, S, R applied after composite, with ILB (Commentary Eqs. C-I3-1, C-I3-2)
Total load deflection	ASD gravity combinations: loads before composite on Is (shored: on ILB), loads after composite on ILB, less the camber
Deck and anchor detailing	I3.2c.1 (hr ≤ 3 in., wr ≥ 2 in., tc ≥ 2 in., anchor 1-1/2 in. above the deck with 1/2 in. cover), I3.2c.3 haunch width, I8.1 (dsa ≤ 3/4 in. on deck, ≤ 2.5 tf off the web), I8.2 (length ≥ 4 dsa), I8.2d(d), (e) spacing
Shear connection ductility	Commentary I3.2d.1 prescriptive conditions (span ≤ 30 ft, ≥ 50 % composite, or ≥ 16 kip/ft); advisory

The key results list the effective width, ΣQn, the degree of composite action, the PNA location and ILB. The diagrams show the loading, the section and the shear and moment of the combination that governs composite flexure.

Drawings

The drawing sheet (PDF / DXF, Drawing sheet button) shows the beam elevation with the slab on the beam, the simple supports, the loads by type and stage (PRE before composite action, CONST. during construction only, POST after) and the span, the member mark in the usual form W21×50 (46) C=2" (total number of anchors, camber); the typical section with the slab or deck as designed, the effective width, the W-shape and the headed stud anchors per rib; and the shear and moment diagrams of the governing combination. The anchor start location, the transverse spacing of anchors in a rib (drawn at four stud diameters), deck anchorage, slab reinforcement and the end connections are not part of the calculation; the sheet notes say so.

Limitations

- Simply supported spans only: no negative moment (I3.2b), cantilevers or continuous composite beams.
- W-shapes with h/tw ≤ 3.76√(E/Fy) (I3.2a(a)); otherwise not feasible (I3.2a(b) elastic superposition is not covered). f'c must be within I1.3(a) and wc within 90 to 155 pcf, else not feasible.
- For parallel deck with wr/hr < 1.5, Rg is defined for a single anchor only; two or more anchors in a row are reported as not feasible.

- The ILB uses the slab force V′ (ΣQn, or the full-composite force when that is smaller) at the point of maximum moment. Long-term creep and shrinkage deflections, ponding of the wet concrete and floor vibration (Design Guide
  1. are not computed.
- The analytical shear-connection ductility procedure of Design Example I.1 is not computed; the prescriptive conditions are reported.
- Lateral cover, edge distance in the direction of shear (I8.2d(b), (c)), transverse spacing of anchors in a rib and the deck anchorage spacing (I3.2c.1(d)) are noted for the engineer to detail; channel anchors are not covered.
- The construction stage uses the loads entered for that stage; ASCE/SEI 37 construction loads are the engineer's input. The pre-composite deflection limit follows AISC Design Guide 3 as quoted in Design Example I.1 (the Guide is not on file).

Verification examples

Verification status: 76 values compared in 7 examples (7 published, 0 hand calculations from the cited equations): 76 within tolerance, 0 recorded deviations. Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

AISC Design Examples v16.0, in LRFD and ASD:

Example	What it checks	Published	ILX Calcs
I.1	W21×50, 45 ft, deck perpendicular, 23 anchors per half	Qn = 17.2 / 14.6 kips, φMn = 769, Mn/Ω = 512 kip-ft	17.23 / 14.65, 772.5, 513.9
I.1	Construction and composite demands, shear	Mu = 344 / 678 kip-ft, φVn = 237 kips	344.3 / 678.4, 237.1
I.1	Deflections	Δ pre = 2.59 in., ILB = 2,520 in.⁴, Δ live = 1.26 in.	2.59, 2,533, 1.256
I.2	W24×76 girder, 30 ft, deck parallel, 26 anchors	Qn = 21.5 kips, a = 1.83 in., x = 0.623 in., φMn = 1,280, Mn/Ω = 850 kip-ft	21.54, 1.83, 0.623, 1,267, 843
I.2	Construction stage, Lb = 10 ft	φMn = 677, Mn/Ω = 450 kip-ft	677.3, 450.6
I.2	ILB with 27 anchors	ΣQn = 581 kips, YENA = 18.3 in., ILB = 4,730 in.⁴, Δ live = 0.543 in.	581.5, 18.27, 4,720, 0.544

AISC Steel Construction Manual, 16th Ed., Table 3-18 (φbMn / Mn/Ωb, pp. 3-178 and 3-179) and Table 3-19 (ILB, p. 3-203). Each entry is reproduced with a solid slab whose thickness gives the tabulated Y2 for the tabulated ΣQn (b = 120 in., f'c = 4 ksi); for the partial-composite rows the anchor count and Fu are chosen so that n Qn equals the tabulated ΣQn:

Shape, PNA	ΣQn, Y2	Published	ILX Calcs
W21×50, TFL (full)	735 kips, 5.0 in.	849 / 565 kip-ft, ILB = 2,730 in.⁴	848.9 / 564.8, 2,727
W21×50, location 7	184 kips, 5.0 in.	591 / 393 kip-ft, ILB = 1,680 in.⁴	588.2 / 391.3, 1,682
W21×44, BFL	358 kips, 6.0 in.	661 / 440 kip-ft, ILB = 2,080 in.⁴	660.7 / 439.6, 2,077
W18×60, TFL (full)	880 kips, 4.0 in.	865 / 575 kip-ft, ILB = 2,490 in.⁴	864.6 / 575.2, 2,494

Example I.1 reads φMn and ILB from Manual Tables 3-18 and 3-19 at ΣQn = 386 kips and Y2 = 7.00 in.; ILX computes them directly with the provided ΣQn = 391 kips, so its values are 0.4 % and 0.5 % higher. In Example I.2 the Example takes d/2 = 12.0 in. and rounds Mn = 16,955 kip-in. to 17,000 kip-in.; ILX uses d/2 = 11.95 in., 0.8 % lower. All values are within 1 %.

CHAPTER 26

Steel section selection

Finds the lightest rolled W-shape that passes every check of a steel beam or a steel column calculation, optionally within a maximum depth and a list of nominal depths, and lists the lightest few passing shapes with their governing ratio.

Inputs of the section selection

- **Selection:** beam or column; the steel (A992, A572 Gr. 50, A913 Gr. 65, A36); the maximum actual depth d (0 = no limit); the acceptable nominal depths in the US designation (W14 = 14, also for the metric W360), blank for any; how many passing shapes to list.
- **Beam inputs:** the inputs of the steel beam (length, supports, line and point loads, self-weight, bracing, Cb, deflection limits); see Inputs of the steel beam.
- **Column inputs:** the inputs of the steel column (lengths and K factors, loads, second-order effects, Cb, tension connection); see Inputs of the steel column.

Auto-design: the selection is itself the auto-design search of the steel beam or steel column (the same W-shape candidates, prefilter and weight objective; see Auto-design), so this calculation has no separate Auto-design button. To let auto-design vary the steel grade or use the depth objective, open the selected shape as a steel beam or steel column calculation.

Checks performed

Check	Basis
A W-shape passes	Each candidate is designed by the steel beam or steel column calculation itself, every check included
Checks of the selected shape	Exactly the checks of a steel beam or steel column calculation of that shape
Search	The auto-design search of the steel beam or steel column: candidates prefiltered by necessary properties from a reference design (beam: Zx from the flexural demand and Ix from the deflection ratio; column: Ag, Zx, Zy), with a 15 % margin for the self-weight change; designed lightest first (equal weights: shallower first)

The key results list the selected shape and the lightest passing shapes (weight, depth, governing ratio and check), weight and depth in the units of the calculation (plf and in., or kg/m and mm). The diagrams are those of the selected shape.

Drawings

The drawing sheet of the selection is the sheet of the selected shape exactly as the steel beam or steel column calculation draws it (elevation, section and diagrams), with a note that the shape is the lightest W-shape that passes every check within the depth limits, and the next lightest passing shapes listed as alternates in the design summary. Nothing is drawn when no shape passes.

Limitations

- W-shapes only (the shapes the beam calculation designs). HSS and pipe columns are designed one at a time in the steel column calculation.
- At most 1,500 designs per selection (the auto-design evaluation cap, more than there are W-shapes, so every candidate within the limits can be designed). There is no time limit: the same input always selects the same shape. When no candidate passes the result is not feasible and says how many shapes were designed.
- Weight is the only criterion; depth, availability and connection needs are the engineer's.

Verification examples

Verification status: 24 values compared in 6 examples (6 published, 0 hand calculations from the cited equations): 24 within tolerance, 0 recorded deviations. Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

AISC Design Examples v16.0 selection examples, LRFD and ASD:

Example	Selection	Published	ILX Calcs
F.1-1A	Beam, 35 ft, nominal depth ≤ 18 in., live load L/360	W18×50, φMp = 379 kip-ft	W18×50, 378.8
F.4	Beam, 30 ft, live load deflection 1.00 in. (L/360)	W24×55, φMp = 503 kip-ft	W24×55, 503.1
E.1A	Column, W14, pinned, 30 ft	W14×132, φcPn = 893 kips	W14×132, 893.2

Selections read from the AISC Steel Construction Manual, 16th Ed. (the lightest tabulated shape that covers the required strength, LRFD and ASD):

Table	Selection	Published	ILX Calcs
3-2 (p. 3-25)	Beam, 30 ft, continuously braced, Mu = 405 / Ma = 270 kip-ft	W21×50, φbMpx = 413, Mpx/Ωb = 274 kip-ft	W21×50, 412.5, 274.5
4-1a (p. 4-16)	W14 column, Lc = 20 ft, Pu = 1,400 / Pa = 938 kips	W14×145, 1,470 / 980 kips	W14×145, 1,473 / 980.0
4-1a	W14 column, Lc = 16 ft, Pu = 1,000 / Pa = 675 kips	W14×99, 1,080 / 716 kips	W14×99, 1,077 / 716.3

A behaviour test designs every lighter shape within the limits and confirms that each fails, so the selection is the lightest.

CHAPTER 27

Eccentric bolt group

A group of bolts under an in-plane load whose line of action misses the centroid, by the instantaneous center of rotation (IC) method or the elastic method of the AISC Manual, 16th Edition, Part 7, with the per-bolt strengths of AISC 360-22 Section J3. The result is the coefficient C, the available strength C rn against your load, the force in every bolt and the critical bolt. Use it for a bracket, a shear tab with a large eccentricity or any pattern the Manual does not tabulate.

Inputs of the eccentric bolt group

- **Load:** service loads by type (each ASCE 7-22 2.3.1 or 2.4.1 combination of the types present is evaluated, all along the same load line; W and E act both ways) or the required strength directly. See Loads and combinations.
- **Load line:** the angle θ from the vertical (0 is straight down; a positive angle turns the load clockwise, so the arrow points down and to the left, as in the Manual tables) and the point the load passes through, ex to the right of and ey above the centroid. The Manual's ex is measured at the level of the centroid: ex − ey tan θ, as in Design Example II.A-24 Solution B. The calculation shows it.
- **Bolt pattern:** bolts per vertical row n, number of vertical rows, pitch s and gage g; or custom coordinates (any origin, the centroid is computed), up to 200 bolts.
- **Bolts:** diameter, grade (Group 120, 150, 200 or A307), threads in or out of the shear plane, single or double shear, hole type, joint type (snug-tightened, pretensioned or slip-critical). A slip-critical joint also takes the faying surface class, the pretension multiplier Du (AISC 360-22 J3.9 gives 1.13, another value only as approved by you as the engineer of record) and whether two or more fillers are present (hf = 0.85). An A307 bolt takes the grip.
- **Connected ply:** thickness of the thinnest connected part, material and edge distance le along the load, used for bearing, tearout and the spacing and edge limits. A thickness of 0 checks bolt shear alone and says so in the results, as Design Examples II.A-24 and II.A-25 assume. Whether hole deformation at service load is a design consideration is your judgment (J3.11a: 2.4 d t Fu and 1.2 lc t Fu when it is, prefilled, or 3.0 and 1.5 when it is not). Unpainted weathering steel sets the smaller maximum spacing of J3.6.
- **Code options:** the strength method, and whether the connection is end-loaded. Table J3.2 note [c] reduces the shear stress to 83.3 % when an end-loaded fastener pattern is longer than 38 in. Whether your bolts are end-loaded is your judgment; on is the prefilled, conservative choice, and it changes nothing below 38 in.

Auto-design varies the bolt diameter, the bolts per vertical row (1 to 16) and the number of vertical rows (1 to 4), fewest bolts first; the pitch, gage, load line, ply and joint stay as you entered them and the grade is locked unless you unlock it. A custom pattern keeps its coordinates and varies the diameter and grade only.

Checks performed

Check	Basis
Bolt group, chosen strength method	Required strength against φ C rn or C rn / Ω, φ = 0.75, Ω = 2.00 (Manual Eq. 7-15; AISC 360-22 J3.7, J3.11)

Check	Basis
Bolt group, the other method	The same comparison, shown and not counted in the status
rn per bolt	The least of bolt shear (Table J3.2, with notes [c] and [d]), bearing and tearout (J3.11a, Eqs. J3-6a to J3-6f, on the ply entered)
Slip resistance (slip-critical joints)	C μ Du hf Tb ns with φ and Ω by hole type (J3.9, Eq. J3-4, Table J3.1); the bolt shear, bearing and tearout check remains
Manual table value (note)	The tabulated C for the pattern, linear in ex and at the next lower tabulated angle, beside the C of the actual load line; the IC value is the one used
Minimum spacing, preferred spacing	2-2/3 d with a clear distance d between holes; 3d preferred (J3.4)
Maximum spacing and edge distance	24 t and 12 in., or 14 t and 7 in. for weathering steel; 12 t and 6 in. (J3.6)
Minimum edge distance	Tables J3.4 and J3.5 (J3.5)
Hole use	Oversized holes and slots along the load need a slip-critical joint (J3.3)
Pretension	A pretensioned or slip-critical joint needs a Table J3.1 value (J3.2); Group 200 bolts come in 1 to 1-1/4 in.

IC method. Each bolt's force follows the load-deformation curve R = Rult (1 − e^(−10Δ))^0.55, with the deformation proportional to the distance from the instantaneous center and 0.34 in. at the farthest bolt (Manual Eqs. 7-9 to 7-14). The calculation finds the center by equilibrium and reports C = P / Rult. The Manual's iterative tables use a 1 % convergence, so the printed values can differ from the exact solution by about that much.

Elastic method. Direct shear P / n plus the moment shear r = M l / Ip at each bolt, added as vectors (Manual Eqs. 7-2 to 7-8). C is P over the largest resultant. It ignores the ductility of the bolts and is more conservative.

Tearout. The clear distance lc is the smaller of le − h/2 and s − h, and that value is used for every bolt: the simplification of Design Example II.A-22, which is conservative. The hole dimension is the one along the nearer of the vertical and horizontal axes to the load, and le is entered, not derived from an outline, because the connected parts are not modelled.

Manual table note. Between tabulated angles the Manual warns that straight-line interpolation may be wrong and says to use a direct analysis or the next lower angle: the note reads the next lower angle, and the check uses the direct analysis at your angle. The note appears for s = 3 or 6 in., up to 12 bolts per row and up to four rows. For a gage the Manual does not tabulate it says so, and gives the IC value at the table's grid points. With Group 150 or Group 200 bolts a note repeats the Manual's statement that the tables, built on the Group 120 curve, are conservative.

The drawings show the bolt pattern with the pitch, gage, edge and load point dimensioned, and the bolt forces of the strength method in proportion, with the instantaneous center and the critical bolt marked.

Limitations

- In-plane loads on one group. Out-of-plane loads (bolts in shear and tension, Manual Part 7 Case II) and a pure moment with no force are not covered: enter a load at the eccentricity you want. The printed C' of the tables is verified but not offered as a result.
- The connected parts are not designed: plate yielding, rupture, block shear and buckling are separate checks.

- One thinnest ply stands for the group's bearing and tearout. Different plies per bolt, and a separate interior bolt strength (as Examples II.A-30 and II.A-17 use), are not modelled; the result is on the conservative side.
- An inclined load uses the hole dimension and the slot direction of the nearer axis (vertical or horizontal).
- ASTM F3148 (Group 144) is not in the grade list: its Table J3.2 strengths are not modelled.
- Manual Table 7-14 is a table of fastener dimensions and is not used; nor are the clearance tables.

Verification examples

Verification status: 98 values compared in 37 examples (37 published, 0 hand calculations from the cited equations): 97 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Examples v16.0, Vol. 1, against the published values (more values, LRFD and ASD, are on the Verification page):

Example	What it checks	Published	ILX Calcs
II.A-24A (IIA-256)	12 bolts, 7/8 in. Group 120-N, ex = 16 in., vertical; Table 7-8	C = 3.55; 86.3 kips LRFD, 57.5 kips ASD	3.554; 86.5, 57.7
II.A-24B (IIA-257)	The same at 15 degrees, load point 7.5 in. above the centroid	ex = 14.0 in.; C = 4.05; 98.4, 65.6 kips	13.99; 4.048; 98.6, 65.7
II.A-25 (IIA-258, 259)	Elastic method on the same group	Ip = 406; r / P = 0.352; 69.0, 46.0 kips	405.8; 0.3525; 69.1, 46.1
II.A-22 (IIA-244, 245)	Bracket plate, 3/4 in. bolts, 3/8 in. plate, le = 2 in.	Shear 17.9, bearing and tearout 32.9 kips per bolt; C = 4.53; C required 2.01	17.89; 32.91; 4.529; 2.012
II.A-20 (IIA-231, 232)	Shear splice, tearout governs	lc = 1.03 in., tearout 22.6 and 15.1 kips per bolt; Table 7-6 C = 3.07; C required 2.65	1.031; 22.62, 15.08; 3.067; 2.652
II.A-30 (IIA-295, 296)	Tee stem tearout at the edge bolt	17.3, 11.5 kips; Table 7-6 C = 2.45	17.28, 11.52; 2.454
II.A-17A, 17B, 19A, 19B	C and C' from Tables 7-6 and 7-7	3.54; 4.07; 2.33, 26.0; 3.53, 38.7	3.536; 4.061; 2.329, 26.03; 3.534, 38.67
J.4A, J.4B, II.D-3	Slip per bolt: short slots across the load, long slots, oversized holes	19.0, 13.3, 16.1 kips LRFD; 12.7, 8.88, 10.8 kips ASD	18.98, 13.29, 16.14; 12.66, 8.87, 10.79

AISC Manual, 16th Edition, tables, read from the page images:

Table	Compared	Result
7-1 (p. 7-21), 7-3 (p. 7-24)	Bolt shear and slip per bolt, a sample of cells, LRFD and ASD	Within 1 %
7-6, 7-7, 7-8	Every printed C (7,692), through the engine	Within 1 %, except the row below
7-9, 7-10, 7-11, 7-12, 7-13	12,142 of the 12,960 printed C through the engine, and 16 cells through the calculation; 108 of the 120 printed C' against Manual Eq. 7-17	Within 1 %, except the row below; C' within 0.5 %

The other 818 cells of Tables 7-9 to 7-13 were not compared: two machine readings of the page disagreed and they were not read by hand. The 12,142 cells were read by machine, and those that missed the method or could not be read (209) were read by eye; the 16 sample cells and Tables 7-1 and 7-3 by eye. Table 7-14 is not a coefficient table (it gives fastener dimensions) and is not part of this calculation.

The one difference: at 75 degrees, s = 6 in. and ex = 2 in., the larger groups (n = 9 to 12, and the row for Tables 7-6 to 7-8 likewise) are printed up to about 2.3 % above the IC method, for example Table 7-10, n = 9: 26.4 against 25.85. The page images confirm the printed values, and several are printed the same for two different gages. The calculation uses the method, which is on the safe side; this is a listed deviation.

Slip through a group, and bearing or tearout through a group, have no published example with an eccentric load: the per-bolt strengths are verified as above and the group strength is C rn, as Manual Part 7 states.

CHAPTER 28

Eccentric weld group

A group of fillet welds under an in-plane load whose line of action misses the centroid, by the instantaneous center of rotation (IC) method or the elastic method of the AISC Manual, 16th Edition, Part 8, with the weld strength of AISC 360-22 Section J2.4. The result is the coefficient C, the required size, the available strength against your load and the force along the welds. Use it for a bracket, a seat, a welded tab or any pattern the Manual does not tabulate.

Inputs of the eccentric weld group

- **Load:** service loads by type (each ASCE 7-22 2.3.1 or 2.4.1 combination of the types present is evaluated, all along the same load line) or the required strength directly. See Loads and combinations.
- **Load line:** the angle θ from the vertical (0 is straight down; a positive angle turns the load clockwise, as in the Manual tables), the distance of the line from the vertical weld at the level of the centroid, and, for an inclined load that does not pass through the centroid level at that point, the height of the load point above the centroid. The Manual's a l is the distance less the centroid distance $\bar{x}$ l; the calculation shows both.
- **Weld group:** a single line, two vertical lines (Table 8-4), two horizontal lines (Table 8-5), a box with l vertical or horizontal (Tables 8-6, 8-7), a C with returns toward or away from the load (Tables 8-8, 8-9), an L with its return toward or away from the load (Tables 8-10, 8-10a, 8-11, 8-11a), or up to 60 custom segments. Two vertical lines with kl = 0 are one on each face of a single plate, the k = 0 column of Table 8-4. For two horizontal lines, l here is the spacing and kl the line length, the reverse of Table 8-5; the table note converts.
- **Weld:** the leg size and the electrode. The IC method uses C1 of Manual Table 8-3, the elastic method FEXX. That the filler suits the base metal (AISC 360-22 Table J2.6) is yours to confirm.
- **Connected parts:** the thinner part, which sets the size limits and the base metal check; its material; whether the welds share a shear plane (Manual Part 9: 3.09 D / Fu or 6.19 D / Fu); whether the weld runs along the edge of the part, which is what brings in the maximum size of J2.2b(b).
- **Code options:** the strength method, and the directional strength increase kds. The tables assume the increase. Turning it off is your judgment, for example for a single-sided fillet whose root can open (AISC 360-22 J2.4(a)).

Auto-design varies the fillet size (1/8 to 3/4 in.) and the length l (4 to 48 in.), least weld metal first; the return, the load line, the electrode and the parts stay as you entered them. A custom group keeps its segments and varies the size only.

Checks performed

Check	Basis
Weld group, chosen strength method	Required strength against φ C C1 D l or C C1 D l / Ω, φ = 0.75, Ω = 2.00 (Manual Eq. 8-30; AISC 360-22 J2.4, Table J2.5), or the elastic method (Manual Eqs. 8-11 to 8-17)
Weld group, the other method	The same comparison, shown and not counted in the status

Check	Basis
Manual table value (note)	The tabulated C for the shape, linear in k and a, at the next lower tabulated angle, beside the C of the actual load line; the IC value is the one used
Minimum fillet size	Table J2.4, on the thinner part
Maximum fillet size	t, or t − 1/16 in. from 1/4 in. up, along an edge (J2.2b(b))
Minimum length	4 w on every segment (J2.2b(c))
Long welds (advisory)	Where a line longer than 100 w is end-loaded, the factor of Eq. J2-1. It is reported and not applied, as in the Manual tables; whether the weld is end-loaded is your judgment
Base metal	3.09 D / Fu or 6.19 D / Fu on the part, with D the smaller of the provided and the required size (Manual Part 9)

IC method. Each element carries 0.60 FEXX (1 + 0.50 sin^1.5 θ) f(p) times its throat area. Its deformation is in proportion to its distance from the instantaneous center, and the element with the least ultimate deformation over distance reaches its limit (Manual Part 8). The calculation finds the center by equilibrium and reports C on l. It is solved for your load line, not read from the table.

Load through the centroid. This takes the a = 0 row of the tables, which the Manual says follows AISC 360-22 J2.4: for a linear group Eq. J2-5; for others the greatest of deformation compatibility, the elements summed with kds = 1.0, and Eq. J2-6 for groups of longitudinal and transverse lines.

Elastic method. Direct shear P / L plus the moment shear at each weld end, added as vectors, with kds = 1.0. It ignores ductility and is more conservative.

Manual table note. Between tabulated angles the Manual warns that straight-line interpolation may be wrong and says to use a rational analysis or the next lower angle: the note reads the next lower angle, and the check uses the analysis at your angle. For an L, the sign of θ picks 8-10 or 8-10a and 8-11 or 8-11a, with the return at the bottom; for a return at the top, change the sign of θ.

Limitations of the eccentric weld group

- Fillet welds in one plane under a load in that plane. PJP and CJP grooves, plug welds and out-of-plane moment on the weld are not covered.
- The connected parts are not designed beyond the base metal thickness that develops the weld: no net section, block shear, plate bending or the member behind.
- The Manual's special case of a load out of the plane of the group is not a separate input.
- Eq. J2-1 for long end-loaded welds is reported, not applied.
- A group with short lines far apart under an inclined load (outside the tables, k above 2) can have more than one equilibrium solution, up to about 7 % apart in a study of such cases. The calculation keeps the one nearest the elastic estimate and says so for custom and out-of-range groups: compare the elastic method there.

Verification examples

Verification status: 166 values compared in 134 examples (134 published, 0 hand calculations from the cited equations): 161 within tolerance, 5 recorded deviations. Code basis: the provisions it uses were read in the target edition.

From the AISC Design Examples v16.0 Vol. 1: II.A-26 (Solutions for a vertical load and a load at 75 degrees: C 1.88 and 3.45, 84.8 and 155 kips LRFD, 56.5 and 104 kips ASD) and II.A-27 (elastic: Ip 385, 47.2 kips LRFD, 31.5 kips ASD), and the weld groups inside II.A-2A, II.A-2B, II.A-10, II.A-11B, II.A-21, II.A-23, II.A-29 and II.C-1 (C, $\bar{x}$, a and the required size). All are within tolerance.

From the AISC Manual 16th Ed.: every readable coefficient of Tables 8-4 to 8-11a, 21,308 of 21,344 cells (two machine reads of the page images; 36 cells were not read and are not compared), against the method. All but 15 are within 1 % or half a printed digit, plus one more that matches the errata:

- Table 8-4, 30 degrees, k = 2.0, a = 0.40 is printed 3.54; the Manual's errata correct it to 3.34, and the method gives 3.33.
- Table 8-11a, 75 degrees, a = 0.10 and 0.15, k from 0.8 up, 11 cells: the printed values equal the a = 0 row and the method gives 1.1 % to 1.6 % more.
- Table 8-10, 0 degrees, k = 2.0, a = 0.10: printed 7.10, the method 7.19 (1.3 %), one solution; not explained.
- Table 8-4, 0 degrees, k = 2.0, a = 0.70, 2.4 and 2.6: printed 1.0 % to 1.1 % above the method; both reads agree.

Hand calculations, no published example: kds off, E60 and E80 to E110 (scaling by C1 and FEXX only), metric input, custom segments and the load point above the centroid. The 6.19 D / Fu base metal option is the Manual's formula with no worked example here.

CHAPTER 29

Member stability bracing

The strength and stiffness a brace must have to make a braced point of a column, beam or beam-column, by AISC 360-22 Appendix 6, and a comparison with the brace you have. Use it when a member's design assumes an unbraced length and the thing doing the bracing is not a floor or roof diaphragm: a strut, a kicker, a framing beam, a cross-frame.

The result reads "Calculated", because the calculation does not design the brace member: you give the brace's stiffness elements and its available strength.

Inputs of the stability bracing

- **Braced member and system:** column, I-shaped beam or beam-column; lateral point or panel brace; for a beam, torsional point or continuous bracing. A point brace holds one location; a panel brace ties one braced point to the next (Appendix 6.1). A brace on several members takes the sum of their requirements, so enter how many (6.1).
- **Required strength:** the member's axial force and moment, by load type (each ASCE 7-22 combination is evaluated) or directly. Enter the largest within the unbraced lengths next to the brace.
- **Lengths:** the member length and the number of braced points in the span (torsional L / n, and the drawing); the unbraced length Lbr next to the brace, the larger Pr / Lbr or Mr / Lbr where the two sides differ. The longest unbraced length the required strength allows is optional: for an intermediate point brace, Lbr need not be taken less than it (6.2.2, 6.3.1b, 6.3.2a). It comes from your member design, for example Manual Table 4-1 or 6-1. It is not used for panel braces, or for beam-columns (6.4(b)), and is required for continuous bracing.
- **Your judgment:** whether the brace is the one nearest an inflection point in double curvature (Cd = 2.0, Appendix 6.3.1a); Cb for torsional bracing; the Commentary's reduction (2n − 1) / 2n for a finite number of column point braces (Eq. C-A-6-6, off by default).
- **Section (beams and beam-columns):** an AISC W, S, M or HP shape, or ho, tw, the two flanges' Iy and the distances c and t for a built-up or singly symmetric section. For a torsional point brace, the web stiffener thickness and width if any.
- **The brace you have (optional):** elements in series, everything between the member and an unyielding support, connections included (6.1): a member in axial $A\,E\cos^2\theta / L$, a beam loaded at midspan $48\,E\,I / L^3$, a cantilever $3\,E\,I / L^3$, a beam in reverse curvature for torsion $6\,E\,I / L$, or a stiffness you enter. Then the braces in parallel, the angle of the brace to the brace force, and the available strength of one brace as you determined it, compression if it can buckle. Without them the calculation reports the required values and says the brace was not checked.

There is no auto-design: the calculation does not design the brace member.

Checks performed

Item	Basis
Column, panel	Vbr = 0.005 Pr; βbr = (1 / φ or Ω) 2 Pr / Lbr (Eqs. A-6-1, A-6-2), φ = 0.75, Ω = 2.00
Column, point	Pbr = 0.01 Pr; βbr = (1 / φ or Ω) 8 Pr / Lbr (Eqs. A-6-3, A-6-4)

Item	Basis
Beam, lateral panel	Vbr = 0.01 Mr Cd / ho; βbr = (1 / φ or Ω) 4 Mr Cd / (Lbr ho) (Eqs. A-6-5, A-6-6)
Beam, lateral point	Pbr = 0.02 Mr Cd / ho; βbr = (1 / φ or Ω) 10 Mr Cd / (Lbr ho) (Eqs. A-6-7, A-6-8)
Beam, torsional point	Mbr of Eq. A-6-9, at least 0.02 Mr; βT of Eq. A-6-11, φ = 0.75, Ω = 3.00; βsec of Eq. A-6-12; βbr of Eq. A-6-10
Beam, continuous torsional	Eq. A-6-9 over the longest unbraced length; Eqs. A-6-10 and A-6-11 with L / n = 1.0; βsec of Eq. A-6-13, no stiffeners
Beam-column	The column and beam values summed, with the actual Lbr (6.4(a), (b)); condition (d) stated when both flanges are in compression
Web distortional stiffness	βsec above the required system stiffness, or the torsional brace is not effective (User Note to 6.3.2a)
Web stiffness and stiffener the brace needs (note)	From the provided brace, Eqs. A-6-10 and A-6-12
Brace stiffness and strength	Your elements and available strength against the required values

Limitations of the stability bracing

- The brace member and connections are not designed: their stiffness and strength are your input.
- The alternatives of Appendix 6.1(a) and (b) (second-order analysis, buckling analysis) are not used. They can justify smaller values, which you may then enter as your own comparison.
- Column bracing is against flexural buckling only. Torsional and flexural-torsional buckling (E4) needs its own restraint.
- Beam bracing is for doubly and singly symmetric I-shapes under flexure in a plane of symmetry with no net axial force, as Appendix 6.3 says. For trusses, see the Commentary.
- A beam-column with torsional bracing is refused: Appendix 6.4(c) leaves the combination to the actual brace details. Run the flexure as a beam and the axial force as a column, then combine them for your brace.
- The drawing shows the braced points equally spaced. The calculation works from the Lbr entered.

Verification examples

Verification status: 38 values compared in 11 examples (11 published, 0 hand calculations from the cited equations): 38 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

From the AISC Design Examples v16.0 Vol. 1, Examples A-6.1 to A-6.6, LRFD and ASD (required strength and stiffness of column and beam point braces, Cd = 2.0, the Commentary's reduction, the longest allowed length, braces at an angle, elements in series, the torsional brace with its web stiffener: 6.30 kips and 46.7 kip/in.; 6.67 kips and 16.7 kip/in.; 11.7 kip-ft, 4,640 and 1,240 kip-in./rad, and a 3.25 in. stiffener): all within tolerance, no deviations.

Hand calculations only, no published example on disk: panel braces for columns and beams, beam-columns, singly symmetric sections and continuous torsional bracing. The Specification states L / n = 1.0 for the stiffness of continuous bracing and does not repeat it for the strength; this calculation reads it into Eq. A-6-9 as well, the reading with consistent units, in line with the Commentary's stiffness per unit length. Check this one yourself if it governs. A torsional brace that bends in single curvature (2 E I / L, Commentary) goes in as an entered stiffness.

CHAPTER 30

Steel joist

An open-web steel joist (K-series, LH-series, DLH-series or KCS) checked against the values of its designation in the SJI load tables, to the SJI Standard Specification (SJI 100-2020) and the SJI Code of Standard Practice (COSP 2020), with ASCE 7-22 load combinations. The load tables are the joist manufacturer's design: you choose a designation and type its load-table values at your span; the calculation checks that the designation carries your loads, uniform or not, and tells you what to show on the drawings for the joist manufacturer.

Inputs of the steel joist

- **Joist:** the series (K, LH, DLH or KCS), the nominal depth (the depth at midspan, SJI 100 5.3) and the section number (the last digits of the designation: 5 for 18K5, 06 for 32LH06, 3 for 22KCS3). A KCS joist also takes the bridging section number its load table lists (the equivalent K section).
- **Span and spacing:** the span as SJI 100 Figure 5.2-1 defines it (clear span plus the standard clear bearing lengths) and the joist spacing (the tributary width of the area loads). The joist is analysed on its design length, the span less 4 in.: the end reactions sit 2 in. inside each end of the span (SJI 100 5.2 and 5.4).
- **Load table values at the span:** for a K, LH or DLH designation, the total load W of the table for your design method (the black figure of the LRFD table, factored, or of the ASD table) and the red figure (the unfactored uniform load that deflects the joist span / 360). Interpolate between span columns where your span falls between them, as the table notes allow. For a KCS joist: the moment capacity, the shear capacity and the gross moment of inertia of the KCS table. The approximate joist weight is added to the dead load when *Include joist weight* is on. A load/load designation (for example 24K300/175) is checked by entering its equivalent standard section.
- **Loads:** area loads (psf on the joist spacing, full length), line loads (uniform, partial or linearly varying, a snow drift for example) and point loads (lb), each with its load type (D, L, Lr, S, R, W). Positions are measured from the left reaction along the design length. Every ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combination of the load types present is analysed (SJI 100 4.1.2); W acts in both directions. See Loads and combinations.
- **Bearing and bridging:** the support (steel, or a steel bearing plate on masonry or concrete), the bearing length (the joist end past the face of the support), the bearing plate width on masonry or concrete, whether you accept a shorter bearing on masonry or concrete (SJI 100 5.4.1.2 permits it when the plate and support get a special design), and the shading of the load table at your span (erection bridging).
- **Deflection:** live load and total load limits as span / n, on the design length. The SJI 100 5.10 roof value 240 is prefilled for live load (360 for floors and for roofs with a plaster ceiling); 0 turns a check off. The shear deformation factor divides the gross moment of inertia (1.15, prefilled, reproduces the red figure exactly).

Auto-design is not offered: the joist capacity comes from the load-table values you type, so there is nothing for the calculation to size. Pick a heavier section or a deeper joist from the table and enter its values.

Checks performed

Check	Basis
Moment within the load-table envelope (K, LH, DLH)	The required moment at every point against W x (L - x) / 2 of the table load W (COSP 2.4(a) Option 1: the moment and shear envelopes of the selected uniform load must cover the actual moments and shears)
Shear within the load-table envelope (K, LH, DLH)	The required shear at every point against W |L/2 - x|, never less than 25 % of the end reaction W L / 2, the least shear the webs are designed for (SJI 100 4.4.2)
Moment capacity, shear capacity (KCS)	The largest moment and the largest shear against the KCS table values (flat envelopes, SJI 100 1.4 and 4.4.2.3; COSP 2.4(a) Option 4)
Distributed load (KCS)	825 plf LRFD, 550 plf ASD (SJI 100 1.4; end panel bending, COSP Option 4(c))
End reaction (K, KCS)	13,800 lb LRFD, 9,200 lb ASD: beyond it an LH joist with a 5 in. seat (SJI 100 1.4 user note, COSP 2.4 caution)
Span to depth	Span ≤ 24 × depth (SJI 100 5.2)
Standardized range	Depth and span within the series' standardized range (SJI 100 1.4 user notes; advisory)
Live load deflection	Each of L, Lr, S and R alone (service), with the effective moment of inertia below
Total load deflection	The ASD gravity combinations (service)
Bearing length	On steel: Table 5.4-1 (2-1/2 in. for K and LH02-06, 4 in. for LH07-17 and DLH10-17, 6 in. for sections 18 to 25); over masonry or concrete: 4 in. K, 6 in. LH and DLH (5.4.1.2)
Bearing plate width	On masonry or concrete: Table 5.4-2 (7, 9 or 14 in.)
Bearing seat, attachment	Standard seat depth (Table 5.4-3: 2-1/2, 5 or 7-1/2 in.) and the minimum seat welds or bolts (Table 5.7-1), as a note
Rows of bridging	Table 5.5-1 by section number, depth and span; bottom chord rows not fewer (5.5.3.2)
Erection stability	The load-table shading at the span (5.5.2.1, 6.1); spans over 60 ft need the blue or gray shading
Net uplift	When a combination with W lifts the joist: the uplift to show on the drawings and the bottom chord bridging row (5.12)

The equivalent uniform loads of the COSP 2.4 caution, 8 Mmax / L^2 and 2 Rmax / L, are shown in the steps and with the key results; for a uniform load they equal the load, and the two envelope checks reduce to w / W.

Deflection. The gross moment of inertia of a standard joist comes from the load-table note $I_g = 26.767\ W\ L^3 \times 10^{-6}$ in^4 (W the red figure, L the span less 0.33 ft); a KCS joist uses the table's value. The deflection is computed with I_g divided by the shear deformation factor: 1.15 makes the red-figure load deflect the design length / 360 exactly, so for a uniform live load the check equals the load-table proration (the red figure times 360 / n). Point and partial loads are analysed with the same stiffness.

What the calculation does not decide. The joist manufacturer designs the chords, webs, panel points, connections, bearing seats and bridging details, and verifies them per SJI 100 4.7. The specifying professional (you) gives the loads by type and their combinations, the deflection limits, net uplift, the ponding investigation (5.11), the supporting members, bearing plates, anchorage and bridging terminations. With point or partial loads the calculation reminds you to show them on the drawings (COSP Option 2, or a load diagram with an SP designation, Option 5), and, where their locations are not given, the extra web from each load to a panel point on the opposite chord; loads of 100 lb or less per chord panel, concentric with the chord, need none (SJI 100 4.1.2).

The drawings show the joist elevation on its design length with the loads by type (the web drawn as a schematic, since the panel layout is the manufacturer's), the end bearing (seat depth, bearing length, support) and the shear, moment and deflection diagrams of the governing combinations.

Limitations

- Simple-span joists on two bearings. Top chord extensions, extended ends, cantilevers, continuous (moment) frames, axial loads and end moments are not covered (SJI 100 4.4.4, 5.1.2): use a special designation with a load diagram.
- The capacity is the load-table value you type; the module does not hold the SJI tables. A designation that the table does not list at your span is your responsibility (the bridging check flags spans outside Table 5.5-1).
- Local chord bending from loads between panel points (add-loads and bend-checks, COSP Option 3) is the joist manufacturer's design; the calculation states what to specify.
- Joist girders, CJ composite joists, sloped seats (the high-end seat depth of COSP Tables 2.2-2 and 2.2-3), ponding (5.11) and floor vibration (SJI Technical Digest 5) are not covered.
- The bridging spacing limit (Eqs. 5.5-2 and 5.5-3) needs the top chord radius of gyration, which only the joist manufacturer knows: the calculation reports the number of rows only.

Verification examples

Verification status: 28 values compared in 7 examples (7 published, 0 hand calculations from the cited equations): 27 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

SJI Code of Standard Practice (COSP 2020), Section 2.4(a) Option 4, ASD examples (pp. 181 to 183), loads entered as D on the design length:

Example	What it checks	Published	ILX Calcs
Option 4, Example 1	240 plf and 1,000 lb at 8 ft, L = 40 ft, 22KCS3	M = 625 kip-in, RL = 5,600 lb, RR = 5,000 lb; 22KCS3 658 kip-in / 6,600 lb	625.0, 5,600, 5,000; 658 / 6,600
Option 4, Example 2	270 plf, 160 plf triangle over 8 ft, 500, 800 and 300 lb, L = 30 ft	M = 443 kip-in, RL = 5,000 lb, RR = 5,340 lb, maximum uniform load 430 plf	442.8, 5,000, 5,340, 430
Option 4, Example 3	300 plf, 500 plf over 15 ft, two 2,000 lb loads, L = 55 ft	M = 2,910 kip-in, R = 14,000 lb, uniform load 800 plf (exceeds the 30KCS5 and the 550 plf limit)	2,909, 14,000, 800 (all three checks not satisfied)
Option 4, Example 3, Option A	Two joists, each half the load, 28KCS5	M = 1,455 kip-in, R = 7,000 lb, 400 plf; 28KCS5 1,704 kip-in	1,455, 7,000, 400; 1,704

The LRFD columns of these examples are the ASD loads times 1.5 (factored loads shown); no ASCE 7 combination of the typed loads reproduces that, so they are not compared.

SJI webinar "Properly Specifying Steel Joists" (2019):

Example	What it checks	Published	ILX Calcs
Option 1 (slide 53)	20 psf dead, 30 psf live, 5 ft spacing, 25 ft	250 / 150 plf	250, 150

Example	What it checks	Published	ILX Calcs
Special deflection limits (slides 72, 73)	96DLH21 at 116 ft, 1149 / 877 plf, 20 psf live at 10 ft	L/360 = 3.86 in.; 200 plf deflects 3/4 in.	3.86; 0.88 (see below)

The slide computes 200 / 877 × 3.86 in. and prints 3/4 in.; the product is 0.88 in., which ILX reports (a listed deviation). The 3.86 in. limit is the design length (115.67 ft) / 360, the length ILX uses.

SJI 100-2020 tables and the K-series load-table notes, for the default 18K5 at 30 ft: two rows of bridging (Table 5.5-1), 2-1/2 in. minimum bearing on steel (Table 5.4-1) and seat depth (Table 5.4-3), 3/8 in. camber (Table 4.6-1), Ig = 112.6 in^4 from the table note (W = 161 plf). The rows for 22K9 at 40 ft (3) and DLH21 at 116 ft (3) and the 7-1/2 in. seat of DLH21 are checked the same way. No published worked example of the moment and shear envelopes of a standard designation under non-uniform loads (COSP Option 1) was found: those checks follow the cited definitions and are covered by hand-verifiable tests (a uniform load gives w / W; a midspan point load meets the 25 % shear floor).

CHAPTER 31

Steel deck (gravity)

A bare fluted steel deck panel, per foot of width, on one to six equal spans with an optional cantilever at the right end: strength by AISI S100-16 (with S2-20) from the section properties in the manufacturer's table, and deflection, with its construction stage. It is a check of a deck you have chosen. It takes no section properties of its own, since a deck's effective properties are its manufacturer's. Diaphragm shear is not checked here. Composite deck-slab strength is not checked: use the manufacturer's tables.

What the SDI items rest on. ANSI/SDI SD-2022, the current steel deck standard, was not read for this calculation. Construction loads and the concreting deflection limit come from the sources named beside each input, are marked (to confirm against the code text) and are all inputs. Confirm them against the standard.

Inputs of the steel deck

- **Use:** roof deck, or form or composite deck during concreting. On form or composite deck only the bare deck under wet concrete is meaningful here; clear the area loads, or enter what the bare deck carries.
- **Deck name, equal spans, span, cantilever:** spans center to center. The cantilever is at the right end, at most one span.
- **From the manufacturer's table:** base steel thickness, yield stress (the grade the properties were computed at), weight, effective section moduli Se+ and Se-, and the deflection moments of inertia for a single span and for continuous spans. Manufacturers name these differently: Id+ or Idn for a single span; Idm, or Id- where a table has no multi-span value. Use the values for your deck: properties differ between manufacturers.
- **Web shear and crippling:** computed from the web's geometry (webs per foot, flat depth, inside radius, web angle, fastened at 18 in. or closer, bearing lengths), or typed from the manufacturer's tables at your bearing length. A manufacturer who prints no radius or web angle needs the second way.
- **Loads:** area loads by type, uplift negative, and line loads across the flutes per foot of width. A point load must be spread across the flutes by you: this calculation does not do it.
- **Pattern live, roof live and snow span by span:** on by default, ASCE 7-22 4.3.3. Manufacturers' load tables load all spans at once, so their tables give less moment at midspan than this. Turning it off is your decision.
- **Deflection limits:** span / n, from IBC Table 1604.3; a cantilever is on twice its length. Enter 0 to skip.
- **Construction stage:** on by default.
  - **Concentrated load** on a 1 ft width at each midspan and at the tip: 200 lb prefilled for roof deck, from a manufacturer's table note citing ANSI/SDI RD-2017, and 150 lb on form deck, from SDI's 2006 Manual of Construction, Figure 16. A Nucor / Vulcraft lecture also shows 150 lb on a roof deck: the prefilled 200 lb is the larger. (to confirm against the code text)
  - **Uniform load:** 20 psf on form deck, same manual. On roof deck, 10 psf, added at a cantilever's tip load as that table note says. That lecture also lists 50 psf: enter it if your standard calls for it. (to confirm against the code text)
  - **Deck weight:** on by default. The construction span table compared below leaves it out on simple and continuous spans, so the published cases clear it. Leaving it on is on the safe side.

- **Wet concrete weight and a factor on a single span,** 1.5, because SDI's manual adds half the concrete weight where a single span is hard to pour under control.
- **LRFD factors:** 1.2 on deck, 1.6 on concrete, 1.6 on construction loads. These are not read from the standard. ASD uses 1.0. (to confirm against the code text)
- **Concreting deflection:** span / 180 and at most 3/4 in., from an SJI / SDI webinar slide. (to confirm against the code text)

Auto-design is not offered: the calculation checks one deck against table values you type, so there is nothing for it to size.

Checks performed

Check	Basis
Positive and negative moment	S100 F3.1: Mn = Se Fy, φ = 0.90, Ω = 1.67
Web shear	S100 G2.1 on one web, kv = 5.34, resolved by the sine of the web angle, times webs per foot; φ = 0.95, Ω = 1.60; or the table value
Web crippling, end and interior	S100 G5, Table G5-5, one-flange, per web then per foot; or the table value
Bending with shear	S100 H2, Eq. H2-1, along the deck
Bending with web crippling	S100 H3(a): the exception for webs 10 in. or closer applies to ordinary deck, so this normally is a note; otherwise Eq. H3-1
Live and total deflection	IBC Table 1604.3 limits as entered, on the entered moment of inertia
Construction moment, shear, reactions	Loads as above with S100 strengths
Concreting deflection	As entered, form deck only

The key results also give the uniform load the deck can carry on all spans, by stress and at your live deflection limit, directly comparable with a manufacturer's table.

Limitations of the steel deck

- Equal spans only; one cantilever, at the right end.
- Section properties are entered. Perforated and acoustic webs are not handled: enter table values for them where the manufacturer prints them.
- One-flange web crippling only. A line load between supports, or one directly over a support, may be a two-flange case: check it yourself.
- Not designed: a point load's spread across the flutes, fasteners, side laps, uplift at the connections, diaphragm shear, ponding (use Rain load), the slab.
- The construction stage places its concentrated load at midspan, as the published table does. A position slightly off midspan gives about 2 % more moment on two spans.
- ANSI/SDI SD-2022, its roof design manual and its construction load wording were not read. See the top of this chapter.

Verification examples

Verification status: 38 values compared in 17 examples (17 published, 0 hand calculations from the cited equations): 35 within tolerance, 3 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Source	What
Vulcraft PunchLok II catalog (Dec. 2016), Design Example, pp. 13-14	1.5PLB22, three 5 ft spans: allowable load by stress 177 psf; reactions 60 and 165 lb/ft, shear 90 lb/ft; allowable reactions and shear
Same catalog, gravity load tables, p. 21	1.5PLB22 load table cells for one, two and three spans, by stress and at span / 360, / 240, / 180
Nucor / Vulcraft lecture, example, slide 24	Three 6 ft spans, 2 psf dead and 60 psf live on the adverse spans: reactions 166.8 and 445.2 lb/ft (pattern loading)
IAPMO ER-2018, Verco, p. 25	PLB-36, 22 to 16 gage: Mn+, Mn-, Vn
Intsel, Table 3.1, citing ANSI/SDI RD-2017	The printed single and double construction spans give a strength ratio of 1.00 at 200 lb, no deck weight

The Vulcraft catalog prints no radius or web angle, so the worked example's allowable reactions and shear use those of Verco's PLB-36, 22 gage. Where that leaves a difference it is listed on the Verification page. All published comparisons are ASD: LRFD uses the same nominal strengths and factors, and is checked by hand in the behaviour tests.

CHAPTER 32

Steel deck diaphragm

One zone of a bare fluted roof or form deck working as a diaphragm between two reaction lines: support and sidelap fasteners, panel stability, wind uplift at the fasteners, stiffness and deflection. To AISI S310-20 with Supplement 1 (2022), with AISI S100-16 for the fasteners.

Gravity on the same deck is the steel deck (gravity) calculation. This one takes only in-plane shear.

Inputs of the deck diaphragm

- **Diaphragm:** span between the lateral supports, depth B along the reaction line. Shear is S = w × span / (2 B).
- **Deck:** 1.5PLB or 3.0PLN, Vulcraft's, with depth, flats, gross moment of inertia and warping constants from its 2016 catalog, or other deck. The catalog marks its dimensions approximate: **check them against your product.** Other deck takes depth, pitch, half the bottom flat e, top flat f, cover width, thickness and Ixg from your manufacturer. The web length and angle follow from those. All are prefilled with 1.5PLB at 22 gage, not a generic deck.
- **Warping constant D:** the catalog's for a listed deck and pattern. Otherwise S310 Appendix 1 Section 1.4 from the flats and the valley spacing at the panel ends, or entered.
- **Steel:** Fy and Fu of the deck. 50 ksi is the catalog's minimum, and 62 ksi the value its connection tables state. Yours may differ.
- **Deck span and spans in a panel:** S310 Table 1.3-1 sets the warping factor by spans in a panel, and 7 or more take one value. The catalog's tables assume 3 spans.
- **Fastener pattern:** 36/7, 36/4 on 1.5PLB and 24/4 on 3.0PLN, as Vulcraft draws them, or **your own positions** from the panel centerline, one per fastener, up to 30. Pattern constants come from the positions. Listed patterns put the sidelap fastener 1/2 in. in from the panel edge, which is what reproduces the catalog's printed factors.
- **Support connection at the sidelap:** one each side, or one through both sheets. S310 Commentary D1 gives N for each: both are counted, or A is left out. The catalog's strength tables work out with one each side. It matters: N sets the corner limit, which usually governs.
- **Support fastener:** No. 12 or No. 14 screws (S100 J4.3), 3/4 in. arc spot welds or others (S100 J2.2.2.1), or a strength entered from tests or a fastener's report (S310 D1.1.3, D1.1.5). S310 limits Eq. D5.2.2-1 to these two screws. For a thin support the flexibility follows S310 Commentary Figure C-D5.2.2-1.
- **Screw strengths Pnss and Pnts:** S310 asks for your manufacturer's. 0 takes the value printed in the SSMA Guide.
- **Sidelap fastener:** PunchLok II by the catalog's equations (Pns and Ss), screws (S100 J4.3, S310 Eq. D5.2.2-2), arc spot welds (S100 J2.2.2.2), button punches (S310 D1.2.6, Eq. D5.2.5-1, and no strength above 3 in. depth), or entered from tests with both Pns and Ss (S310 D1.2.7, D5.2.6).
- **Sidelap and edge fasteners per deck span:** counted over the panel for ns and ne. Edge fasteners go into the edge support at a reaction line: your detail.
- **Deck web:** bearing at the exterior support, inside bend radius, web flat. The catalog prints no radius: 3/16 in. is prefilled so you enter your deck's. It enters only Snℓ.

- **Uplift:** a net pressure at the required level for your method, with a reaction coefficient, or a force per fastener. T = coefficient × pressure × deck span × the widest tributary at a fastener. 1.1 is the interior reaction of three equal spans. Yours depends on your layout. **0 applies no uplift, and the calculation says so.**
- **Pull-over diameter:** S100 J4.4.2 for your screw head and washer, at least 5/16 in. in tension.
- **Pnft from tests:** S310 D3.1. Required for an entered support fastener when there is uplift.
- **Deflection:** a load factor on the strength-level line load, and span / n. S310 C3 leaves both to you.
- **Loads:** wind and seismic line loads along the span at strength level. Each load kind takes its own factors.

Auto-design varies the gage, the pattern across the panel (listed patterns), and sidelap and edge fasteners per span. The deck, its span, the diaphragm, the kind of fastener and the loads are kept.

Checks performed

Check	Basis
Limits of the method: depth, thickness, steel, pitch	S310 Chapter D (a) to (d)
Diaphragm shear by connections, wind and seismic: the least of Sni, Snc, Sne, Snp	S310 Eqs. D1-1 to D1-4a, Table B1.1-1
Support fastener strength	S100 J2.2.2.1 or J4.3, S310 D1.1.1, D1.1.2, or entered
Sidelap fastener strength	S310 D1.2
Factors: by load and fastener, and a mix takes the more severe	S310 Table B1.1-1, B1.1
Diaphragm shear by stability: the lesser of Sno and Snℓ	S310 Eqs. D2-1, D2-2, D2-3
Shear and uplift at the support fastener, wind	S310 D3.1.1, D3.1.2.1 (a), (b), (c), S100 J2.2.3, J4.4
Stiffness G'	S310 Eqs. D5.1.1-1, -2, D5.2, Table 1.3-1, Appendix 1
Deflection: shear of the deck only, simple span	S310 C3, with the mechanics $\Delta = w L^2 / (8 B G')$
Support fastener spacing at most 18 in.	S310 D1
Sidelap and edge spacing at most 3 ft, for Lv above 5 ft	S310 D1

Wind uplift lowers Sni, Snc and Sne through Pnft, as S310 D3.1 states. D3.1 does not name Snp. This calculation lowers it too, on the safe side. The edge fastener Pnfs is not reduced, as D3.1 permits.

S310 D3.1.2.1 applies its pull-over and pull-out equations only within the limits of S100 J4.5.1 and J4.5.2, and asks for tests outside them. **A No. 12 screw into a support thicker than 0.0724 in. is outside J4.5.2**, which is most joists and beams. Outside the limits this calculation uses the linear Eqs. D3.1.2.1-13 to -18 with the S100 J4 factors, and a note says so. S310 states those equations for strengths from small-scale tests, so this is your determination. Enter Pnft from tests if you have it.

An entered support fastener with uplift reads NG, with a row saying the interaction is not checked, until Pnft is entered. S310 D3.1.3 gives its interaction by tests, or from the fastener's pull-over and pull-out strengths.

Equations with an anomaly: S310's User Notes and Commentary say Eqs. D3.1.2.1-2, -4, -6, -8 reduce the shear strength by 6 to 23 percent as the tension approaches zero. The calculation applies them as written once there is any uplift.

Limitations of the deck diaphragm

- **One zone.** Zone the span with a calculation for each zone.
- **The edge panel takes the interior panel's patterns and width.** S310 asks for both panels to be checked if they differ.
- **Where shear reaches a line from two sides**, S310 compares Sne with the reaction per length. Here it is compared with the shear entered.
- Not covered: concrete fill (D4), cellular, acoustic and standing seam panels, arc seam welds, top arc seam sidelap welds, wood supports, insulation between panel and support, weld washers.
- Not designed: chords, collectors, edge supports, their connections, or fastener edge distances. The drawing says so.
- Deflection is the shear of the deck only. Chord flexure, chord splice slip and framing are yours.
- Support and uplift forces are taken at the widest tributary, on one fastener. S100 J2.2.3's 70 percent for a weld at a sidelap is taken whenever the pattern has one, on the safe side.
- A manufacturer's evaluation report may modify these equations, as IAPMO ER-2018 does for Verco's decks. This calculation follows S310 as written.
- Older catalog tables use the factors of AISI S100 Table D5. **Do not expect this calculation to match them.** Its #12 screw strength from S100 J4.3 is also lower than the older manual value in the catalog's Table 1.

Verification examples

Verification status: 66 values compared in 39 examples (34 published, 5 hand calculations from the cited equations): 63 within tolerance, 3 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Compared with the Vulcraft PunchLok II catalog (December 2016) and AISI S310-20 Commentary Table C-1.2. Strength compares **nominal** values, because the catalog's factors are the older ones: the catalog's allowable value times its own factor, against Sn before any factor.

Example	Source	Published	ILX Calcs
1.5PLB22, 36/7, 5 ft, 3 VSC2, seismic	Catalog p. 32, 623 plf × 2.50	1,558 plf	1,558 plf
Same, 4 VSC2, wind	Catalog p. 32, 771 plf × 2.35	1,812 plf	1,811 plf
1.5PLB22, 36/4, 5 ft, 3 VSC2, seismic	Catalog p. 24, 486 plf × 2.50	1,215 plf	1,215 plf
3.0PLN22, 24/4, 15 ft, 4 VSC2, seismic	Catalog p. 60, 261 plf × 2.50	653 plf	653 plf
1.5PLB20, 36/4, welds, 5 ft, 3 VSC2, seismic	Catalog p. 42, 746 plf × 3.00	2,238 plf	2,233 plf
1.5PLB22, 10 ft, buckling	Catalog p. 32, 709 plf × 2.00	1,418 plf	1,417 plf
G', 1.5PLB22, 36/7, 5 ft, 4 VSC2	Catalog p. 32	61 kip/in.	60.6 kip/in.
Worked example: Dn, C	Catalog p. 11	2.73, 4.66	2.73, 4.66
D, WR deck, every valley, t = 0.0295 in.	S310 Commentary Table C-1.2	1,237 in.	within 1 %
N, each side and common	S310 Commentary, printed p. 15	7/3, 5/3 per ft	7/3, 5/3 per ft

Differences stated plainly. The catalog's worked G' of about 98 kip/in. multiplies by 0.90, its own factor for three spans, on four spans. This calculation uses S310's 0.80 and gets 101 kip/in. The catalog's weld tension values are 9 percent below S100 Eq. J2.2.3-2 at the steel its table states, for a reason it does not give. The catalog's wind tables with uplift use an older interaction, so they are not a check of S310 D3.1.2.1.

Hand calculations, marked as such: Pnf of a screw, the factors of Table B1.1-1 on Snf, Snp, Sne.

Not verified against a published value: Snℓ, Sni apart from Snc, the uplift interaction under S310's factors, a full ASD or LRFD diaphragm worked to S310-20 from end to end. The current SDI Diaphragm Design Manual is not used by this calculation.

CHAPTER 33

Steel truss

Steel truss: pick a layout, span and pitch, and get chords and webs sized and checked to AISC 360-22, with ASCE 7-22 load combinations, for roof and floor trusses.

Inputs of the steel truss

- **Roof or floor, truss type:** roof trusses offer 18 layouts, floor trusses the parallel-chord ones.
- **Geometry:** span between the outer bearing lines, pitch (rise in 12), heel height, depth on parallel chords, flat length on a hip, rise on a bowstring, room width and height on an attic, overhangs, panels (Auto: about 6 ft on a pitched truss, about 1.5 depths on parallel chords; a count the layout cannot have snaps to the nearest one and the results say so).
- **Supports:** pin, roller or none at each end, intermediate supports (moved to the nearest chord joint, and the results say where), bearing on the bottom or top chord of a parallel-chord truss.
- **Loads:** area loads in psf times the truss spacing: top chord dead, roof live, snow, unbalanced snow, floor live, bottom chord dead and live. Live load on half the span as well (ASCE 7-22 4.3.3, on by default, because web forces reverse under it). Wind rows (pressures normal to the top chord from your wind analysis, each checked from both directions), total horizontal seismic, SDS for the vertical effect, and point loads anywhere on a chord, also between panel points.
- **Members:** Size the members automatically is on by default: at every calculation the lightest section of each group that passes is chosen. Turn it off to enter the sections and have them checked as they are.
- **Bracing:** what holds each chord out of plane, and the webs: as needed (a compression web that fails unbraced gets a lateral brace at mid-length before anything is upsized, and the results, schedule and sheet name it), none, or every web. These are your judgment as engineer of record, and the braces themselves are not designed.
- **Members with their own section:** a mark from the schedule and a section, for one heavy web or chord panel. The rest keep the section of their group.
- **Serviceability:** live and total deflection limits as span / n, and the horizontal movement limit at a roller.

Auto-design varies the top chord, bottom chord and web sections, lightest first, within the given families, and fixes the result (it turns automatic sizing off). It never changes the geometry, loads, bracing or code options.

Analysis. A plane frame, first-order, with chords continuous over the panel points and pinned at heels and pitch breaks, and webs pinned at both ends, or every panel point pinned if you choose. Loads between panel points are carried in chord bending, so a chord has axial force and bending together. Every load case is solved once and combined with ASCE 7-22 2.3.1 or 2.4.1 (and 2.3.6 or 2.4.5 with seismic). Deflections always use the ASD combinations.

- **Sections:** W, WT, double angle, HSS or pipe; the strong axis lies in the plane of the truss, and a WT or double angle has its stem or web legs toward the webs. Fy and Fu by family. Net area An/Ag and shear lag U are your inputs because the end connection is not designed. Double-angle connector spacing and type (AISC 360-22 E6).

Checks performed

Check	Basis
Tension yielding, rupture	AISC 360-22 D2
Compression	E3, E4 (flexural-torsional of WT and double angles), E6 (built-up), E7 (slender elements)
Flexure, each sign	F2, F7, F8, F9, F10.3 (a stem or web leg in compression is much weaker)
Shear	Chapter G
Axial + bending	H1.1, with B1 of Appendix 8 at every station
Deflection	Your limits, IBC Table 1604.3
Slenderness	E2 and D1 user notes, advisory

The report prints the steps of the governing member of each group and lists every member in the schedule. Turn on Report every member step by step, under Serviceability, to print every member.

Limitations of the steel truss

- **Connections are not designed.** Gussets, welds, bolts, splices, heel joints and anchorage (ILX Connection covers gussets), block shear. The forces in every member at every joint are listed for that design.
- One section each for the top chord, bottom chord and webs, apart from the members you give their own.
- One lateral brace per web at most, at mid-length.
- Work lines meet at the joints: joint eccentricity and secondary moments are not modeled. First-order analysis, with moments amplified member by member where the standard does so.
- Gable-end, gambrel, piggyback and unequal-pitch layouts are not generated. Use the 2-D truss analysis for any other geometry.
- Ponding, fire, floor vibration, system bracing design, wind pressure calculation and trusses carrying other trusses (use point loads) are outside this calculation.
- Single angles in compression (E5) are not offered.

Verification examples

Verification status: 39 values compared in 9 examples (8 published, 1 hand calculations from the cited equations): 39 within tolerance, 0 recorded deviations. Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

These results carry the label "Preview: not yet verified, do not rely on these results" on the screen, in the report and on the sheet. The frame solver is checked against closed-form beams and the pin-jointed solver. Member strengths run through the module against AISC Design Examples v16.0 D.3, D.6, E.5, E.6, E.7 and F.10, with hand statics for the forces. The combined check has no published example, and no published complete truss is on file.

CHAPTER 34

Steel beam with torsion

A steel beam loaded off its shear center: a W, S, M or HP shape, a channel, a welded I-girder of plates, or a rectangular or round HSS or pipe. The calculation finds the torsional rotation and its derivatives along the span for the end restraints you choose, combines the torsional stresses with the bending stresses point by point, and checks them to AISC 360-22 Chapter H, following AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), with ASCE 7-22 load combinations.

Inputs of the steel beam with torsion

- **Member:** a rolled shape from the AISC Shapes Database v16.0 or a doubly symmetric welded I-girder (depth, flange width and thickness, web thickness; its torsional properties come from DG9 Eqs. 3.4 to 3.11). The steel grade must suit the section: A992, A572 Gr. 50 or A36 for open shapes, A500 or A1085 for HSS, A53 Gr. B for pipe. Self-weight is optional; it acts at the centroid, which for a channel is eo + $\bar{x}$ from the shear center and so adds a torque.
- **Length and end conditions:** each end is set twice, because the two are independent. *Flexure:* pinned, fixed or free. *Torsion:* pinned (twist prevented, warping free), fixed (twist and warping prevented) or free. This is your judgement as the engineer: DG9 Appendix C.2 warns that a torsionally fixed end is harder to achieve than a flexurally fixed one, and that simply welding to an end plate or a column flange may not provide it. The calculation does not design the end detail. For a closed section the torsional pinned and fixed ends act alike (no warping).
- **Loads:** point loads and full or partial line loads per load type (D, L, Lr, S, R, W, E), each with its eccentricity e from the shear center (for a channel, positive toward the flange tips; a load at the centroid has e = eo + $\bar{x}$). The torque is P e or w e. Every ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combination of the load types present is analysed; W and E act in both directions. See Loads and combinations.
- **Lateral bracing:** of the compression flange, continuous, at the ends only, at a spacing or at given points. Cb is computed (Eq. F1-1) or entered.
- **Load points free to displace laterally and twist:** on by default where the beam is not continuously braced. On, the warping normal stress is amplified as DG9 4.7.3 suggests (below). Clear it where the load points are braced against lateral displacement and twist. This too is your judgement.
- **Rotation and deflection limits:** DG9 4.8 gives no rotation limit: it depends on what the beam supports (masonry, curtain wall). The prefilled 0.5° is only a starting value; set your own, or 0 to turn the check off. Live load 360 and total load 240 are the IBC Table 1604.3 floor values; 0 turns a check off.

Auto-design finds the lightest shape of the same family (W, S, M, HP; C or MC; rectangular HSS; round HSS; pipe) that passes every check, with an optional depth limit. A welded I-girder is sized by you.

Checks performed

Check	Basis
Flexural strength	AISC 360-22 F2 and F3 (welded I: Table B4.1b Case 11 and the kc of F3), channels with c of Eq. F2-8b; HSS and pipe F7, F8

Check	Basis
Shear strength	AISC 360-22 G2.1 for open sections, G4 and G5 for HSS and pipe
Normal stress yielding (open sections)	f_un = bending + warping normal stress against Fn = Fy (H3.3(a), Eq. H3-7), DG9 Eqs. 4.8a, 4.8b
Shear yielding (open sections)	f_uv = St. Venant + warping + flexural shear stress against Fn = 0.6 Fy (H3.3(b), Eq. H3-8), DG9 Eqs. 4.9a, 4.9b
Buckling under normal stress (open sections)	f_un against Fn = Fcr = Mn / Sx of the unbraced segment (H3.3(c), Eq. H3-9), where lateral-torsional or flange local buckling can occur
Bending stress below the elastic buckling stress	With the amplification on: σb / (φ Fe) must stay below 1 (DG9 4.7.3)
Torsional strength (HSS, pipe)	Tn = Fcr C (H3.1, Eqs. H3-1 to H3-5); pipe C from the H3.1 User Note
Combined torsion, shear and flexure (HSS, pipe)	Eq. H3-6 at every point where the required torsion exceeds 20 % of the available torsional strength (H3.2); elsewhere torsion is neglected
Torsional rotation (service)	The largest rotation under the ASD combinations, in both design methods (DG9 4.8), against your limit
Live and total load deflection	Vertical deflection of the shear center against span / n; a cantilever uses twice its length (IBC Table 1604.3 note h)

The torsional analysis. Open sections solve E Cw θ'''' − G J θ'' = t (DG9 Eq. C.4) exactly, piecewise between the load points, with θ, θ' and θ'' continuous and the internal torque T = G J θ' − E Cw θ''' falling by each concentrated torque (DG9 C.2): pinned ends θ = θ'' = 0, fixed ends θ = θ' = 0, free ends θ'' = 0 and T = 0. DG9 instead reads the same functions from the charts of its Appendix B; the module needs no charts and handles any combination of loads and end restraints, each load type solved once and superposed. Closed sections have no significant warping (DG9 4.6.2) and carry St. Venant torsion only, T = G J θ'.

Stresses at every station (DG9 Chapter 4): τt = G t θ' (Eq. 4.1), τw = E Sw θ'''/t (Eq. 4.2a), σw = E Wn θ'' (Eq. 4.3a), σb = M / Sx (Eq. 4.5), τb = V Q / (I t) (Eq. 4.6). For an I-shape the maxima always add (DG9 4.6.1). A channel has no such rule, so the signs are tracked at the flange tip (point 0) and the flange-web junction (point 2), Wn0 = (h/2)(b' − E0) and Wn2 = E0 h/2 with E0 = eo + tw/2; a channel's shear stresses at the flange points are added as absolute values, with the flexural shear at point 1 included (conservative: DG9 Example 5.5 adds them with their signs and takes none at point 1). The maximum stress is sought along the whole span, not only at the stations a hand calculation would pick.

Amplification (DG9 4.7.3). Where the load point can displace laterally and twist, σw is multiplied by φFe / (φFe − σb) in LRFD (Eq. 4.22), and by the ASD equivalent with Fe / Ω (Eq. 4.23), with Fe from Eq. F2-4. The guide calls this conservative and applies it at the load point; the module applies it along every unbraced segment, which is more conservative still. The check above stops the calculation from reporting a stress once σb reaches the buckling limit.

Specification. DG9 was written for the 1993 LRFD and 9th Edition ASD specifications. The module uses the limit states of AISC 360-22 H3 (φT = 0.90, ΩT = 1.67 on all of them).

The drawings show the elevation with the loads by type, the section with the shear center and the governing load at its eccentricity (dimensioned), and the moment, warping stress or torque, and rotation diagrams of the governing combinations.

Limitations

- A single prismatic span. Continuous beams, cross-section changes, axial load and minor-axis bending are not covered. Torsion from a load at a varying eccentricity, or a linearly varying torque, must be entered in uniform pieces.
- Open sections need a compact web (F2 and F3 scope); slender-web girders (F4, F5) are refused with a message. Single angles, tees and Z-shapes are out of scope (the DG9 methods for them differ).
- The stress on an HSS in torsion is taken as T / C, and the section is assumed not to warp. Slotted or open-ended closed sections are not closed sections.
- The end connections must provide the restraint you assume: their design is not part of this calculation. Warping restraint at a real connection lies between pinned and fixed; bracket it by running both.
- The analysis is first order. The DG9 4.7.3 factor is the only second-order allowance, and Fe of Eq. F2-4 is the only stability value it uses.
- Local effects at the load point (web crippling, bracket design) are not checked.
- Shear buckling under torsion: DG9 4.7.1 notes that no such limit state has been defined; none is checked.

Verification examples

Verification status: 122 values compared in 8 examples (8 published, 0 hand calculations from the cited equations): 102 within tolerance, 20 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

How the torsion was verified. The published torsion values of DG9 Chapter 5 and of AISC Design Examples H.6 are **chart readings**: the guide reads the torsional functions from its Appendix B graphs to two digits. A comparison with them is a weak check, so:

- the solver is verified independently in the test suite against closed-form solutions of the differential equation (pinned ends with a concentrated and a uniform torque, fixed ends with both, a cantilever with an end torque, a very long member, the closed-section case), and against a separate finite-element integration by energy for every combination of pinned, fixed and free ends under mixed loads, to better than 0.01 %. These are hand calculations and a numerical integration, not published values;
- the published chart-based values are compared with a stated tolerance of **3 %** instead of the usual 1 %. Where the reading is further off than that, the value is listed as a deviation with the exact function next to the reading.

DG9 Chapter 5 (printed pages 19 to 29). Loads entered as D and L giving the printed factored and service loads, no self-weight:

Example	What it checks	Published	ILX Calcs
5.1, W10X49, 15 ft, torsionally pinned, 90 kip-in. at midspan	σb, σw, f_un at midspan (ksi)	12.4, 28.0, 40.4	12.36, 28.53, 40.89
	τt flange, τt web, τw flange at the support (ksi)	10.2, 6.16, 0.56	10.06, 6.11, 0.57
	f_uv flange and web at the support (ksi)	11.4, 8.61	11.26, 8.53
	Rotation, service loads (rad)	0.062 (chart 0.09)	0.0663 (exact 0.0955), deviation
5.2, HSS10X6X1/2 (TS in the guide)	Torsional shear stress (ksi)	1.72	0.924, deviation (below)

Example	What it checks	Published	ILX Calcs
	Rotation, service loads (rad)	0.0013 (J = 187 in.⁴)	0.00137 (J = 176 in.⁴), deviation
5.4, welded plate girder, 25 ft, two torques	J, Cw, a, Wno, Sw1, Qf, Qw	107, 564,000, 117, 153, 1,380, 289, 740	106.7, 562,600, 116.9, 153, 1,377, 289, 740
	σb, σw, f_un at D (ksi)	26.6, 18.4, 45.0	26.61, 18.68, 45.28
	τt flange and web, τw flange at E (ksi)	8.75, 4.37, 0.87	8.78, 4.39, 0.85
	f_uv web at E (ksi)	16.5	16.55
	τt flange at D (ksi)	4.47 (chart −0.12, −0.10)	4.74 (exact −0.128, −0.106), deviation
	Rotation, service loads (rad)	0.023	0.0242, deviation
5.5, MC18X42.7, 12 ft, fixed ends, 3.6 kip/ft at the centroid	σb at the support and midspan (ksi)	8.41, 4.20	8.43, 4.22
	f_un, point 2 at the support (ksi)	17.9	18.34
	σw0, σw2 at the support (ksi)	20.1, 9.49 (chart 0.46)	21.0, 9.91 (exact 0.481), deviation
	τw at points 1, 2, 3 at the support (ksi)	1.30, 1.01, 0.702	1.31, 1.01, 0.71
	f_uv point 3 at z/l = 0.2 (ksi)	4.35	4.36
	Largest f_uv along the span (ksi)	4.35 (three stations)	4.72 at 1.4 ft, deviation
	Rotation, service loads (rad)	0.012 (chart 0.15)	0.0130 (exact 0.159), deviation

The deviations in plain words:

- **Chart readings (DG9-CHART):** the module uses the exact functions; it is higher or lower than the guide by what the two-digit reading leaves out, up to about 10 % on a stress that is itself a small difference.
- **Example 5.2 torsional shear (DG9-5.2-TAU):** the guide divides the whole 90 kip-in. torque by 2 t Ao, though the same example gives each support half of it when it computes the rotation. The module uses the 45 kip-in. each side actually carries, over C = 48.7 in.³.
- **Example 5.2 properties (DG9-5.2-J):** J = 187 in.⁴ of the 1997 tube against 176 in.⁴ in the current database.
- **Example 5.5 stations (DG9-5.5-STATIONS):** the guide looks at z/l = 0, 0.2 and 0.5; the module looks everywhere and finds a higher shear stress at z/l = 0.12.
- **Example 5.5 flange shear (DG9-5.5-SIGNS):** at points 1 and 2 at z/l = 0.2 the guide gets 3.44 and 3.12 ksi with signs; the module gets 4.00 and 4.10 ksi with absolute values, conservative and below the web value there.

AISC Design Examples v16.0, Chapter H (AISC 360-22), both design methods where the example shows both:

Example	What it checks	Published	ILX Calcs
H.5A, HSS6X4X1/4	φT Tn, Tn / ΩT (kip-in.)	273, 181	272.7, 181.4
H.5B, HSS5.000X0.250, 14 ft	φT Tn, Tn / ΩT (kip-in.)	215, 143	214.7, 142.8

Example	What it checks	Published	ILX Calcs
H.5C, HSS6X4X1/4, 8 ft, 6 in. eccentricity	Tr, Vr LRFD; ASD	66.2 kip-in., 11.0 kips; 44.2, 7.36	66.24, 11.04; 44.16, 7.36
	Eq. H3-6 at Tr = 0.2 Tc, LRFD; ASD	0.333; 0.340	0.335; 0.338
H.6, W10X49 (the beam of DG9 5.1), LRFD	σw, f_un, f_uv flange (ksi, chart based)	28.0, 40.4, 11.4	28.53, 40.89, 11.26
H.6, ASD	σw, f_an, f_av flange (ksi, chart based)	18.7, 26.9, 7.56	19.02, 27.26, 7.51
H.6	Cb; φT Fy; φT 0.6 Fy	1.32; 45.0; 27.0	1.316; 45.0; 27.0
	Fcr for buckling (ksi)	65.8	55.3, deviation

H.6 buckling stress (AISC-H6-FCR): the example multiplies the tabulated Mn at Cb = 1 by Cb = 1.32 without the Mp limit of Eq. F2-2; the module keeps Mn at or below Mp. The result is lower and on the safe side; yielding under normal stress governs in both.

Not compared with a published example, because none was found: the warping check under partial line loads, the amplification of DG9 4.7.3 (checked against the equation by hand), welded flanges noncompact or slender (checked against F3 and Table B4.1b by hand), and free end restraints on an open section with mixed loads (checked against the finite-element integration above).

CHAPTER 35

Concrete-filled HSS column

A rectangular or round HSS, a pipe or a built-up box of four plates, filled with concrete, designed as a composite member to AISC 360-22 Chapter I (LRFD or ASD), with ASCE 7-22 load combinations when you enter loads by type. It classifies the section for local buckling and checks compression, tension, flexure about both axes, shear and the interaction of axial force and flexure. It can also check the load transfer between the steel and the concrete at a connection.

Inputs of the filled column

- **Section:** an HSS or pipe from the AISC Shapes Database v16.0, or a built-up box of width B, depth H and plate thickness t. HSS use the design wall thickness and the corner radii of Manual Part 1 (outside 2t, inside t), so the concrete area and inertia are compatible with the database steel properties. A box has sharp corners and four equal plates; plate extensions past the corners are ignored, as Design Example I.7 does.
- **Steel:** A500 Gr. B or C, A1085 and A53 Gr. B for HSS and pipe; A572 Gr. 50 and A36 for box plates. Section I1.3 limits Fy to 75 ksi.
- **Concrete fill:** f′c from 3 to 10 ksi for normal weight concrete, 3 to 6 ksi for lightweight (I1.3), and the unit weight wc from 90 to 155 pcf, which sets Ec = wc^1.5 √f′c. Stronger materials belong to Appendix 2, which this type does not cover.
- **Lengths:** the member length L (used for B1), and Kx Lx and Ky Ly for the column strength. K is your decision: prefilled 1.0, the value of the direct analysis method (C3).
- **Loads:** service loads by type, with ASCE 7-22 combinations, or required strengths entered directly. Axial force is positive in compression. Moments at the top, bottom and midheight about each axis; Vx is the shear that goes with Mx (in the plane of bending about x) and Vy the one with My. A shear left blank is computed from the moments.
- **Second-order effects:** moments from your second-order analysis, or Appendix 8 amplification. For B1, I1.5 gives EI* = 0.64 EIeff with the direct analysis method and EIeff with the effective length method. B2 is your story value.
- **Interaction:** the H1.1 equations by default (Method 1 of Commentary I5). The other option is the plastic stress distribution for compact sections (Method 2 Simplified, Commentary Eqs. C-I5-1a and b) and Eqs. I5-1a and b for noncompact and slender sections.
- **Load transfer (I6):** whether the external force enters through the steel or through the concrete, and the transfer mechanism, direct bond or direct bearing. For direct bond you can enter the load introduction length (0 takes two times the least transverse dimension, I6.4b). For direct bearing enter the concrete area A1 under the internal plates.

Auto-design varies the shape within the given family (rectangular HSS, round HSS or pipe), lightest first, or the plate thickness of a box, and the steel grade when you unlock it. Concrete, loads and the box's outside dimensions stay as entered. An optional limit caps the depth.

Checks performed

Check	Basis
Material limits, f′c, wc, Fy	I1.3, I2.1b
Steel at least 1 % of the section	I2.2a
Width-to-thickness ratios within the maximum permitted; compact, noncompact or slender	I1.4, Tables I1.1a, I1.1b
Compression: Pno, EIeff, Pe, Pn, never less than the bare steel member	I2.2b, Eqs. I2-2 to I2-4, I2-9 to I2-13; Chapter E
Tension: steel yielding	I2.2c, Eq. I2-14
Flexure about x and y: Mp, or between Mp and My, or Mcr	I3.4b, Eq. I3-5; Manual Tables 6-3, 6-4; Commentary Fig. C-I3.8
Shear in each direction: Vn = 0.6 Av Fy + 0.06 Kc Ac √f′c, with Kc from Mu/(Vu d)	I4.2, Eq. I4-1
Axial force and flexure, per combination	I5; H1.1, H1.2; Commentary Eqs. C-I5-1; Eqs. I5-1, Table I5.1
Longitudinal shear to transfer against direct bond or direct bearing (a recommendation, not counted in the status)	I6.2, I6.3, I6.4; Eqs. I6-1, I6-2, I6-3, I6-5

The shear strength follows the 2022 Specification, which adds the concrete to the steel. Kc is 1 unless the section is compact with a short shear span (10 for rectangular and 9 for round below Mu/(Vu d) = 0.5, linear up to 0.7). A section counts as compact here only when it is compact for compression and for flexure about both axes.

Limitations

- **No internal bars.** The strengths ignore longitudinal reinforcement (Asr = 0), including its contribution to fire resistance. Fire design is not covered.
- **Interaction methods.** The strain compatibility and effective stress-strain methods (I1.2b, I1.2d) are not provided. Eqs. I5-1 are written for one bending axis: with biaxial bending a noncompact or slender section falls back to H1.1 and the steps say so. The full five-point curve of Method 2 is not offered, only Method 2 Simplified, which avoids the unsafe region near point D that Commentary I5 warns about.
- **Noncompact and slender rectangular HSS in flexure.** My and Mcr use the stress blocks of Design Example I.7 with sharp corners. All but a few HSS are compact, so this rarely applies to a tube.
- **Noncompact round sections in flexure.** Neither the Specification nor the Manual gives equations for My of a round filled section. The type applies the I3.4b definition directly (steel linear to Fy at the extreme fibre, concrete linear to at most 0.7 f′c) over the circle. No published example confirms this: review it for such a section.
- **Load transfer.** Concurrent loading of the steel and concrete (I6.2c) and shear connectors (I6.3b) are not calculated, and direct bond is taken over the whole perimeter (the end condition of Design Example I.3). For a force entering on fewer faces, use the perimeter and force of each face yourself. The connection itself is not designed.
- **Construction stage.** The bare tube under loads placed before the concrete cures, and the fluid pressure of the concrete, are noted but not calculated (I1).
- **Not covered:** W-shapes and other encased members, Appendix 2 high-strength materials, seismic provisions (AISC 341), and the net section of a tension connection.

Verification examples

Verification status: 151 values compared in 36 examples (36 published, 0 hand calculations from the cited equations): 148 within tolerance, 3 recorded deviations. Code basis: the provisions it uses were read in the target edition.

The tests compare the type with these published values, each read from the source at the page cited.

Source	Case	Published	ILX Calcs
AISC Design Examples v16.0, I.3, pp. I-38 to I-44	HSS10×6×3/8: force to transfer, steel / concrete, LRFD	49.7 / 123 kips	49.6 / 123 kips
same	Direct bond and direct bearing (A1 = 25.1 in.²), LRFD	7.19 / 138 kips	7.19 / 139 kips
I.4, pp. I-48 to I-51	Pno, Pe, Pn at 14 ft	729, 769, 490 kips	729, 768, 490 kips
same (also Table 4-B)	Available compression, LRFD / ASD	368 / 245 kips	368 / 245 kips
I.5, p. I-53	Available tension, LRFD / ASD (also Manual Table 5-4)	468 / 311 kips	468 / 311 kips
I.6, pp. I-56 to I-64	H1.1 ratio, LRFD / ASD	1.11 / 0.914	1.11 / 0.914
same	Method 2 Simplified ratio, LRFD / ASD	0.948 / 0.739	0.944 / 0.737
I.6, p. I-66	Shear Vn; available LRFD	195; 176 kips	194; 175 kips
I.7, pp. I-70 to I-75	30 in. box, 1/2 in. plate, noncompact: Pno, Pn	7,910, 7,140 kips	7,918, 7,147 kips
same	Mp, My, Mn	3,270, 3,050, 3,230 kip-ft	3,274, 3,043, 3,228 kip-ft
I.7, pp. I-77 to I-82	1/4 in. plate, slender: Fn, Pno, Pn	18.7 ksi, 4,820, 4,400 kips	18.7 ksi, 4,822, 4,397 kips
same	Mcr; H1.1 ratio, LRFD; Vn	1,350 kip-ft; 0.799; 579 kips	1,345 kip-ft; 0.803; 581 kips
Steel Tube Institute, Filling the Void (2024), pp. 2-5	HSS14×14×3/8, f′c = 6 ksi, 42 ft: Pno, Pe, Pn, φPn	1,837, 969, 831, 623 kips	within 1 %
same, p. 6	Nine square HSS at 42 ft, f′c = 4 and 6 ksi: composite φPn	18 values	all within 1 %
same, p. 6	Bare steel φPn of the same nine	9 values	8 within 1 %; see below
Manual 16th Ed., Table 5-6, p. 5-25	Four round HSS, tensile yielding, LRFD and ASD	8 values	all within 1 %

Two documented differences, both on bare steel strengths and neither affecting these composite results: I.7 states 1,980 and 541 kips for the bare box, without showing the calculation, and the type gets 1,882 and 509 kips. Those figures reproduce with the HSS wall factors of Table E7.1 and a flat width of H − 3t, while the type reads the Specification for a plate box. The Steel Tube Institute table lists 821 kips for the bare HSS20×20×3/8, which fits a 0.375 in. wall, while the article's 0.349 in. wall gives 730 kips by hand and 729 kips here.

What is checked only by hand calculation. No free published worked example was found for a round filled section, for Eqs. I5-1 or for Kc above 1. The tests check these against hand calculations from the cited equations: round Pno, EIeff and Pn; the Table 6-4 plastic moment against an independent strip integration (within 1.5 %, Table 6-4 assumes a thin wall); Eqs. I5-1 and Table I5.1 as printed; and Kc. The Companion to the Manual, Volume 2, Tables 4-A to 4-F (the filled HSS strength tables) was not available, so only the Table 4-B cells that Examples I.4 and I.6 print are compared.

CHAPTER 36

Steel stair

One straight flight on two stringers, with its treads, the landing beams that carry it, a landing post or hanger, and the guard. It follows AISC Design Guide 34, Steel-Framed Stairway Design (2018), with AISC 360-22, the live loads of ASCE 7-22 Chapter 4 and the stair geometry of IBC 2021 Sections 1011, 1014 and 1015 (or OSHA 1910.25 for an industrial stair). The calculation designs members. **It designs no connection:** take the reactions and the guard post base moment to ILX Connection.

Inputs of the steel stair

- **Stair use and code:** sets which code limits and minimum loads are checked: an IBC stairway for 50 occupants or more, under 50 occupants (36 in. wide), a dwelling unit (riser up to 7-3/4 in., tread from 10 in., 40 psf), an OSHA industrial stair (60 psf, 1,000 lb on a tread, angle 30 to 50 degrees), or Other (no limits, you state them). Limits are on by default; a metric value at the code's own conversion (178 mm for 7 in.) passes.
- **Flight:** number of risers (treads = risers − 1), riser, tread, clear width between stringers, connection allowance (added to the treads to reach the stringer supports; half a tread is usual, DG34 3.4.4) and headroom. The span on plan is treads × tread + allowance.
- **Stair loads:** dead and live load per area on plan; each stringer carries half the width. Self-weight is added, multiplied by the slope ratio. The guard dead load on the stringer (20 lb/ft in DG34) and a table of other loads on the stringer (snow on an exterior stair, a partition) are per length on plan. A **point live load on the stringer** does not act with the uniform load: it is placed at midspan for moment and next to a support for shear. Leave it at 0 under the IBC (the concentrated load is for treads); DG34 10.2.3 applies the OSHA 1,000 lb there.
- **Stringer:** a channel (C, MC), W-shape, rectangular or round HSS, pipe, or a plate. The unbraced length Lb is **your call**: 0 means welded pans or plate over most of the web brace it (DG34 4.4); the guide treats bolted grating treads as unbraced, so enter the sloped length. Cb is an input: 1.0 is on the safe side. Deflection limits are span / n on the span on plan (DG34 Table 3-6 prefilled: 360 live, 240 total); 0 turns a check off.
- **Tread:** a bent plate (checkered plate with a nosing and back leg), a flat plate, or by others (pans, grating and precast treads are not checked: the supplier's tables or AISI S100). The plate spans the clear width and its whole width takes the load. Plate Fy: ASTM A786 floor plate has no guaranteed yield; AISC Manual Table 3-18 uses a bending stress of 24 ksi LRFD and 16 ksi ASD, which is the 26.7 ksi prefilled. Verify the actual value. The uniform and the concentrated load are separate cases.
- **Landing beams:** a header (carries the stringer end reactions at the positions you list, up to 8, plus half the landing) and a rear beam (half the landing), with their span, landing depth, loads and unbraced length.
- **Landing post and hanger:** each beam end sits on a post (compression at the effective length you enter), a hanger (tension), or a wall or framing by others. Ae / Ag of the hanger follows from its end connection, which is not designed here; AISC Manual Table 5-2 lists 0.75.
- **Guard and handrail:** post, top rail, handrail and an infill bar, post spacing, heights, and the loads (200 lb in any direction, 50 plf, 50 lb on infill: ASCE 7-22 4.5.1). A rectangular HSS is checked on its weaker axis because the load acts in any direction. The post K is an input (DG34 uses 2).
- **Seismic (optional):** SDS and Fp / Wp, which you get from ASCE 7-22 Chapter 13 or the seismic nonstructural calculation. Ev = 0.2 SDS D enters the combinations.

Auto-design varies the stringer shape within its family (channel, W, rectangular HSS, round HSS or pipe), lightest first, with an optional depth limit. A plate stringer, the beams, post, hanger, guard and tread stay as entered.

Checks performed

Check	Basis
Riser, tread, width, headroom, flight rise, landing depth	IBC 2021 1011.2, 1011.3, 1011.5.2, 1011.6, 1011.8; OSHA 1910.25 through DG34 Table 3-8
Stair angle (OSHA)	30 to 50 degrees, 1910.25(c)(1)
Guard and handrail heights	IBC 2021 1015.3, 1014.2
Live load, tread load, guard load at least the code value	ASCE 7-22 Table 4.3-1, 4.15, 4.5.1; OSHA 1910.25(b)(6)
Stringer flexure and shear	Horizontal plane method (DG34 4.3.2) on beam analysis; AISC 360-22 F2, F6, F7, F8, F11, G2, G3, G4, G5
Stringer deflection	Deflection on plan times the factor of DG34 Eq. 4-1 (uniform x = 4, point x = 3, vertical y = 2)
Seismic: weak-axis flexure and shear, biaxial interaction	AISC 360-22 F6, F7, F11, G, Eq. H1-1b
Tread flexure, shear, deflection	F6 (bent, as DG34 10.2.2), F11 (flat), G2, G3
Header and rear beam flexure, shear, deflection	F2 and the rest as the shape requires, simple span on beam analysis
Landing post	E3, E7
Hanger	D2 yielding and rupture; D1 slenderness as an advisory
Top rail and handrail	F8 or F7, G5 or G4, E3 (compression along the rail, as DG34), span / 120
Guard post	M = H h, V = H, compression with K, h / 60. One load in one direction at a time, so no interaction, as DG34
Infill bar	F11, G3, span / 120

Limitations

- **No connections:** stringer ends, beam ends, hanger ends, tread welds, the guard post base and its effect on the stringer flange (DG34 10.1.8) belong in ILX Connection or by hand.
- Straight flights only. Integrated landings on the stringer, winders, curved stairs and cantilevered treads are out. Enter a landing on the stringer as separate beams.
- Simple-span stringers, as DG34 4.3.3 recommends. Stiff ends between thick walls add frame action and axial force at the connections; that analysis is not here. No axial force is combined with the stringer flexure: on plan, moment comes out the same and shear on the safe side.
- Seismic: the axial force from drift along the stringer (the note in DG34 10.2.3), seismic relative displacement at the connections and the overstrength combinations for concrete anchorage are not computed.
- Vibration is not checked. DG34 3.3.1 sends it to AISC Design Guide 11, and that needs the stair's mass and modes, not a stringer check.

- Treads are one-way between stringers with the whole plate width effective; a concentrated load on a small area of a thin flat plate (local punching, two-way action) is not looked at.
- Snow or other table loads act on the stringer only; add them to the landing loads yourself.
- Plate stringer shear uses kv = 1.2, on the safe side for a solid bar. Neither the guide nor the Specification spells this case out.
- The IBC limits are IBC 2021; OSHA limits and the guide's deflection limits are read through DG34 Tables 3-3, 3-6 and 3-8, not the OSHA text.

Verification examples

Verification status: 107 values compared in 9 examples (9 published, 0 hand calculations from the cited equations): 100 within tolerance, 7 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Guide 34 (2018), on the 2016 Specification, 15th Ed. Manual and ASCE 7-16. Pages are PDF pages of the library copy. The file `tests/test_verify_steel_stair.py` lists all of the values compared.

Example	What it checks	Published	ILX Calcs
4.1(b), p. 39	Horizontal plane method, service loads	V 1.82, M 5.47	1.82, 5.47
10.1.2, pp. 73-75	C12X20.7 stringer, LRFD	wu 0.424, Mu 4.96, φMn 69.1, Vu 2.05, φVn 65.8	0.424, 4.95, 69.1, 2.05, 65.8
10.1.2	ASD and deflection	wa 0.292, Ma 3.41, 46.0, Va 1.41, 43.8; FΔ 1.40; Δ 0.0140, 0.0214 in.	0.292, 3.41, 46.0, 1.41, 43.8; 1.405; 0.0135, 0.0215
10.1.3, pp. 76-77	C10X15.3 header	wu 0.393, Ru 5.90, Mu 13.4, 42.9, 46.7; ASD 0.267, 4.05, 9.17, 28.6, 31.0	0.393, 5.90, 13.3, 42.9, 46.7; 0.266, 4.04, 9.15, 28.6, 31.0
10.1.4, pp. 77-79	C10X15.3 rear beam	Ru 1.80, Mu 4.13; Ra 1.22, Ma 2.81	1.80, 4.13; 1.22, 2.80
10.1.5, p. 79	HSS3X3X3/16 post	60.4 / 40.2; Pu 5.82, Pa 3.98	60.4 / 40.2; 5.90, 4.04 (see below)
10.1.6, pp. 79-80	L3X3X3/16 hanger	35.3, 35.6 / 23.5, 23.7; Pu 1.71, Pa 1.15	same; 1.80, 1.22 (see below)
10.1.7, pp. 80-89	Guard: rail, handrail, post, infill	φMn 1.45, 1.05; post Pu 0.338, Mu 1.12, 9.36; infill 0.0845	1.45, 1.05; 0.338, 1.12, 9.40; 0.0841
10.2.2, pp. 97-99	Bent checkered plate tread	Vu 1.62, Mu 1.21, 10.8, 1.36; ASD 1.02, 0.761, 7.20, 0.909	1.62, 1.21, 10.8, 1.37; 1.02, 0.761, 7.19, 0.910
10.2.3, pp. 100-105	HSS12X2X1/4 stringer, OSHA, seismic	Mu 7.88, Vu 2.14, 75.4, 142; about y 13.3, 16.4; interaction 0.119	7.85, 2.14, 75.4, 142; 13.3, 16.4; 0.125 (see below)

Three differences are recorded, each with the reason:

- **Reactions on the post and hanger.** Figures 10-5 and 10-6 give a dead load of 0.0677 kip/ft, which leaves out the 15.3 lb/ft self-weight the same examples put in their line load. The reactions taken to the post and hanger (Pu 5.82 and 1.71 kips) therefore come out lower than the beam reactions the guide prints a page earlier (5.90 and 1.80 kips). The program carries the self-weight through: 5.90 and 1.80.
- **Lateral moment on a sloped stringer.** Example 10.2.3 takes the seismic lateral moment on the span on plan, 14.7 ft. The total lateral force is that of the plan length but the member spans its sloped length (17.4 ft), so the

program's moment is 1.185 times larger: 0.622 against 0.540 kip-ft. It raises the interaction from 0.119 to 0.125. This is a choice for the mechanics over the example, on the safe side.

- **Small printed slips, no effect:** 1.40 × 0.00962 in. is 0.0135 in., not the printed 0.0140; Eq. F11-2 in the infill check is compared in kip-in. against Mp in kip-ft, so the guide stops at Mp = 0.0939 where F11-2 gives 0.0933 kip-ft. Both are inside the tolerance.

The guide does not check its own Table 10-1 limit on the infill bar: at a 3 ft span, a 50 lb point load deflects the 1/2 in. square bar 0.32 in. against span/120 = 0.30 in. The program does check it and flags it: the point load is on the safe side of the 1 ft² the code allows.

Hand checks where the guide prints no value (`tests/test_steel_stair.py`): channel lateral-torsional buckling with c of Eq. F2-8b, the three ranges of F11, the channel's minor axis, closed forms of the stringer deflection with Eq. 4-1, a flat tread, the seismic sloped-length moment. Not verified against a published example: plate stringer shear, W and round stringers (same library paths as the steel beam and column types), OSHA limits against the OSHA text (read through DG34 only).

CHAPTER 37

Steel lintel

A steel lintel over an opening in a masonry wall: a single angle, two angles back to back, a W, channel or rectangular HSS, or one of those with a plate under it. It carries the wall above, either as the full height or as the 45 degree arching triangle, plus any uniform and point loads you enter. The calculation checks flexure, shear, deflection and bearing on the masonry, and gives torsion as an advisory.

Inputs of the steel lintel

Lintel. Single angle (veneer or cavity wall), double angle (solid wall; a double angle record with the long legs back to back has them vertical), W, channel or rectangular HSS, or a shape with a plate. For a single angle, pick which leg is vertical. The toe of the vertical leg is in compression under gravity load, which is how the checks read it.

Opening and bearing. The clear span and the bearing length N at each end. The span used is clear span + N, the distance between the bearing centres (BIA TN 31B, TEK 17-1D). The minimum bearing length is prefilled at 4 in. (TMS 402, TEK 17-1D; BIA says 3 in.).

Lateral restraint. Your call, not the program's. Continuous means the masonry restrains the lintel along its length, so a single angle is designed about its geometric axis (F10). At the ends only, or at the ends and midspan, an equal-leg angle takes F10.2 with Sc = 0.8 S. An unequal-leg angle without continuous restraint is NOT FEASIBLE here: F10 sends it to principal axis design with H2, which this calculation does not do.

Load eccentricity e. The distance from the shear center to the load. For a single angle under a wythe, take it from the vertical leg's centerline to the centre of the wythe. It drives the horizontal leg check and the torsion advisory.

Wall above and arching. The wall weight per square foot of face and the masonry height above the lintel. Arching is the engineer's assumption, on by default as the usual case: the lintel then carries the wall within the 45 degree triangle on the span. The program checks two of TEK 17-1D's conditions, the height (half the span plus 8 in.) and running bond; it lists the others for you to confirm: end bearing, no control joints beside the lintel, no openings in the triangle, room for the thrust beside the opening, and cured masonry.

Other loads. Uniform loads and point loads, each at a height above the lintel. With arching, a uniform load at or above the apex goes to the masonry beside the opening. One below it is applied in full, or its part within the triangle dispersed at 45 degrees (TEK 17-1D Figure 4 gives both, so you choose). A point load outside the triangle goes to the masonry beside the opening. One inside is dispersed at 2 vertical : 1 horizontal (TMS 402), 60 degrees (BIA) or not at all.

Deflection limits. Clear span / 600 and 0.3 in. under dead plus live load, prefilled from TMS 402 and BIA TN 31B. Set either to 0 to leave it out.

Auto-design varies the shape of the lintel's family, lightest first. The vertical leg, grade and plate are kept.

Checks performed

Check	Basis
Flexure, single angle	AISC 360-22 F10: yielding 1.5 My, lateral-torsional buckling without continuous restraint (F10.2(2)), leg local buckling with the toe in compression (F10.3)
Flexure, double angle	F9.1(c), F9.2(b) with the negative B, F9.4(b) with F10.3
Flexure, W, channel, HSS	F2, F7 through the shared steel library
Flexure, with a plate	The built-up section's elastic strength, top and bottom fiber, the top times the base shape's Mn / My (at most 1). A hand reading on the safe side: no plastic strength is taken
Shear	G3 for angles (kv = 1.2), G2 and G4 for the rest
Deflection	Service D + L against your limits, TMS 402 and BIA TN 31B prefilled
Bearing length and strength	TMS 402 9.1.8, Bn = 0.8 f'm Abr, phi 0.60; ASD 0.33 f'm (TMS 402-13 8.1.5: the 2022 text is not on disk)
Horizontal leg	A strip of the leg in bending, per inch of lintel, F11.1
Plate weld	Shear flow V Q / I against two lines of intermittent E70 fillet weld, J2.4; minimum size Table J2.4
Arching height and bond	NCMA TEK 17-1D
Torsion (advisory)	Rotation under service load and the movement of the top of the section against BIA's 1/16 in.; St. Venant shear stress beside the bending stress for H3.3
Angle size (advisory)	BIA TN 31B: 1/4 in. and a 3-1/2 in. leg under nominal 4 in. brick

Limitations

Gravity load only, on one simple span. Not covered: wind on the wall, the masonry jambs and their thrust, web local yielding and crippling at the bearing, shelf angle anchors, flashing, corrosion protection, control joints, and the lintel under wet masonry during construction.

Torsion is an advisory. It counts St. Venant torsion with the ends held against rotation and no warping; the masonry above often helps and the calculation does not rely on that. For a W or channel with a real eccentricity, run the steel beam with torsion calculation. Bearing stress is taken as uniform over the bearing width, although BIA notes a rotating angle bears harder at its toe.

The flexure of a shape with a plate is a hand reading: AISC 360-22 has no chapter for a shape with a plate on one flange under these limit states, so the program keeps to first yield.

Verification examples

Verification status: 21 values compared in 13 examples (11 published, 2 hand calculations from the cited equations): 21 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No free source prints a worked example of a whole steel lintel: published verification of this type is thin, and this section says what it rests on.

Source	What is compared	Published	ILX
AISC Design Examples v16.0, F.11A	L4X4X1/4, 6 ft, braced at the ends: LRFD and ASD	3.69, 2.46 kip-ft	3.69, 2.46
AISC Design Examples v16.0, F.11B	The same, braced at midspan too	3.98, 2.65 kip-ft	3.98, 2.65
AISC Design Examples v16.0, F.1-1A	W18X50, 35 ft, continuously braced	379, 252 kip-ft	through the steel library
BIA TN 31B Table 1	The table's load read back: at a load where L/600 governs, the deflection is span / 600; where moment governs, the required moment is the printed resisting moment	4 and 2 rows	within tolerance
BIA TN 31B Table 3	Bearing area arithmetic	3,500 lb	3,500 lb

BIA's table is on the 1980 allowable stress method (Fb = 22 ksi). It confirms the load path, deflection and section properties, not the strength. The arching triangle moment W L / 6, the double angle, the built-up section, the horizontal leg, the plate weld and the dispersions are hand calculations from the cited equations, named as such in the test files. F.11B reuses F.11A's leg local buckling value with Sc = 0.8 S at a midspan brace; the program follows the example there.

CHAPTER 38

Floor vibration

An evaluation of steel-framed floors, pedestrian bridges and linear stairs for vibration from people, by AISC Design Guide 11, 2nd Ed. (2016). Pick one evaluation per calculation: walking on a building floor, a pedestrian bridge, a rhythmic activity, sensitive equipment or occupants, or a linear monumental stair. The steel stair calculation leaves vibration to this type on purpose, because Design Guide 34 refers to this Guide.

The Guide gives recommendations, not code limits. Each limit, damping ratio and driving force is your input with the Guide's value prefilled.

Inputs of the floor vibration

Slab and day-to-day loads. Concrete above the deck, deck height, unit weight and f'c set the dynamic modular ratio (1.35 Ec, Section 3.2) and the slab weight. The loads are expected, not design, loads (Section 3.3): 4 psf for mechanical and ceiling; Table 3-1 for live load, 11 psf in a paper office, 6 to 8 in an electronic one, 6 in a residence, 0 in assembly areas and malls.

Beam or joist. A W-shape with its composite moment of inertia computed, an open-web joist (Section 3.5: chord properties, Cr, Eq. 3-7), or It entered. Effective slab width is the spacing up to 0.4 of the span. A panel at a free edge takes Cj = 1.0 with the typical beam's section, as Example 4.3 does; enter It to evaluate an edge member by Section 3.2. The beam panel weight takes your choice of factor: 1.5 where continuous or shear connected with an adjacent span over 0.7 of this one, 1.3 for joist bottom chords extended before the concrete; joists on seats do not take 1.5.

Girder. A W-shape, It entered, or none where the beams bear on walls. Joist seats use Eq. 3-11 and Cg = 1.6; beams into the web use the full composite section and Cg = 1.8. Floor width and length are the distances over which the framing repeats. Enter a joist girder's Ig from Eq. 3-10; it is not computed.

Damping. The sum of Table 4-2 (0.01 structure, ceiling and ductwork, fit-out, partitions) or entered.

Walking. The occupancy sets 0.5 %g or 1.5 %g (Table 4-1), or enter your own; Po is prefilled at 65 lb.

Pedestrian bridge. Span, weight, vertical and lateral stiffness, Po (92 lb), walkers and whether they march, damping (0.01 for bare framing), running as an option with your limit.

Rhythmic activity. Activity (Table 5-2), the participants' weight (less when they use part of the span), the total weight, column shortening for taller buildings, and the limit from Table 5-1.

Sensitive equipment or occupants. How the limit is given, the generic VC list of Table 6-2 or your value, the walking speed of Table 6-1, and the locations of equipment and walker for the mode shape of Eq. 6-2. Section 6.2 occupants use Eq. 6-9.

Linear stair. Stringers by shape and count or It entered, stair weight, rise and run, damping (engineer's judgment; 0.01 for a bare stair, 0.03 to 0.04 with a soffit and frictional treads and guards), walker and observer on the diagonal or 1.0 on the safe side, and whether a rapid descent may be perceptible (4.5 %g rather than 3 %g) and whether groups are to be evaluated.

Auto-design varies the W-shapes of the beam and girder among up to 18 sizes around each, lightest framing first. It does not check strength: run the beam's own calculation on the result.

Checks performed

Check	Basis
Walking acceleration	DG11 Eq. 4-1, Tables 4-1 and 4-2, Figure 4-3; frequency Eq. 3-3, 3-4; weight Eq. 4-2 to 4-6
Beam panel mode alone, when the beam span is under half the girder span	DG11 4.1.2
Bridge walking, running, lateral frequency	DG11 4.2, Eq. 4-1, Eq. 2-16, Table 4-4
Rhythmic acceleration	DG11 Eq. 5-1, 5-2, Tables 5-1, 5-2, Eq. 3-5; largest over the step frequency range at 0.01 Hz
One-third octave velocity	DG11 Eq. 6-3, or 6-9 for occupants
Peak velocity, peak acceleration, narrowband velocity and acceleration, one-third octave acceleration	DG11 Eq. 6-4, 6-5, 6-6, 6-7, 6-8
Stair frequency, vertical and lateral	DG11 Eq. 4-7: over 5 Hz and 2.5 Hz
Stair descents: normal, rapid, group	DG11 Eq. 4-8, Table 4-5
Under 3 Hz (advisory), over 9 Hz (note)	DG11 4.1

Limitations

Closed-form predictions for regular framing. Not covered: finite element analysis (Chapter 7), the transmission of vibration across joists (Section 8.3), trusses by Eq. 3-6, joist girders by Eqs. 3-7 to 3-10 (enter Ig), unequal girders in one bay beyond entering the governing one, high-frequency floors under Section 2.2 other than Chapter 6, stairs other than a linear flight. Strength is not checked. Floors on cold-formed joists have no path here: the Guide is written for steel beams and open-web joists.

Verification examples

Verification status: 88 values compared in 16 examples (16 published, 0 hand calculations from the cited equations): 87 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Against the Guide's own worked examples, 4.1 to 4.6, 5.1, 5.2 and 6.1 to 6.4, intermediate values included. Example 4.1 gives fn 3.99 Hz, W 109,000 lb and 0.49 %g; 4.2 gives 5.37 Hz, 65,700 lb, 0.503 %g, slightly over the limit as printed. 4.5 gives 2.69 %g walking and 12.8 %g running; 4.6 gives 7.02 and 4.73 Hz, 1.44, 2.08 and 6.23 %g; 5.2 gives 40.1 %g. Example 6.1 gives 9,890 mips at midbay and 7,350 mips scaled; 6.4 gives 2,220 and 6,780 mips.

Example 6.3 reads its velocities from Figure 6-15 to two digits rather than computing them, so those values carry a wider tolerance, and one is a recorded deviation. The peak velocity, narrowband and one-third octave acceleration equations appear in no example; they are checked by hand against the printed equations in the test file.

The AISC Design Guide 11 copy in the library has an errata page at its end that is for Design Guide 9, not 11; its Examples 5.1 and 5.4 are that guide's. No errata list for Design Guide 11 itself was found, apart from the Rev. 9/19 marks in the text.

CHAPTER 39

Beam with web openings

Rectangular or circular openings through the web of a W-shape, steel or composite, unreinforced or with bars along the opening edges, by AISC Design Guide 2 (Darwin, 1990, revised 2003), Chapter 3. The result is the moment and shear strength at each opening, their interaction R, the proportioning rules of the guide, the bar welds, and the range of the span where the opening may sit. Use it for duct and pipe penetrations. The beam away from its openings is a separate calculation (steel beam or composite beam).

Inputs of the beam with web openings

- **Beam:** a W-shape and its steel, Fy up to 65 ksi. **Construction:** steel or composite. On a composite beam the composite expressions apply in positive moment; the steel expressions and the steel resistance factor apply in negative moment (section 3.3).
- **Openings:** up to 6. Each has its centerline position, length ao, depth ho, and eccentricity e (positive upward, as the guide takes it on composite beams; on steel beams it enters as a size). A circular opening uses ho as its diameter Do, with the equivalent sizes of Eqs. 3-29 and 3-30. **N** and **No** count studs on a composite beam: N completely between the high moment end of the opening and the support, No completely over the opening. A stud on the edge of the opening is not counted (section 3.6).
- **Required strengths:** from the beam and its loads (each ASCE 7-22 combination, at the opening's centerline), or entered at each opening. Under ASD, the guide's section 3.8 multiplies the required strengths by 1.7 and uses the expressions with φ = 1.00. The guide names dead and live load; other load types take the same 1.7 here, which is our reading.
- **Reinforcement:** openings marked Bars take the bar entered, on one or both sides of the web. Ar counts both bars when on both sides. The bars are taken within the tee, against the opening edge, as Example 2 places them, and at least as strong as the beam.
- **Slab and studs:** solid slab, or deck with ribs across or along the beam; thicknesses, effective width, f'c. **Qn is your value**, with the rib reduction, by AISC 360-22 I8.2a. The guide's examples use the 1986 rules (no Rg and Rp as today), and today's rules usually give less per stud. The slab reinforcement ratio and studs per foot near the opening are entered and checked against the guide's values (section 3.7c). An unshored beam is checked as a steel beam under the construction loads entered.
- **Stability (steel):** the K of the tee as a column, which the guide does not state: 1.0 is prefilled and it is your value. If the compression flange is not braced throughout, enter Lb and Cb.

Auto-design varies the W-shape, lightest first, with optional depth limits, and the steel when unlocked. The openings, their bars, the loads and the studs stay as you entered them.

Checks performed

Check	Basis
Moment and shear at each opening	R = [(Mu / φMm)³ + (Vu / φVm)³]^(1/3) at most 1, Eq. 3-3; φ = 0.90 steel, 0.85 composite; ASD φ = 1.00 on 1.7 × the required strengths

Check	Basis
Mm, steel	Eqs. 3-6 to 3-8
Mm, composite	Eq. 3-9 where the unperforated section's plastic neutral axis is at or above the top of the flange, Eq. 3-10 where Pc is at least Pc,min, else plastic equilibrium of the net section (section 5.5); never above Mpc
Vm	Bottom and top tees, Eqs. 3-12 to 3-14; composite top tee Eqs. 3-15 to 3-21; limit ⅔ V̄p (+ V̄c) or 0.45 V̄p (section 3.7a2)
Flange and web	Eqs. 3-23, 3-25
Opening proportions	ao/ho, po (Eq. 3-24), ho/d, tee depths, tee aspect ratio (sections 3.7a2, 3.7b1)
Corner radii	2 tw or 5/8 in. (section 3.7b2)
Near a support and near point loads	Edge at least d from a support; no point load over an opening; nearest point load d/2 or d from the edge, an advisory, since bearing stiffeners also satisfy it (Eqs. 3-27, 3-28)
Bars	Width to thickness (Eq. 3-22), extension, welds within the opening and on each extension (Eqs. 3-31, 3-32, AISC 360-22 J2.4), bars on one side (Eqs. 3-33 to 3-36)
Spacing of openings	Eqs. 3-37, 3-38, composite 3-39
Tee as a column (steel)	Where ν is above 4, or where bars are used and the moment is above 90 % of φMm. AISC 360-22 E3, about both axes
Lateral buckling (steel, not braced)	AISC 360-22 F2 with J reduced by Eq. 3-26
Composite	Slab reinforcement 0.0025, 2 studs per foot, construction loads (section 3.7c)

The key results also give the range of the span where each opening may sit: R at most 1 and the edge at least d from a support, for that opening's size and its studs as entered. On a composite beam, N changes along the span, so move the opening and re-enter N to confirm the ends of the range.

Limitations

- W-shapes only: the guide is for compact rolled sections. Its web slenderness rules are applied as written.
- The guide's load and resistance basis is LRFD of 1986; its factors and equations are applied as written, with ASCE 7-22 combinations. Its compact flange and web limits (65/√Fy, 420/√Fy, 520/√Fy) are in ksi as printed.
- **The tee column force** (the moment over the distance between the tee centroids) **and the condition "high moment" for reinforced openings** (above 90 % of φMm) **are our reading**: the guide says the tee must be checked and "only for large openings in regions of high moment", without a number. The stem's local slenderness is noted, not applied.
- Bars on a composite beam's reinforced top tee follow Eqs. 3-14 and 3-20 as written. Bar shear rupture is not computed; the guide says the limit on Pr covers the steel section, and Example 2 checks the bar.
- Deflection with openings, fatigue (section 5.7 advises against openings under significant fatigue loading), and the beam away from the openings are not covered. Bars welded to one side need the bracing note of section 3.7a4.
- Concentrated loads over an opening are not allowed: the expressions assume constant shear through it.

Verification examples

Verification status: 82 values compared in 13 examples (13 published, 0 hand calculations from the cited equations): 78 within tolerance, 4 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 2, Examples 1, 1A, 2, 3 and 4, all parts. The tables and values are on the Verification page, each with its page. Four values are recorded as deviations, each traced to the guide's older shape table, its rounding or its own slips, as below.

- **The guide's load factors match today's** (1.2 D + 1.6 L; ASD 1.7 on D + L), so every required strength compares directly.
- **Shapes:** the guide uses the 1986 tables, this calculation the 16th Edition (W24X55 d = 23.57 in. against 23.6 in.).
- **Slips in the guide, documented on the Verification page:** Example 4's μ of the reinforced top tee is 8.35 on p. 34 and 9.05 on p. 35. The first follows from its own numbers; both give the same Vmt, limited by Eq. 3-21. Example 3 takes Qn as 21.0 and later 21.3 kips.
- **R by the guide is read from a chart** to two digits; this calculation computes it.
- **No published example:** circular openings, the section 5.5 equilibrium, the tee column, lateral buckling with reduced J and the spacing rules are by hand from the equations (`tests/test_beam_web_opening.py`).

CHAPTER 40

Crane runway beam

A simple-span W-shape, plain or with a cap channel, under the wheels of a top-running bridge crane, by AISC Design Guide 7, 3rd Edition (2019), Chapters 11, 12 and 14, with AISC 360-22 and the crane loads of ASCE 7-22 Section 4.9. The rail and its fastening are ILX Connection's crane rail fastening, which takes the forces found here.

Inputs of the crane runway beam

- **Runway beam:** the W-shape, steel, simple span, rail and electrification load, unbraced length and Cb. The guide advises simple spans. It notes that a wheel on the top flange lowers lateral-torsional buckling strength; its examples use Cb = 1.0 at the full span.
- **Crane:** service class, rated capacity, bridge and trolley weights, the maximum wheel load from the manufacturer (0 takes it from the weights), and the wheels on this rail by distance from the first. Two cranes list the wheels of both at their closest.
- **Impact:** by ASCE 7-22 4.9.3 from the service class (25 % for D to F, 10 % for A to C), or entered. **The guide's Example 14.1.1 uses 25 % for a class B cab-operated crane, which is ASCE 7-16's rule.** ASCE 7-22 gives 10 %, so a new calculation may read lower than the guide.
- **Lateral and longitudinal forces:** 20 % of capacity plus trolley, over all the wheels of the crane; 10 % of the maximum wheel load (reported for the tieback, and not combined, as the guide's procedure). The lateral force is taken equally by both rails and all wheels, and acts at the top flange. ASCE 7-22 asks for "due regard" to lateral stiffness: where the rails differ, enter accordingly.
- **Cap channel:** the channel, steel, and the continuous fillet each side. With a weaker cap, lateral-torsional buckling and the weak axis use its Fy (14.1 step 5).
- **Code options, all yours to change:**
  - **Weak axis strength:** the guide defines Mny on the top flange area; Example 14.1.1 reads the whole section's from a Manual table. The top flange is prefilled, the guide's definition and the lower of the two.
  - **Cr in web sidesway buckling:** AISC 360-22 J10.4 defines My about the axis of bending, Fy Sx. The guide's examples use Fy Sy, which gives half the strength. The specification is prefilled.
  - **Deflection limits:** the guide's span/600, 800, 1,000 by class and span/400 laterally, as entries.
  - **Fatigue:** nSR by class (Table 11-1) or entered, and the tension flange's stress category. One crane passage can be several cycles (11.1): your value.

Auto-design varies the W-shape, lightest first, with optional depth limits, and the steel when unlocked. The cap channel, crane and span stay as you entered them.

Checks performed

Check	Basis
Wheel position	Largest moment under a wheel: every wheel and run of wheels on the span, midspan midway between the wheel and the resultant (14.1 step 2)

Check	Basis
Tension flange, with impact	Eq. 14-1; Mn by F2, or Fy S1 with a cap (Eq. F5-10)
Compression flange, no impact, with Mry	Eq. 14-1; Mn by F2, or F5-1, F5-2 with a cap; Mny as selected, or Fy Zyt with a cap
Shear, with impact	AISC 360-22 G2
Web sidesway buckling, under one wheel with impact	J10.4; does not apply past the slenderness limit
Cap channel weld	q = V Q / I, half each side, against J2.4; minimum size Table J2.4
Vertical and lateral deflection	No impact; lateral on the top flange (half the Iy) or on the top flange and channel
Fatigue, tension flange	Appendix 3 Eq. A-3-1 on the wheel moment without impact (AIST TR-13 3.10.2.3, through the guide); peak stress 0.66 Fy; none at or below 20,000 cycles
Fatigue, cap weld	Eq. A-3-2, category F, shear on the throat
Cap on class E or F	An advisory: the guide says not to use them

Limitations

- Simple spans and W-shapes only; plate girders and continuous runways are not covered.
- The rail and its fastening, tiebacks, stiffeners and stops are not designed. Web local yielding and crippling under a wheel (J10.2, J10.3) are not checked. The guide's procedure does not list them either.
- Dead load moment and wheel moment add at their own maxima, as the guide does: slightly on the safe side.
- The AIST TR-13 load combinations and its increased lateral load at the top of the rail are not included. The guide offers them where the owner requires them; TR-13 itself is not on our shelf.
- Fatigue covers the tension flange and the cap weld. Compressive fluctuation is not counted, and the guide's warnings on web-to-flange and stiffener details belong to plate girders.
- J10.4 is written for one concentrated force. The guide says the criterion does not address several wheels, and knows of no problems from this use.
- Wheels are equal, and the same lateral force is used on each.

Verification examples

Verification status: 62 values compared in 5 examples (4 published, 1 hand calculations from the cited equations): 54 within tolerance, 8 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 7, 3rd Edition, Examples 14.1.1 to 14.1.4, on the Verification page with their pages. The examples run with the guide's own choices, as inputs: 25 % impact, whole-section Mny in 14.1.1, My = Fy Sy, and 50 ksi on the cap channel. Example 14.1.3 names an A36 cap and uses 50 ksi on it, against its own step 5.

Where we differ, documented on the Verification page:

- **Lr with a cap channel.** AISC 360-22 F5-5 gives 407 in. The guide's Table A-2 prints 457 in., which is Eq. F4-8, and puts it in Eq. F5-3. This calculation follows F5 and comes out 5 % lower on the lateral-torsional buckling moment, on the safe side. It reproduces the table's 457 in. by F4-8 as a separate value.
- **Rounding.** Example 14.1.1 rounds the slenderness ratio to 1.28 before cubing it, and Example 14.1.3 rounds 3,470 / 1.67 to 2,100 kip-in.: about 1 %.

- **Weld shear in Example 14.1.3.** It adds 75.0 and 2.74 kips where it found 76.2 kips: the weld demand stays within 1 %, as tested.

No published example: two cranes, fatigue, the weaker cap channel, by hand (`tests/test_crane_runway.py`). A worked fatigue example is in Fisher and Van de Pas (2002), which is not on our shelf.

CHAPTER 41

Castellated or cellular beam

A simply supported W-shape cut and rejoined with hexagonal (castellated) or circular (cellular) openings, noncomposite or composite, symmetric or from two root beams, by AISC Design Guide 31 (2017), Chapter 3, with AISC 360-22. The result is the tee forces at every opening for every load combination, the Vierendeel interaction, web post buckling, horizontal and vertical shear, and deflection. The guide's method is written to the 2016 specification; the equations it uses from Chapters D to H are the same in 2022 for these checks.

This chapter departs from the guide's printed examples on one safety matter: read Verification examples below.

Inputs of the castellated or cellular beam

- **Root beam:** the W-shape that is cut. Asymmetric sections take a second root beam below (2.2.6); the spacing is common to both. The steel is the same in both.
- **Castellated pattern:** post width e, horizontal cut b, tee depth dt. The rest follows: h = d - 2 dt, dg = ho + 2 dt, S = 2e + 2b, cut angle = atan(h / b). The web post equations apply only for angles from 43° to 62°, e/tw from 10 to 30 and 2h/e up to 8. Outside these limits the calculation stops with NOT FEASIBLE.
- **Cellular pattern:** diameter Do and spacing S. The web post equations apply only for S/Do from 1.08 to 1.5 and dg/Do from 1.25 to 1.75.
- **First opening:** the distance to its centerline; 0 centers the pattern on the span.
- **Loads:** line loads over the whole span and up to 6 point loads on the top flange, by load type, with the ASCE 7-22 combinations. Self-weight is the average of the root beams. Gravity loads only.
- **Bracing:** noncomposite: Lb, 0 for a flange braced by the deck, and Cb. On a composite beam the deck is taken to brace the top flange (3.6).
- **Composite:** beam spacing and position for the effective width (Eqs. 3-4 to 3-6), f'c, unit weight, rib height hr, concrete above the ribs tc, the studs over the span Ns and **Qn, your value** with the rib reduction by AISC 360-22 I8.2a. The studs are spread evenly, as the guide's q = Ns Qn / span. **Dead load on the steel alone** splits D for the deflections only.
- **Tee torsional buckling:** with the Cw term omitted (the E4 User Note, lower) or with the tee's own Cw (Eq. E4-7 as written). Which you use is your decision; the first is prefilled.
- **Web post curve (castellated):** the guide's text interpolates Mocr/Mp by e/tw. Its examples use the e/tw = 10 curve, which is lower. The text is prefilled.
- **Deflection limits:** yours, span / n; 0 skips the check.

Auto-design varies the root beam, lightest first, with optional depth limits, and scales the opening pattern to its depth; the steel when unlocked. A second root beam, the loads, the span and the studs stay as you entered them.

Checks performed

Check	Basis
Tee force	Pr = Mr / deffec, Eq. 3-1; composite: Eqs. 3-8 to 3-13, the stress block depth iterated at each opening, and To and T1-new where the studs up to that opening fall short
Vierendeel moment	Eqs. 3-2, 3-3; composite Eqs. 3-16 to 3-18, the shear net of the deck's Vc, Eqs. 3-14, 3-15; shared by tee area
Tee compression	E3, E4, E7 on L = e or Do/2, Kx = 0.65, Ky = 1.0 (3.2.2.1), with the true shear center
Bottom tee	Noncomposite: treated as compression, as the guide allows (3.2.2). Composite: tension, D2
Tee flexure	F9 on Lb = e or Do/2: stem in compression, and flange in compression for the reversal over the post
Interaction	H1-1a, H1-1b at every opening and midspan, for every combination
Concrete	Stress block within tc
Web post	Castellated Eqs. 3-19 to 3-29, with φ and Ω by the cut angle (Fig. 3-8), top and bottom separately; cellular Eqs. 3-30 to 3-37. Between 47° and 58° the two equation sets are interpolated on the angle, our reading of 3.4
Horizontal shear	J4.2 on e tw, the thinner web
Vertical shear	G2 at the net section, per tee, kv = 1.2; at the gross section, kv = 5.34, on the thinner web
Whole beam	Noncomposite: Fy Zx-net, and F2 to F5 on the gross section with Lb and Cb (3.6)
Point loads	J10.1, J10.2, J10.3, and J10.4 taking the flange as not restrained against rotation, on the top root beam (3.8). An advisory where a load lies over an opening
Deflection	0.90 × Ix-net, transformed when composite (3.7); composite: live and dead after set against the limits, as the examples; the dead load on the steel alone is reported, for camber

Limitations of this calculation

Simple spans and gravity loads only. Not covered: cantilevers, continuous spans, uplift; tapered or curved beams; infills and large copes (2.3); fatigue and dynamic load, where the re-entrant corners of castellated beams are a concern (3.1); fire (2.4.4); vibration (the floor vibration calculation).

The **pre-composite stage is not checked**: run a noncomposite calculation on the wet load and construction load, with the unbraced length then in force. The first web post at each end is not checked: the guide's examples assume the connection keeps it from buckling. Erection bracing is usually set by the manufacturer (2.4.3). The guide's opening spacing has no match with the stud spacing or the deck: the studs are spread evenly. J10 assumes a continuous web at the load: over an opening, design the infill.

Verification examples

Verification status: 263 values compared in 11 examples (11 published, 0 hand calculations from the cited equations): 228 within tolerance, 35 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 31 Examples 4.1 to 4.4, LRFD and ASD: section properties, forces at the openings, tee strengths, interaction, web posts, shear, deflection, and Table 4-19 for partial composite action. Run on the examples' e/tw = 10 web post curve.

Tee compression: not matched on purpose (DG31-FTB). Examples 4.1 and 4.2 say the Cw term is omitted, yet their printed Fez still has π^2E/L^2 in it, as if Cw were 1 in.^{6}, and their top tees measure yo from the stem tip, not from the shear center at the flange. Example 4.3's bottom tee does it correctly.

Example	Printed Fez	Module Fez, Cw omitted	Printed available Pc	Module available Pc	Printed largest ratio	Module, Cw omitted	Module, the tee's own Cw
4.1 LRFD	3,440 ksi	89.5 ksi	64.9 kips	51.6 kips	0.741	0.936	0.823
4.1 ASD	3,440 ksi	89.5 ksi	43.2 kips	34.3 kips	0.815	**1.03**	0.904
4.2 LRFD	678 ksi	76.5 ksi	65.6 kips	51.7 kips	0.759	0.975	0.911
4.2 ASD	678 ksi	76.5 ksi	43.7 kips	34.4 kips	0.835	**1.07**	1.00

The printed strengths are about 26 % high, on the unsafe side. **Under ASD, both beams the guide shows as adequate fail on tee compression here** with the Cw term omitted; under LRFD they pass. With the tee's own Cw, Example 4.1 passes either way and Example 4.2 under ASD sits at 1.00. **There is no option to use the printed values.** It is recorded for the owner to raise with AISC; check for guide errata before relying on this reading.

Example 4.1's total deflection also reads as a small failure here: its own 2.68 in. is above span / 180 = 2.67 in., which it rounds to o.k.

Other recorded differences: H in Example 4.1 (DG31-H41); cellular web post ratio, about 6 % lower here, unrounded (DG31-ROUND); a printed horizontal shear in Example 4.4 (DG31-44-ASD); Example 4.4's Ix-net and composite Ix, 2,078 and 5,518 in.^{4} here against 1,710 and 5,100 printed, from the net section's own values (DG31-44-I); and the gross shear in its summary (DG31-44-VG).

By hand only, no published example: J10 point loads, the whole-beam LTB with Lb, angles between 47° and 58°, and the tee's own Cw.

CHAPTER 42

Guard or barrier post

A steel post cantilevered from its base, under the code loads for a handrail, a guard or a passenger vehicle barrier, by ASCE 7-22 4.5.1 and 4.5.3, AISC 360-22 Chapters E to H and AISC Design Guide 34 (2018). The result is the post in flexure, shear and compression, its deflection, and its base: a plate on cast-in rods, or a post on a channel stringer's flange. Use the steel stair for the rail and the infill of a stair guard.

Inputs of the handrail, guard or barrier post

- **System:** handrail or guard, or vehicle barrier for passenger vehicles. Truck and bus garages go to the AASHTO bridge specification (4.5.3): not here.
- **Post:** round or rectangular HSS, pipe, or W-shape. Height from the base to the top rail; the length of rail this post carries, usually the post spacing; K in compression, 2.0 prefilled, as Example 10.1.7, because a continuous rail may brace it and that is your decision.
- **Guard loads:** 200 lb at the top, and 50 lb/ft along the rail, not together. The uniform load has three exceptions in 4.5.1.1: clear its box where one applies. Both are prefilled and may be raised; less than the code's is a failed check.
- **Barrier loads:** 6,000 lb, at a height from 18 to 27 in. above the floor, 27 in. prefilled because it gives the largest moment. Less, or outside the range, is a failed check.
- **Downward load together with the horizontal:** a single load in any direction cannot be both, but Example 10.1.7 keeps them together. It is prefilled, on the conservative side, and is yours to clear.
- **Dead load:** rail and infill, and the post's weight. It only adds axial load here.
- **Deflection limit:** height / 60, the guide's Table 3-6 (twice the height over 120, from the IBC's limit for flexible finishes), under the load without a factor. Enter 0 to skip, as usual for a barrier.
- **Base, plate on rods:** the plate and its steel, the weld all around, rods by count, spacing, diameter and grade, the embedment, f'c, the concrete thickness, whether it is cracked, and the distance to a near edge in each direction. Post-installed anchors are not modelled: the largest rod forces are reported to take to that calculation.
- **Base, channel stringer:** the channel and its steel, for Eq. 7-1.

Auto-design varies the post within its family (round or rectangular HSS, pipe, W-shape), lightest first. The plate, rods, loads and height are kept.

Checks performed

Check	Basis
Code loads	ASCE 7-22 4.5.1, 4.5.1.1, 4.5.3
Post flexure	F2, F6, F7 or F8, about each axis because the load acts in any direction, Lb = the height, Cb = 1
Post shear	G2, G4, G5, G6

Check	Basis
Post compression	E3, E7 at K × the height
Axial load and flexure	H1-1, about the weaker axis
Deflection	V a² (3h − a) / (6 E I), the lower I
Rods	ACI 318-19 17.6, 17.7, 17.8, in each direction. LRFD whichever method you use for the post, with 1.6 on the load, because Chapter 17 is strength design. The dead load is left out as it holds the rods down, and the rods share the moment elastically about the group's center, without bearing under the plate
Plate	F11 at the post's face on the tension-side rods; the width is the plate's or the post's plus a 45° spread each side, **our reading**: no guide states it for this detail
Weld	J2.4, as a line around the post, no directional increase; Table J2.4 minimum
Post on a channel flange	The guide's Eq. 7-1, Beff = N + 2(2.5)(k − tf/2 + bf), web as a rectangular bar, F11-1

Limitations of this calculation

Not covered: rails and infill; post-installed anchors; posts core-drilled or in grout pockets, or side-mounted; bearing under the plate; a W-shape post's weld; the torsion in a stringer and its end connections; load sharing through the rail; truck and bus barriers. Eq. 7-1 is the guide's author's suggestion from experience and finite element analysis, for a post alone. ASTM E985 was withdrawn when the guide was written; where a jurisdiction asks for a test (E935), this does not replace it.

Verification examples

Verification status: 23 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 23 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 34, Example 10.1.7, the guard post: required strengths and available flexure, shear and compression, LRFD and ASD; and Example 10.1.8, the post on a C12X20.7's flange. Run at Fy = 46 ksi, as the guide's, through the A500 Gr. B select: this library takes 50 ksi for Gr. C, by the 16th Ed. Manual.

By hand, no published example on disk: the barrier, the rods' forces, the plate, the weld and the deflection. The rods' limit states are the shared ACI Chapter 17 library, verified with the concrete anchor. A guard post anchorage example would be worth finding (NAAMM's Pipe Railing Manual, purchase).

CHAPTER 43

Monorail beam

An S- or W-shape hung from two supports, with or without an overhang past the last one, carrying a hoist trolley on its bottom flange. It follows AISC 360-22 F2 and G2, ASCE 7-22 4.9, Tanner (1985) for the overhang's Cb, and AISC Design Guide 7 (2019) 14.1, 17.2.

Bottom flange local bending NOT checked: enter the allowable wheel load from CMAA 74 or the crane manufacturer. Until you do, the status reads NG, a check row and the key results say it is not checked, and the sheet says so too, whatever the other checks show. The bottom flange is the main limit state of a monorail, so a result without it is not a pass.

Bottom flange local bending under the wheels is not computed. The guide requires that check and points to CMAA 74, which we do not have. Enter the allowable wheel load from the trolley or beam manufacturer, or from CMAA 74. Until you do, the calculation carries an advisory saying the check is open.

Inputs of the monorail beam

- **Beam:** the shape, its steel, the layout (overhang or simple span), the back span and the overhang, to the farthest place the trolley goes.
- **Hoist:** rated capacity, trolley and hoist weight, wheels on the flange. LRFD takes 1.2 on the trolley and 1.6 on the lifted load, as Design Guide 7 Example 14.1.2.
- **Impact:** by ASCE 7-22 4.9.3, 25 % for powered monorails, none when hand-geared, or entered.
- **Lateral force:** by ASCE 7-22 4.9.4, 20 % of the capacity and trolley; the clause is written for powered trolleys. Or entered. The longitudinal force, 10 %, is reported for the hangers' bracing and not combined.
- **Cb of the overhang:** Tanner's Table 1, by k = overhang / whole length, with Lb the whole length, for a beam braced against twist **only at the interior support**. Tanner derives it for a section with small warping stiffness, as S-shapes are. Here it goes into F2, which also counts warping; that is our reading, and the paper's own formula is given beside it as a cross-check. If your interior hanger cannot restrain twist, this Cb does not apply: enter your own.
- **Back span:** Lb and Cb are yours.
- **Deflection limit:** span / 450 on the wheel loads, Design Guide 7 17.2. On the overhang the limit is taken on its own length, on the conservative side, because the guide states it for a span.
- **Allowable wheel load on the flange:** see above, on the same load basis as the method you use.

Auto-design varies the shape, lightest first, within S- or W-shapes as given. The flange check stays your entered wheel load, so it does not follow the size and takes no part in the search: confirm it for the shape chosen. With none entered, the shape it proposes still reads NOT checked when you run it.

Checks performed

Check	Basis
Flexure at the exterior support, trolley at the end	F2 with Lb the whole length and Tanner's Cb
Flexure in the back span, trolley at midspan	F2; the wheels taken at one point, on the conservative side
Lateral load	Eq. 14-1 of Design Guide 7, on the bottom flange alone: Mry over Fy bf^2 tf / 4
Shear	G2
Deflection	At midspan and at the end of the overhang, wheel loads, no impact
Bottom flange under a wheel	Your entered value; with none, a row that reads NOT checked and an NG status (Design Guide 7 17.2)

Limitations of this calculation

Not covered: bottom flange local bending by a method; continuous runways, curves and switches; a brace at the end of the overhang; hangers, sway bracing, splices, trolley stops; fatigue, which the guide says hangers are prone to if allowed to sway; the flange wear it asks to inspect. Using a lateral load at the traction surface on the flange alone follows the crane runway's Eq. 14-1 idea; no monorail example on disk shows it, so it is our reading.

CMAA 74 and ANSI MH27.1 are not on disk. When purchased, they would confirm the flange check, the wheel-load factors, and whether a higher impact or lateral force applies.

Verification examples

Verification status: 16 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Tanner (1985), AISC Engineering Journal, Table 1 (all 11 Cb values) and the numerical example: S12X40.8, 7 ft overhang of 20 ft, 5 kips: k = 0.35, Cb = 0.78, M = 36.0 kip-ft, bending stress 9.5 ksi, allowable 11.3 ksi by the 1978 formula. The paper's S12X35 no good and S15X42.9 good also come out. Its formula is older than AISC 360-22, so **the module's F2 strength itself has no published comparison.**

By hand only: the loads, the lateral load, the back span, the reactions and the deflections.

CHAPTER 44

Steel tension member / brace

A rolled shape, HSS, plate or threaded rod in axial tension, with its end connection: yielding on the gross section, rupture on the effective net area with the shear lag factor U computed by the case of AISC 360-22 Table D3.1, staggered holes, block shear and bearing at a bolted end, and the L/r recommendation. To AISC 360-22 Chapter D with B4.3b, J3.7, J3.11 and J4.3. Pin-connected members and eyebars (D5, D6) are in the steel column type, not here.

Inputs of the tension member

- **Member:** a shape of the AISC Shapes Database v16.0 (W, S, M, HP, C, MC, WT, MT, ST, single and double angles, rectangular and round HSS, pipe), a plate (width and thickness) or a threaded rod (diameter). The steel grade (AISC Manual Tables 2-4, 2-5; ASTM F1554 for rods) and the fabricated length L.
- **Loads:** the axial loads by type, tension positive; the ASCE 7-22 combinations are formed, W and E both ways. A combination that puts the member in compression is flagged: design it as a column.
- **End connection:** bolted or welded; whether the force reaches every element of the section (Table D3.1 Case 1); which part is connected (W flanges or web, tee flange or stem, the angle leg); U computed or entered (your value and basis).
- **Bolts:** diameter (standard holes of Table J3.3), the number of bolt lines over the whole connection and the bolts per line, spacing and end distance. The net area takes each hole 1/16 in. wider (B4.3b), either across a straight section or over failure paths you enter with their $\Sigma\, s^2/4g$. Whether hole deformation at service load is a design consideration (J3.11a: usually yes). Block shear: the planes, tension length and holes of one block, the number of blocks, and whether the tension stress is uniform (Ubs = 1) or not (0.5): your reading of the Commentary figures.
- **Welds:** longitudinal only (Case 4), longitudinal and transverse (Case 2), transverse only (Case 3), or for HSS a single slotted concentric gusset (Case 5) or two side gussets (Case 6); the weld lengths l1, l2 (l is their average), the weld size, the gusset thickness and the slot's clearance (1/16 in. in Design Examples D.4, D.5).

Auto-design varies the shape (lightest first within the given family), the plate thickness or the rod diameter. Loads, length, grade and the connection layout are kept.

Checks performed

Check	Basis
Tensile yielding, Fy Ag	360-22 D2(a), Eq. D2-1
Tensile rupture, Fu Ae, Ae = U An	360-22 D2(b), Eq. D2-2, D3-1, B4.3b
Shear lag U by case, with the lower bound for open sections	Table D3.1 Cases 1 to 8, D3
Block shear at the bolted end	J4.3, Eq. J4-5
Bearing and tearout at the member's holes	J3.11a, Eqs. J3-6a to J3-6d
Longitudinal welds at least 4 times the weld size	Table D3.1 note [a]

Check	Basis
Threaded rod: rupture on Fnt = 0.75 Fu, body yielding	J3.7, Table J3.2; D2
L/r at most 300 (a recommendation, shown as advisory; not for rods)	D1 User Note

$\bar{x}$ is taken as the Commentary shows it (Figure C-D3.1): d/2 − Zx/A for a W connected by its flanges, Zy/A by its web, $\bar{x}$ of the channel, $\bar{y}$ of the tee, $\bar{x}$ or $\bar{y}$ of the angle by the connected leg. For a tee connected by its stem the same reasoning gives Zy/A, and Case 7 for a tee takes the parent's depth as twice the tee's (both to confirm against the code text).

Limitations

- The bolts, welds and gusset plate are not designed here (bolt group, weld group and gusset types); neither are bolt spacing and edge distances (J3.3 to J3.6).
- Pins and eyebars (D5, D6), fatigue (Appendix 3) and tension with flexure (Chapter H) are in the steel column type.
- A failure path takes one thickness: a path through elements of different thickness is entered as an equivalent.
- A bolted HSS end has no Table D3.1 case: enter U.
- The 2022 HSS Cases 5 and 6 are read from the specification text, whose layer prints the exponents and the pairing of B and H poorly; Case 5 matches Design Examples D.4 and D.5, Case 6 is a hand check (to confirm against the code text).

Verification examples

Verification status: 48 values compared in 10 examples (7 published, 3 hand calculations from the cited equations): 48 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Examples v16.0 D.1 to D.6 and D.9 (yielding, rupture, U, An, L/r). Block shear, bearing, the threaded rod and Case 6 are hand calculations from the cited equations: the Examples assume the connection does not govern.

CHAPTER 45

Concentrated forces on beams

A W, S, M or HP member under a force normal to its flange: a beam or column bearing on it, a hanger below it, a reaction, or the flange forces of moment connections framing into a column. The limit states of AISC 360-22 J10, with transverse stiffeners and web doubler plates designed for what the member lacks.

Inputs of the concentrated force

- **Loaded member:** the shape, its steel, and the distance from the force to the member's end (near the end J10.1, J10.2, J10.3 and J10.5 reduce the strength).
- **Single force:** P by load type onto the flange (compression positive, a hanger negative) and the member's moment at the force (it sets Cr of J10.4). The length of bearing lb along the member, the loaded width across the flange (J10.1 is not checked under 0.15 bf), the thickness of the flange or plate delivering the force (J10.8(b)). Your statements: an equal compressive force on the other flange at the same place (J10.5); whether the flanges are restrained against relative lateral movement at the force (else J10.4 applies), whether the loaded compression flange is restrained against rotation (Appendix 6), the largest unbraced length along either flange; an unframed end not restrained against rotation (J10.7).
- **Moment connections:** one or two beams, the beam end moments by load type (positive with the beam's top flange in tension), the column's axial force and story shear; the beam depth, flange thickness and width. The flange force is M/(db − tfb); for flange plates enter the beam depth plus two plate thicknesses and the plate's thickness. Whether the analysis includes inelastic panel-zone deformation (J10.6(b)): your statement.
- **Stiffeners and doubler:** a pair of transverse stiffeners (steel, width, thickness, corner clip, full depth or not, fitted to bear), a web doubler plate (thickness, steel).

Auto-design varies the stiffeners (none first, then the width and thickness of a pair) and, for moment connections, the doubler. The member, the forces and your statements are kept.

Checks performed

Check	Basis
Flange local bending (tension)	J10.1, Eq. J10-1
Web local yielding	J10.2, Eqs. J10-2, J10-3
Web local crippling (compression)	J10.3, Eqs. J10-4, J10-5a, J10-5b
Web sidesway buckling	J10.4, Eqs. J10-6, J10-7
Web compression buckling (a pair of compressive forces)	J10.5, Eq. J10-8
Web panel-zone shear, with a doubler	J10.6, Eqs. J10-9 to J10-12; J10.9, G2.1
Full-depth stiffeners at an unframed end	J10.7

Check	Basis
Stiffeners for the difference, in tension or compression; a full-depth bearing stiffener as a column (0.75 h, 25 tw or 12 tw strip)	J10.8, J4.1, J4.4, Chapter E
Bearing of fitted stiffeners	J7, Eq. J7-1
Stiffener width, thickness and length	J10.8(a) to (c)
Doubler slenderness for Cv1 = 1	G2.1(b)

With stiffeners, the member's own limit states at that location are shown as notes (their available strengths), and the stiffeners carry the difference between the required force and the least of them. For a pair of forces at both flanges from two beams, the smaller compression is taken as the pair; the difference goes into panel-zone shear.

Limitations

- The welds of stiffeners and doublers are not designed.
- A doubler serves panel-zone shear only; doublers for local yielding or crippling are not designed.
- HSS members (Chapter K) and cantilever ends (Appendix 6.3) are not covered.
- The required strength of a full-depth bearing stiffener checked as a column is the difference, as J10 says for stiffeners in general (to confirm against the code text for bearing stiffeners).
- The member's own flexure and shear are designed by the beam or column types.

Verification examples

Verification status: 22 values compared in 6 examples (4 published, 2 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Examples v16.0 J.7 (girder under a beam: web yielding, crippling, sidesway buckling) and II.B-1 (column side: flange bending, web yielding, crippling), AISC Design Guide 13 Example 6-1 (panel zone, flange bending; the guide is written to the 1999 Specification and only equations unchanged in 2022 are compared). Web compression buckling, stiffeners and the doubler are hand calculations.

CHAPTER 46

Steel fire resistance

A W-shape beam or column in a fire: its strength at the steel temperature by the simple methods of AISC 360-22 Appendix 4.2.4d (with the critical temperatures of 4.2.4e for reference), and the thickness of its fire protection for a rating by the calculation methods of Appendix 4.3.2, which are the equations of IBC 2021 722.5.

Inputs of the fire design

- **Member:** the shape (W, S, M, HP), its steel (Appendix 4.2.3b covers Fy up to 65 ksi), beam or column.
- **Strength at temperature:** the steel temperature T in °F or °C, your value from a heat transfer analysis of the design-basis fire (4.2.2), taken uniform over the section. The nominal loads D, L, S and the forces AT from the fire (restrained thermal expansion, your entry), combined by Eq. A-4-1 with 1.2D or 0.9D (your choice where the dead load counteracts). The design is LRFD with the ambient resistance factors, as Appendix 4.1.2 requires; the LRFD / ASD switch does not apply. Lb and Cb for flexure (Lb = 0 when the compression flange is braced continuously); K and L about each axis for a column, and whether the column is a gravity column between cooler columns (Eq. A-4-10, your statement: a fire on one floor only).
- **Protection:** the rating R (from the building code), the thickness provided, and the member's W/D, computed from the shape or entered (IBC Tables 722.5.1(1), (4)). Sprayed beam: the W/D and thickness of the beam in the listed assembly, and whether the rating is restrained. Sprayed column: the listed assembly's constants C1, C2 (for R in hours or minutes, as the listing prints them) and the W/D range of its tests. Gypsum board column: the total Type X board thickness.

Auto-design varies the protection thickness, thinnest first: sprayed material by 1/16 in. from 3/8 in., gypsum board by 1/2 or 5/8 in. layers. The member, the temperature, the loads, the rating and the listed assembly are kept.

Checks performed

Check	Basis
Retention factors kE, kp, ky, interpolated	Appendix 4.2.3b, Table A-4.2.1
Load combination (0.9 or 1.2)D + AT + 0.5L + 0.2S	Eq. A-4-1
Compression of a nonslender column, Fn(T) = 0.42^√(Fy(T)/Fe(T)) Fy(T)	4.2.4d(b), Eqs. A-4-9, A-4-10
Flexure of a compact W, lateral-torsional buckling at temperature	4.2.4d(e), Eqs. A-4-13 to A-4-19
Shear yielding	4.2.4d(g), Chapter G
Axial force and flexure	4.2.4d(h), Chapter H1.1
Critical temperatures (notes)	4.2.4e, Eqs. A-4-22, A-4-23
Sprayed beam: substitution thickness, W/D ≥ 0.37, h ≥ 3/8 in., R ≥ 1 h, compact if restrained	4.3.2c, Eq. A-4-32 (IBC Eq. 7-17)
Sprayed column: thickness from the listed constants, within their W/D range	4.3.2a(b), Eq. A-4-26 (IBC Eq. 7-13)

Check	Basis
Gypsum board column: rating of the board, W/D taken as 3.65 above it, at most 4 hours	4.3.2a(a), Eqs. A-4-24, A-4-25 (IBC Eq. 7-12)

At or below 400 °F the ambient strengths are used (4.2.4d). The heated perimeters are those of IBC Figures 722.5.1(1) and 722.5.2: column contour 4bf + 2d − 2tw, box 2(bf + d); beam contour 3bf + 2d − 2tw. Note that the IBC numbers the substitution's beams the other way round from AISC: here "listed" and "this beam" are named.

Limitations

- The steel temperature is your entry: no heat transfer (lumped mass) analysis is made.
- Slender sections (compression) and noncompact sections (flexure) are out of the simple method's scope and stop as not feasible.
- Composite beams (4.2.4d(f)), filled and encased columns, HSS (their A/P correlations), intumescent or mastic coatings and the connections are not covered.
- The reading of Eq. A-4-10 (n in both terms) is from the text layer of the specification (to confirm against the code text).
- The protection's attachment, density and installation follow the listed assembly.

Verification examples

Verification status: 15 values compared in 10 examples (5 published, 5 hand calculations from the cited equations): 15 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 19 (2003) Examples VI-2 (gypsum board), VI-3 (sprayed column), VII-1 and VII-2 (beam substitution): the rating equations are unchanged in 360-22 Appendix 4.3.2 and IBC 2021 722.5. No published example of the 360-22 simple methods at elevated temperature is in the free library: the strength values are hand calculations of Eqs. A-4-9 to A-4-19 with Table A-4.2.1.

CHAPTER 47

Roof ponding

Flat roof framing under rain or snow: the ponding stability and strength analysis that AISC 360-22 B3.10 requires of a roof that can retain water. AISC 360-22 removed the former Appendix 2 and gives no method of its own (its Commentary points to SJI Technical Digest 3); this type applies the methods of AISC 360-16 Appendix 2 and its Commentary, valid for flat roofs of rectangular bays with uniformly spaced secondary members and uniformly loaded primary members.

Inputs of the ponding check

- **Primary members:** a rolled shape, or I and S entered (joist girders, trusses), and the span Lp; or none, for secondary beams bearing on walls (Cp = 0).
- **Secondary members:** a rolled shape or I and S entered (joists), the span Ls and the spacing S; for an entered joist or truss I taken from the chords only, the 15 % reduction for web strain (former Appendix 2.1 User Note). The deck's moment of inertia Id per foot.
- **Loads:** the roof load at the onset of ponding and the rain or snow load without the ponding contribution (nominal; the rain to the secondary drains with the primary drains blocked, ASCE 7-22 Chapter 8, is your entry), and Fy.
- **Method:** improved (stress index) or simplified; either satisfies the former appendix.

Auto-design varies the secondary shape, lightest first within its family; the primary shape too when you unlock it.

Checks performed

Check	Basis
Cp + 0.9 Cs ≤ 0.25 (simplified)	360-16 Eq. A-2-1, A-2-3, A-2-4
Deck Id ≥ 25 $S^4/10^6$	360-16 Eq. A-2-2
Bounded ponding deflections, (π/4) αp αs < 1	360-16 Commentary Eq. C-A-2-4
Primary: Δw/Δo within Up = (0.8 Fy − fo)/fo	360-16 Eq. A-2-5, Commentary Eq. C-A-2-1
Secondary: δw/δo within Us	360-16 Eq. A-2-6, Commentary Eq. C-A-2-2

The improved method's Figures A-2.1 and A-2.2 are drawn from the Commentary's Marino (1966) equations: the type computes the ratios directly instead of reading the charts. fo is the bending stress of each member under the load at the onset of ponding plus the rain or snow, the secondary as a simple span under q S, the primary uniformly loaded by q Ls.

Limitations

- Rectangular flat bays only; roof slope, camber, cantilevers, continuous framing and non-uniform drifts are not modeled (SJI Technical Digest 3 gives fuller methods).

- The members' strength under the full load is designed by their own calculations.
- The last term of Eq. C-A-2-2 is read from a poor text layer (to confirm against the code text).

Verification examples

Verification status: 8 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published ponding example is in the free library: the values are hand calculations of the former Appendix 2 and its Commentary equations. SJI Technical Digest 3 (2018) would add published examples.

CHAPTER 48

Seismic member checks

The member requirements of AISC 341-22 for a W-shape beam or column of a seismic force-resisting system: the width-to-thickness limits of moderately and highly ductile members, the stability bracing of beams, the column's required axial strength with the overstrength seismic load, and strong-column weak-beam at special moment frame joints. The member's own strength under the analysis load effects is designed by the steel beam and column types.

Inputs of the seismic member

- **Member:** beam or column, the W-shape, its steel and Ry (Table A3.2 unless you enter it), the ductility the system's chapter designates, and whether the member is in a moment frame (Table D1.1b Case 11 for webs) or another system (Case 13).
- **Column:** the dead, live and seismic axial forces from your analysis, Ω0 of the system (ASCE 7-22 Table 12.2-1), SDS for Ev, the live load factor of the seismic combination, and K and L about each axis.
- **Strong-column weak-beam (SMF):** the columns at the joint (the column above taken as the same shape), the beams framing in, their steel, Ze at the hinge (ZRBS for a reduced beam section) and Mv per beam from your connection design (AISC 358); an E3.4a exception as your statement.
- **Beam:** its axial force and the spacing of its stability bracing.

Auto-design varies the member's W-shape, lightest first, within 15 % of the given nominal depth. The loads, system, beams and bracing are kept.

Checks performed

Check	Basis
Flange b/t at most 0.30 or 0.38 √(E/(Ry Fy))	341-22 Table D1.1b Case 7
Web h/tw by Ca = αs Pr/(Ry Fy Ag)	Table D1.1b Case 11 (moment frames), Case 13
Column compression and tension with Ω0 QE and Ev, moments neglected	D1.4a(b); ASCE 7-22 2.3.6, 2.4.5; 360-22 E3, D2
ΣM*pc/ΣM*be > 1.0, Mpr = Cpr Ry Fy Ze	E3.4a, Eqs. E3-1 to E3-3; AISC 358 2.4.3
Beam bracing spacing at most 0.17 or 0.086 ry E/(Ry Fy)	D1.2a, D1.2b, Eq. D1-2
Required brace strength and stiffness (note), special bracing at hinges	D1.2a(b), Eq. D1-1; 360-22 App. 6.3.1a; Eq. D1-4
Protected zones (note)	D1.3

Limitations

- W, S, M and HP shapes only; HSS, built-up boxes, composite members, braces and links are not covered.

- The column required strength from the system's own analysis (D1.4a(a)) and intersecting frames are your analysis.
- Connections, panel zones and splices are not designed; Mv is your entry.
- Cpr and the Appendix 6 bracing stiffness are taken as cited without the AISC 358 and 360 text read here (to confirm against the code text).

Verification examples

Verification status: 9 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published worked example of these checks is in the free library (the AISC Seismic Design Manual is not on disk): hand calculations of the cited equations.

CHAPTER 49

Encased composite column

A W-shape encased in a rectangular reinforced concrete section, to AISC 360-22 Chapter I: the detailing limits, axial compression and tension, the axial force and strong-axis moment interaction by the plastic stress distribution (Commentary Method 2 Simplified with AISC Manual Table 6-2a) or Chapter H, shear, and the load transfer between the steel and the concrete. Concrete-filled HSS and box columns are the filled HSS column type.

Inputs of the encased column

- **Section:** the encased W or HP shape and its steel; the encasement h1 (along the flanges) by h2 (along the web); f'c and the concrete unit weight (Ec = wc^1.5 √f'c); four corner bars or eight (three per face), their size, the face-to-center distance and Fysr; the ties and their spacing.
- **Length and loads:** K and L about each axis; the axial force, the strong-axis moment and the shear by load type (ASCE 7-22 combinations), the moments from a second-order analysis (your statement); the interaction method.
- **Load transfer:** where the external force enters (the steel section or the encasement), and steel headed stud anchors (number within the load introduction length, diameter, Fu) or a transfer strength you enter (direct bearing, your design).

Auto-design varies the encasement size (its proportions kept) and the bar size; the steel shape too when you unlock it.

Checks performed

Check	Basis
f'c 3 to 10 ksi, Fy ≤ 75 ksi, Fysr ≤ 80 ksi	360-22 I1.3
Steel core ≥ 1 % of Ag; bars ≥ 0.4 % (≤ 8 %, ACI 318-19 10.6.1.1); ties No. 3 at 12 in. or No. 4 at 16 in., ≤ half the least dimension; bars to the core	I2.1a
Compression Pno, Eleff with C1, Pe, Pn; not less than the bare steel member	I2.1b, Eqs. I2-2 to I2-7; Chapter E
Tension Fy As + Fysr Asr	I2.1c, Eq. I2-8
Axial force and strong-axis flexure, points A, B, C reduced by λ = Pn/Pno	I5; Commentary Eqs. C-I5-1a, b; Manual Table 6-2a
Or Chapter H interaction (Method 1)	H1.1
Shear on the steel section	I4.1(a), G2.1
Load transfer V'r by studs (Qnv = Fu Asa) or entered	I6, Eqs. I6-1, I6-2a; I8.3a

Limitations

- Strong-axis bending only; weak-axis and biaxial bending, and Method 2 with point D, are not built.

- Four or eight bars only: Table 6-2a assumes bars at the faces and at the centerline.
- The neutral axis of point B in the web or below the steel uses the Table 6-2a equations as cited (to confirm against the Manual); Example I.11 exercises the flange case.
- The cage's other ACI 318 details (crossties, splices), AISC 341 D1.4b seismic detailing and the second-order analysis are yours.

Verification examples

Verification status: 28 values compared in 7 examples (7 published, 0 hand calculations from the cited equations): 28 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Examples v16.0 I.8 (load transfer), I.9 (axial compression), I.10 (axial tension) and I.11 (combined force, Method 1 and Method 2 Simplified).

CHAPTER 50

Plate girder

A simply supported welded I-girder from its plates, to AISC 360-22: flexure (F4 or F5 with Rpc or Rpg), shear panel by panel with tension field action in interior panels, the intermediate stiffeners' rigidity, the bearing stiffeners at the supports, the flange-to-web welds, the proportion limits and the deflection.

What it adds over the steel beam. The steel beam's built-up I already has F4 / F5 flexure, G2 shear with tension field action, the G2.4 stiffener rigidity and the F13.2 limits (the same library). The plate girder adds the girder laid out panel by panel from the stiffener positions, each panel against its own required shear (end panels by G2.1), each stiffener's G2.4 rigidity with its own ρw, the bearing stiffeners (J10.8 column and J7 bearing, for what the web lacks or the full reaction), the flange-to-web welds for VQ/I, Cb on the governing unbraced segment and the deflection.

Inputs of the plate girder

- **Girder:** the span, the web (h × tw) and the two flanges, the steel (A572 Gr. 50 or A36 plates).
- **Loads and bracing:** uniform and concentrated loads by type (ASCE 7-22 combinations), the girder's weight as dead load if D does not include it, the brace spacing of the compression flange (0 when braced continuously), Cb computed (Eq. F1-1 on the governing segment) or entered.
- **Intermediate stiffeners:** their positions from each support (mirrored about midspan; none for an unstiffened web), tension field action in interior panels, width, thickness, pairs or single, steel.
- **Bearing stiffeners and welds:** the bearing stiffener pair at each support, its corner clip, the bearing length, and whether it is sized for what the web lacks (J10) or the full reaction (your choice); the flange-to-web fillet size and FEXX; the deflection limit L/n on total or live load (your limit).

Auto-design varies the web and flange thicknesses, lightest first (equal flanges stay equal). Depth, widths, span, loads, stiffeners and bracing are kept.

Checks performed

Check	Basis
Flexure, compression flange yielding, LTB, local buckling, tension flange yielding	360-22 F4 (Rpc) or F5 (Rpg); Cb by F1-1
Proportion limits Iyc/Iy, aw, h/tw	F13.2
Shear of each panel; interior panels with tension field action when a/h ≤ 3	G2.1, G2.2 (Eqs. G2-6 to G2-8)
Intermediate stiffeners: (b/t)st, Ist with ρw	G2.4, Eqs. G2-16 to G2-19
Bearing stiffeners as a column (two plates and a 12 tw strip, Lc = 0.75 h)	J10.8, J4.4
Bearing of the fitted stiffeners	J7, Eq. J7-1
Bearing stiffener thickness and width-to-thickness	J10.8(b), Table B4.1a
Flange-to-web welds for q = V Q/Ix, minimum size, web base metal	J2.4, Table J2.4, J4.2

Check	Basis
Deflection at most L/n	Your limit

Limitations

- Simple spans only; continuous girders, cover plates and holes in the tension flange (F13.1) are not covered.
- G2.3 (tension field action in end panels, new in 2022) is not built: end panels are taken by G2.1, as Design Examples G.8A and G.8B do.
- The web under concentrated loads is the concentrated forces type; stiffener welds, splices, fatigue and the bracing's design are not part of this calculation.
- The deflection is the midspan value of the superposed loads.

Verification examples

Verification status: 16 values compared in 3 examples (2 published, 1 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

AISC Design Examples v16.0 F.14 (flexure, shear, weld shear flow, deflection) and G.8B (end and second panel shear with stiffeners at 42 and 132 in.). The stiffener rigidity and bearing stiffener values are hand calculations.

CHAPTER 51

Web-tapered member

A doubly symmetric welded I-section with a single linear web taper and equal prismatic flanges, the typical column or rafter of a metal building frame, in axial force, major-axis flexure and shear, by AISC Design Guide 25 (2nd Ed., 2021) on the AISC Specification. The required strengths are your entries from a second-order frame analysis.

Inputs of the tapered member

- **Member:** the length, the clear web depth at the small and the large end, the web thickness, the flanges (both the same), the steel.
- **Bracing and loads:** the in-plane Kx over the whole length (1.0 with a direct analysis), the brace points measured from the small end (both flanges braced, as girts or purlins with flange braces do), and the required strengths per case: P (compression positive), the moments at the two ends (varying linearly) and the largest shear.

Auto-design varies the flange and web thicknesses, lightest first. Depths, flange width, length, bracing and required strengths are kept.

Checks performed

Check	Basis
In-plane elastic buckling, I at 0.5 L (Ismall/Ilarge)^0.0732 from the small end	DG25 Eqs. A-3, A-4 (Eq. 4-4)
Out-of-plane buckling per unbraced length, Iy at its middle	DG25 Eq. 5-4
Compression at every section: Fe = γe fr, Fn, effective area of the slender web and flanges, Pn = Fn Ae	DG25 5.3.2; 360-22 E3, E7.1 (Table E7.1)
Tension yielding of the smallest section	360-22 D2
Compression flange yielding (Rpc, Rpg) and flange local buckling at every section	360-22 F4, F5
Lateral-torsional buckling per unbraced length by the stress ratios fr/FeLTB1, Cb and γeLTB1	DG25 Eqs. 5-23 to 5-31
Shear of the unstiffened web	360-22 G2.1
Axial force and flexure with the largest ratios in each unbraced length	360-22 H1.1; DG25 5.5

The flexural checks are made at the ends and quarter points of each unbraced length (the sections where the ratios are largest for a linear taper and a linear moment); the lateral-torsional buckling equations of single curvature (Eqs. 5-25 to 5-27) apply when the moment keeps its sign within the length, the general ones (Eqs. 5-23, 5-24 with γeLTB = Cb γeLTB1) otherwise.

Limitations

- Doubly symmetric sections with one linear taper only: singly symmetric, stepped or multiply tapered members are not covered (DG25 Appendix C).
- Torsional and constrained-axis torsional buckling are not computed: they do not govern when both flanges are braced at the same points.
- Unstiffened webs; tension field action and the inclined-flange term of the web shear (DG25 Eq. 5-54) are not used (conservative here).
- The frame analysis, its second-order effects and stiffness reductions (DG25 Ch. 4, 6) are yours.
- The inelastic lateral-torsional buckling equation is read with π √(Fyc/(γe fr)) and π √(Fyc/FL) from a text layer that drops the symbols; Example 5.2b's 2,520 kip-in. confirms the reading.

Verification examples

Verification status: 17 values compared in 5 examples (5 published, 0 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

AISC Design Guide 25 (2nd Ed.) Example A1 (equivalent moment of inertia and PeL), Example 5.2a (axial compression of a linearly tapered column braced at 90 in.) and Example 5.2b (flexure: Cb, γeLTB1, LTB and FLB strengths of the lower unbraced length).

CHAPTER 52

Sawn lumber beam

A sawn lumber beam, joist or header in bending, shear and bearing, to NDS 2024 Chapters 2 to 4 with ASCE 7-22 load combinations, in ASD or LRFD. One model covers simple spans, cantilevers, overhangs and continuous beams. Every combination is checked with its own load duration factor CD (ASD) or time effect factor λ (LRFD), so a dead load alone (CD = 0.9) can govern.

Inputs of the sawn lumber beam

- **Member use:** a label for the report; the Wizard cards prefill each use. *Floor overhang* and *roof overhang*: a support at 3/4 of the length, the bottom edge with its own unbraced length (the overhang's length). *Header*: a simple span, no repetitive member factor; the jack, king and trimmer studs are in the stud wall. *Ridge beam*: a simple span with the rafter loads per foot, an uplift row (negative W), the bottom edge free over the span, the shear reduction off (rafters hung from the side do not load the opposite face, NDS 3.4.3.1). *Hip* and *valley*: a simple span of the plan length with a triangular load from 0 at the low end to the peak; the peak for jack rafters of equal slope on both sides of a 45 degree plan line is w = q Lh / 2, q the load per horizontal square foot and Lh the plan length of the hip. That triangle is the usual idealization, not a code rule: it is your judgment, as is whether the top end is really supported (a hip carried only by other hips and a ridge board is not). The axial force of the sloped member and deflection normal to the slope are not included. Every card starts from typical loads: replace them.
- **Member:** species (Douglas Fir-Larch, Hem-Fir, Spruce-Pine-Fir, Southern Pine) and grade (Select Structural to Stud), nominal size (2x4 to 4x16 dimension lumber, 6x6 to 12x16 timbers) and the number of plies acting as one member. *Other* takes reference values you enter (including the size factor). The self-weight uses the density entered (35 pcf is the basis of the Supplement section tables).
- **Length and supports:** any number of pins or fixed ends (as for the steel beam) and the bearing length lb at the supports. The bearing area factor Cb applies to bearings more than 3 in from the member end (NDS 3.10.4).
- **Loads:** line loads in plf and point loads in lb, each with its load type (D, L, Lr, S, R, W, E). A negative value acts upward; a line load with a value at To is trapezoidal or triangular.
- **Partial loading:** on by default. On a member with more than one span or an overhang, L, Lr and S also act on part of the spans (ASCE 7-22 4.3.3 and 7.5; an overhang counts as a span, S keeps half its value on the other spans, the dead load stays everywhere). Up to three spans take every subset; more take alternate spans, adjacent pairs, single spans and all but an end span. The combination names say which: "D + L [live on span 1]". ASCE 7-22 7.5.1 lets you turn snow partial loading off for rafters across the ridge of a gable of slope 1/2 to 7 on 12, by the option.
- **Bottom edge under negative moment:** sheathing on top does not brace the bottom edge. Choose its own unbraced length where nothing holds it (an overhang, an interior support, wind uplift on an exposed ridge beam).
- **Lateral stability:** the unbraced length lu of the compression edge (0 = braced continuously, CL = 1.0) and the load case of NDS Table 3.3.3 for the effective length le. The general values are permitted for any single span or cantilever.
- **Service conditions and factors:** wet service (CM), temperature (Ct), incising (Ci), repetitive member (Cr = 1.15 for three or more members at 24 in or less joined by sheathing), flat use (Cfu), the shear reduction near the

supports (3.4.3.1, for a member bearing on one face and loaded on the other) and the bearing deformation limit (0.04 in tabulated, or 0.02 in with 0.73 Fc⊥, NDS 4.2.6).

- **Deflection limits:** span / n for live and total load (IBC Table 1604.3 values prefilled, 0 turns a check off). The total deflection includes creep when checked: ΔT = Kcr ΔLT + ΔST (NDS 3.5.2) with the long-term load D plus the sustained part of L that you enter (0 is usual for occupancy live load; storage is long-term). Kcr = 1.5 for seasoned lumber in dry service, 2.0 for unseasoned lumber or wet service.
- **Load duration (code options):** CD per load type for ASD (NDS Table 2.3.2: live 1.0, roof live 1.25, snow 1.15, wind and earthquake 1.6; rain is not listed in the table, 1.15 is prefilled and the choice is yours) and the live load type for λ in LRFD (occupancy 0.8, storage 0.7, impact 1.25).
- **Ply fastening** (more than one ply): off by default, which leaves it as your design and gives the same results as before. On, the fasteners between plies are checked for a load from one side or both, single shear at each interface, the worst combination with its load duration or time effect factor. Common nails from each face (up to 3 plies) or through bolts (up to 4). The share of the load from the side, the length that takes a side point load and the service condition are inputs.

Auto-design (editor toolbar) varies the nominal size and number of plies; species and grade when unlocked. Objectives: least wood (section area); shallowest, then least wood; lowest cost index (lumber). Kept unless you select them: grade, species. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Bending stress, positive and negative moment, every combination	fb = M/S ≤ Fb' = Fb (CD or KF φ λ) CM Ct CL CF Cfu Ci Cr (Table 4.3.1); CL by Eq. 3.3-6 with RB = √(le d / b²)
Shear stress	fv = 3V/(2bd) ≤ Fv', with the loads within d of the support faces neglected (uniform) or reduced by x/d (point) when chosen
Bearing at the supports	fc⊥ = R/(b lb) ≤ Fc⊥' = Fc⊥ CM Ct Ci Cb (no CD in ASD; KF φ without λ in LRFD, Table 4.3.1)
Beam slenderness	RB = √(le d / b²) ≤ 50 (3.3.3.7) when the compression edge is unbraced
Live load deflection	Each of L, Lr, S and R alone, against span / n; cantilevers use twice the overhang
Total load deflection	The ASD gravity combinations with creep (NDS 3.5.2)

An upward reaction is reported as an advisory (provide an uplift connection). The steps show every factor with its basis.

Drawings

The drawing sheet (PDF / DXF, Drawing sheet button) shows the beam elevation with the supports, the loads by type in plf and lb and the spans, the section at the actual dressed size (each ply drawn, the member mark with species and grade), and the shear, moment and deflection diagrams of the governing combinations at the elevation's scale. Ply fastening, bearing hardware, connections and brace locations are not part of the calculation and are not drawn; the sheet notes say so.

Limitations

- Every NDS provision of the module was checked against the NDS 2024 text (2024 Edition with Commentary; the steps cite section, table and page): Tables 2.3.2 (CD), 2.3.3 (Ct), 2.3.5 (KF), 2.3.6 (φ), 3.3.3 (le, including the rows for four to seven loads new in 2024), 4.3.1, 4.3.8 (Ci), N3 (λ); Eqs. 3.3-5, 3.3-6, 3.4-2, 3.5-1, 3.10-2, 4.2-1 and 4.3-1 (timber size factor); 3.3.3.7 (RB ≤ 50), 3.4.3.1, 3.5.2 (Kcr), 4.1.4 (wet service above 19 %) and 4.3.9 (Cr). Corrected against the text: Fc⊥ takes no λ in LRFD (Table 4.3.1; the module had applied λ, which understated the LRFD bearing capacity by the factor λ), and the RB ≤ 50 limit is now checked.
- The reference values of NDS 2024 Supplement Tables 4A, 4B (Southern Pine Select Structural, No. 1, No. 2 and No. 3 / Stud; the dense and non-dense grades are entered as reference values) and 4D, the Table 4A / 4B / 4D size factors, flat use factors and wet service factors were read against the NDS 2024 Supplement (Chapter 4, 2024-07) on 2026-11-08: every built-in value agrees (none changed from the 2018 Supplement). Flat use of timbers is not offered (Table 4D gives Cfu on Fb and E for beams and stringers). The NDS 2024 Appendix E sample solutions confirm SPF No. 2 Ft = 450 psi and Fv = 135 psi, Hem-Fir No. 2 Ft = 525 psi and Fv = 150 psi, Southern Pine No. 2 2x4 Ft = 675 psi and Fv = 175 psi, and the Ft size factor 1.5 of 4 in wide lumber.
- Not feasible: Stud grade wider than 6 in (use No. 3), timbers used flat, incised timbers, built-up timbers.
- Ply fastening is calculated for side loads on nails and through bolts. Screws and proprietary fasteners are not, and fasteners are not placed along the member by the local shear.
- Notched ends: see Wood bearing and notches. Lateral bracing design, vibration and fire design are the engineer's.

Verification examples

Verification status: 55 values compared in 8 examples (4 published, 4 hand calculations from the cited equations): 50 within tolerance, 5 recorded deviations. Code basis: the provisions it uses were read in the target edition.

NDS 2024 has no worked beam example (its only sample solutions, Appendix E, are connection net-section and tear-out checks; the adjusted Ft and Fv they publish are verified through the column module). The module is verified against Breyer, Design of Wood Structures, Examples 6-18 / 6-19 (published values as tabulated in the free calcs.com validation page) and by hand calculations in closed form of the NDS 2024 equations (each derived step by step in `backend/tests/test_verify_wood_beam.py`):

Example	What it checks	Published / hand value	ILX Calcs
BREYER-6-18	2x6 DF-L No. 1 rafter, 13.5 ft, D 14 + Lr 20 psf, braced; ASD	Fb' = 1870 psi, M = Fb' S = 1178 lb-ft, Fv' = 225 psi, V = 1237.5 lb, Fc⊥' = 625 psi	1869 psi, 1178 lb-ft, 225 psi, 1237.5 lb, 625 psi
BREYER-6-18	Same rafter, LRFD (Ex. 6-19)	Fb' = 2.58 ksi, M' = 1625 lb-ft, Fv' = 311 psi, V' = 1710 lb, Fc⊥' = 938 psi	2582 psi, 1627 lb-ft, 311 psi, 1710 lb, 939 psi
HAND-1	2x10 DF-L No. 2 joist, 14 ft, D 20 + L 55 plf, ASD	fb = 1030.8 psi, Fb' = 1138.5 psi, fv at d = 49.5 psi, Δ_L = 0.300 in, Δ_T = 0.464 in	same (within 0.1 %)
HAND-1	LRFD	Fb' = 1966.4 psi, Fv' = 311.0 psi, Fc⊥' = 939.4 psi (no λ)	same
HAND-2	4x10 DF-L No. 2, lu = 12 ft (beam stability)	RB = 14.08, CL = 0.979, Fb' = 1057 psi	same

Example	What it checks	Published / hand value	ILX Calcs
HUD-5.8	Ridge beam, 13 ft, D 201 + S 268 plf, uplift 170 plf (HUD Residential Structural Design Guide, pp. 5-71 to 5-72; it stops before choosing a member)	469 plf, V 3,049 lb, M 9,908 lb-ft; uplift case V 124 lb, M 401 lb-ft	Same for D + S; uplift 121 lb, 393 lb-ft: the guide rounds the net load to 19 plf, exact 18.6 (documented deviation)
HUD-5.5A, B	Header 2-2x10 SPF (south) No. 2, 6.5 ft, 600 and 960 plf (pp. 5-62 to 5-64)	M 3,169 and 5,070 lb-ft, S 42.78 in^3, V 1,950 lb, fv 106 psi, Δ 0.11 in	Same. The guide's Fb' 1,454 and 1,918 psi and Fv' 175 psi use system factors 1.3, 1.05, 1.8 and CH = 2, which are not in the NDS: ILX reports 1,066 and 87.5 psi (documented deviation)
HAND-HIP	2-2x12, 17 ft, triangular load 0 to 200 plf	$V = wL/3$, $R = wL/6$, $M = wL^2/(9\sqrt{3})$, $\Delta = 0.01304\ WL^3/EI$ with $W = wL/2$ (no published hip example found)	same
HAND-OVERHANG	2x10, 10.5 + 3.5 ft, D 20 + L 55 plf	M 973 lb-ft with L on the back span against 817 lb-ft on the full length; R 61 lb and support moment 459 lb-ft with L on the overhang	same

Only the capacities of the Breyer rafter are compared (the free reproduction does not give the rafter spacing). The open item is a comparison with the AWC Structural Wood Design Examples (beam stability, notched and point-loaded sawn beams), which are not free.

CHAPTER 53

Glulam and SCL beam

A glued laminated timber, LVL or PSL beam in bending about the strong axis, shear and bearing, to NDS 2024 Chapters 2, 3, 5 and 8 with ASCE 7-22 combinations, in ASD or LRFD. Supports, loads, combinations, shear, bearing and deflection work as in the sawn lumber beam.

Inputs of the glulam beam

- **Member use:** a label for the report; the Wizard cards prefill each use. *Floor overhang* and *roof overhang*: a support at 3/4 of the length, the bottom edge with its own unbraced length (the overhang's length). *Header*: a simple span, no repetitive member factor; the jack, king and trimmer studs are in the stud wall. *Ridge beam*: a simple span with the rafter loads per foot, an uplift row (negative W), the bottom edge free over the span, the shear reduction off (rafters hung from the side do not load the opposite face, NDS 3.4.3.1). *Hip* and *valley*: a simple span of the plan length with a triangular load from 0 at the low end to the peak; the peak for jack rafters of equal slope on both sides of a 45 degree plan line is w = q Lh / 2, q the load per horizontal square foot and Lh the plan length of the hip. That triangle is the usual idealization, not a code rule: it is your judgment, as is whether the top end is really supported (a hip carried only by other hips and a ridge board is not). The axial force of the sloped member and deflection normal to the slope are not included. Every card starts from typical loads: replace them.
- **Partial loading:** on by default. On a member with more than one span or an overhang, L, Lr and S also act on part of the spans (ASCE 7-22 4.3.3 and 7.5; an overhang counts as a span, S keeps half its value on the other spans, the dead load stays everywhere). Up to three spans take every subset; more take alternate spans, adjacent pairs, single spans and all but an end span. The combination names say which: "D + L [live on span 1]". ASCE 7-22 7.5.1 lets you turn snow partial loading off for rafters across the ridge of a gable of slope 1/2 to 7 on 12, by the option.
- **Bottom edge under negative moment:** sheathing on top does not brace the bottom edge. Choose its own unbraced length where nothing holds it (an overhang, an interior support, wind uplift on an exposed ridge beam).
- **Product:** glulam (stress classes 16F-1.3E to 30F-2.1E SP and the combinations 24F-V4 and 24F-V8 DF of Supplement Table 5A), or LVL / PSL with the manufacturer's reference values (Fb at 12 in depth, Fv, Fc⊥, E, Emin) and the volume factor exponent n, CV = (12/d)^n.
- **Laminating species:** sets the exponent x of the glulam volume factor (x = 20 Southern Pine, 10 others). The Southern Pine stress classes require Southern Pine.
- **Width b and depth d**, the self-weight density, the supports and loads, the bearing length.
- **Lateral stability, service conditions, deflection limits and load duration** as for the sawn beam. Glulam is in wet service at 16 % moisture content or more; SCL in wet service needs the manufacturer's values.

Auto-design (editor toolbar) varies the width (standard widths) and depth (lamination increments); stress class when unlocked. Objectives: least wood (section area); shallowest, then least wood; lowest cost index. Kept unless you select them: stress class / combination. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Bending, positive moment	Fbx+ × (CD or KF φ λ) CM Ct × the lesser of CL and CV (NDS 5.3.6); CV by Eq. 5.3-1 with L = the span holding the peak moment (an upper bound of the distance between zero-moment points), b ≤ 10.75 in
Bending, negative moment	Fbx− (top face in tension) with the same factors
SCL bending	Fb × CL × CV, with the lesser of CL and CV when CV ≤ 1.0 and both when CV > 1.0 (NDS 8.3.6.1)
Beam stability CL	Eq. 3.3-6 with Ey,min (glulam, 5.2.7) or Emin (SCL); Fb* excludes CV only when CV ≤ 1.0 (3.3.3.8)
Beam slenderness	RB ≤ 50 (3.3.3.7) when the compression edge is unbraced
Shear	Fvx' (prismatic members); a note reminds that non-prismatic, notched or impact-loaded members and members at connections take Cvr = 0.72 (5.2.6, 5.3.10)
Bearing at the supports	Fc⊥x' with Cb (KF φ without λ in LRFD)
Live and total deflection	Ex (glulam) or E (SCL); creep Kcr = 1.5 dry, 2.0 wet

Drawings

The drawing sheet (PDF / DXF, Drawing sheet button) shows the beam elevation with the supports, the loads by type and the spans, the b × d section with the stress class (glulam laminations drawn schematically at 1 1/2 in, 1 3/8 in for Southern Pine; LVL and PSL as solid sections), and the shear, moment and deflection diagrams of the governing combinations at the elevation's scale. Camber, layup, connections, bearing hardware and brace locations are not part of the calculation; the sheet notes say so.

Limitations

- The NDS provisions were checked against the NDS 2024 text (steps cite section and page): Eq. 5.3-1 (CV, x = 20 Southern Pine, 10 others, b ≤ 10.75 in), 5.3.6 (CV and CL not cumulative), 5.2.4 (Fbx+ / Fbx−), 5.2.7 (which E for deflection and stability), 5.1.4 (wet service at 16 %), 8.3.6.1 (SCL CV with CL), Tables 2.3.5, 2.3.6, 5.3.1, 8.3.1 and N3. Corrected: Fc⊥ takes no λ in LRFD (Table 5.3.1); for SCL with CV > 1.0, Fb* of the beam stability factor now includes CV (3.3.3.8); RB ≤ 50 is checked.
- The Supplement Table 5A values and the glulam wet service factors were read against the 2024 Supplement on 2026-11-12 (Table 5A p. 80, Table 5A Expanded pp. 81-82, adjustment factors p. 79). Two values were corrected:
  - Ex,min of the 28F and 30F SP classes is 1.09 x 10^6 psi (it was 1.11).
  - Fby of 24F-V8 DF/DF is 1,550 psi (it was 1,450). The axial E values (Eaxial) are not printed in the 2024 Table 5A and stay to be confirmed against the code text. SCL volume factor exponents and wet service values come from the manufacturer (NDS 8.3.3, 8.3.6).
- Bending about the weak axis (y), curved and tapered members (Cc, CI), and the shear reduction Cvr are not modelled. SCL in wet service is reported as not feasible.
- The volume factor uses the span length, which is conservative for continuous beams.

Verification examples

Verification status: 33 values compared in 5 examples (2 published, 3 hand calculations from the cited equations): 31 within tolerance, 2 recorded deviations. **Thin: only 2 published examples.** Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

NDS 2024 has no worked glulam or SCL beam example. The module is verified against AWC Structural Wood Design Examples 2015/2018 Example E1.3 (published values as tabulated in the free Tekla Tedds verification example), AWC Technical Report 14 Example 3.1 (free), and hand calculations in closed form of the NDS 2024 equations (`backend/tests/test_verify_glulam_beam.py`). The published snow loads are ASCE 7-16 (ASD D + S); they are entered as S / 0.7 because ASCE 7-22 snow is strength level (D + 0.7S).

Example	What it checks	Published / hand value	ILX Calcs
AWC-E1.3	5 × 22 24F-1.8E SP roof beam, 32 ft, purlins 1000 D + 4000 S lb at 8 ft, braced at the purlins	M = 83.8 kip-ft, RB = 11.406, Fbx+* = 2760 psi, CL = 0.978, CV = 0.951, Fb' = 2625 psi, fb = 2496 psi, V = 7918 lb, Fv' = 305 psi, fv = 108 psi, R = 12,988 lb, Fc⊥' = 650 psi, fc⊥ = 433 psi, Δ = 1.843 in	83.8, 11.406, 2760, 0.975 (NDS 2024 uses Ey,min), 0.951, 2625, 2494, 7918, 305, 108, 12,980, 650, 433, 1.843
TR14-3.1	2-1/2 × 21 24F-V4 ridge beam, 28 ft, D 75 + S 300 plf, braced at 4 ft	Fbx* = 2760 psi, CV = 0.9871, fb from M = 36,750 ft-lb; CL = 0.88 and M' = 37,250 ft-lb by the TR 14 buckling-moment method	2760, 0.9871, 2400 psi; CL = 0.857, M' = 36,200 ft-lb by NDS 3.3.3 (documented deviation AWC-TR14-3.1-CL)
HAND-1	5-1/8 × 18 24F-1.8E, 24 ft, D 150 + L 400 plf	CV = 0.9475, Fb' = 2274 psi (ASD), 3928 psi (LRFD), fv at d = 92.4 psi, Δ_L = 0.666 in	same
HAND-2	Same section over two 16 ft spans, L 500 plf	fb− = 693.8 psi against Fbx−' = 1430.8 psi	same
HAND-3	3-1/2 × 11-7/8 LVL, lu = 14 ft	Fb' = 2600 CL CV with CV = 1.0014 > 1.0 (both apply)	same

CHAPTER 54

Wood column and post

A solid sawn, glulam or SCL column in compression, tension and bending, to NDS 2024 3.6 to 3.9 and 15.4 with ASCE 7-22 combinations, in ASD or LRFD. The column is pinned at both ends of its height; the effective lengths per axis come from the buckling length coefficient and the unbraced lengths you enter.

Inputs of the wood column

- **Member:** sawn lumber (species, grade, nominal size, or reference values entered), glulam (Table 5A stress class; the axial Ft and Fc are used) or SCL (manufacturer values). Axis 1 is the strong axis (dimension d), axis 2 the weak axis (b).
- **Height and bracing:** the column height, Ke (NDS Appendix G; 1.0 for pinned ends is prefilled), the unbraced lengths lu1 and lu2 for buckling about each axis (0 = the height) and the unbraced length of the compression edge for the beam stability of strong-axis bending (0 = the height).
- **Loads:** axial loads per load type (compression positive, tension negative) with their eccentricities e1 and e2, uniform side loads (wind on a post) about either axis over the height, and the area of holes at the net section for tension.
- **Service conditions and load duration** as in the beams.

Auto-design (editor toolbar) varies the sawn size (or glulam / SCL width and depth); grade when unlocked. Objectives: least wood (section area); lowest cost index. Kept unless you select them: grade. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Slenderness	le/d ≤ 50 (3.7.1.4), for members in compression
Compression	fc = P/A ≤ Fc' = Fc* CP, CP by Eq. 3.7-1 (c = 0.8 sawn, 0.9 glulam and SCL), the smaller CP of the two axes
Combined bending and compression	Eq. 3.9-3 (uniaxial or biaxial, with the stability limit Eq. 3.9-4); with an eccentric load Eq. 15.4-1 with Eq. 15.4-2
Strong-axis bending stability	CL by Eq. 3.3-6 with le from the general values of Table 3.3.3; for glulam the lesser of CL and CV (x = 20 for the Southern Pine classes); RB ≤ 50 (3.3.3.7)
Tension	ft = T/An ≤ Ft' (3.8)
Combined bending and tension	Eqs. 3.9-1 and 3.9-2 (bending about one axis), Fb* without CL and Fb** without CV
Shear from side loads	fv = 3V/(2A) ≤ Fv'

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows the column elevation with the axial loads by type onto the top, the side loads along the height, the height, the member mark and a callout with Ke, the unbraced lengths and the eccentricities, and the member section (b x d, species and grade or stress class). End fixity, brace locations between the ends, base and cap hardware are not part of the calculation and are not drawn; the sheet notes say so.

Limitations

- Built-up and spaced columns (15.2, 15.3), round and tapered columns, and tension with bending about both axes (reported as not feasible) are outside the module. Fixed ends are represented through Ke.
- Checked against the NDS 2024 text: 3.6.3, 3.7.1 (Eq. 3.7-1, c = 0.8 / 0.9, le/d ≤ 50), 3.8, 3.9 (Eqs. 3.9-1 to 3.9-4), 15.4 (Eqs. 15.4-1, 15.4-2), 5.2.7 (glulam Ex,min / Ey,min for buckling about x / y), 5.3.7 (Cfu = (12/dy)^(1/9) on Fby), 8.3.6 (SCL volume factors). Corrected: the weak-axis eccentric term of Eq. 15.4-1 now carries the amplification 0.234 [(fb1 + fc 6e1/d1)/FbE]² (it was missing; the ratio was slightly low with eccentricity about both axes); the glulam volume factor of the Southern Pine classes uses x = 20; Eq. 3.9-1 uses Fb* with CV for glulam and Eq. 3.9-2 uses Fb** without CV for SCL; RB ≤ 50 is checked.
- The sawn lumber reference values (NDS 2024 Supplement Tables 4A, 4B, 4D) were read against the 2024 Supplement on 2026-11-08 and agree. Beams and stringers (timbers whose width exceeds the thickness by more than 2 in) take the Table 4D flat use factor on Fb and Emin about the weak axis (NDS 2024 4.3.7.2, Table 4.3.1 footnote 1): Select Structural 0.86 on Fb, No. 1 0.74 on Fb and 0.90 on Emin. The glulam values (Supplement Table 5A) stay marked (to confirm against the code text): Supplement Chapter 5 is not on file (`research/DOCUMENTS_NEEDED.md`). The SCL tension volume factor (8.3.6.2) is the manufacturer's: enter Ft already adjusted for it.
- Side loads are uniform over the full height; deflection of the column is not checked.

Verification examples

Verification status: 35 values compared in 13 examples (10 published, 3 hand calculations from the cited equations): 35 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published values of AWC Structural Wood Design Examples 2015/2018 Example E1.4 (as tabulated in the free Tekla Tedds verification example), Aghayere & Vigil, Structural Wood Design 2nd ed., Example 5.6 (as tabulated in the free calcs.com validation page), the NDS 2024 Appendix E sample solutions (the side members of bolted connections, ASD with all factors 1.0) and hand calculations in closed form (`backend/tests/test_verify_wood_column.py`). Published ASCE 7-16 snow loads are entered as S / 0.7 (ASCE 7-22 strength-level snow); in LRFD only the capacity is compared.

Example	What it checks	Published / hand value	ILX Calcs
AWC-E1.4a	4x4 SP No. 2 post, 10 ft, Ke = 1.0	Fc* = 1450 psi, FcE = 357 psi, CP = 0.232, Fc' = 336 psi, P = 4120 lb	1450, 356.6, 0.232, 336.3, 4120
AWC-E1.4b	6x6 SP No. 2 post (Table 4D), 10 ft	Fc* = 525 psi, FcE = 760 psi, CP = 0.801, Fc' = 421 psi, P = 12,725 lb	525, 759.8, 0.801, 420.6, 12,723

Example	What it checks	Published / hand value	ILX Calcs
AV-5.6	6x6 DF-L No. 1, 12 ft, D 6800 + S 10,200 lb	ASD: P = 17,000 lb, FcE = 695 psi, CP = 0.502, capacity 17,463 lb; LRFD: FcE = 1042 psi, CP = 0.502, capacity 26,300 lb	17,000, 695.5, 0.503, 17,497; 1040, 0.501, 26,206
NDS-E.6	SPF No. 2 2x4 and 4x4, NDS 2024 p. 175	Ft' = 450 × 1.5 = 675 psi, ft at Z_NT' = 2,468 lb = 675 psi, Fv' = 135 psi	same
NDS-E.8	Hem-Fir No. 2 2x4, p. 178	Ft' = 525 × 1.5 = 788 psi, ft at Z_NT' = 3,470 lb, Fv' = 150 psi	same (787.5 psi)
NDS-E.9	Southern Pine No. 2 2x4, pp. 178-179	Ft' = 675 psi, Fv' = 175 psi	same
HAND-1	6x6 DF-L No. 1, 10 ft, D 6000 + L 10,000 lb	CP = 0.692, Fc' = 691.7 psi (ASD); CP = 0.640, Fc' = 1105.6 psi (LRFD)	same
HAND-2	Same post, D 8000 lb and a 100 plf wind side load	Eq. 3.9-3 = 0.332 under D + 0.6W	same
HAND-3	2x6 DF-L No. 2 in tension	Ft' = 747.5 psi (ASD), 1291.7 psi (LRFD)	same

CHAPTER 55

Wood stud wall

Bearing studs at a spacing under axial load and out-of-plane wind, to NDS 2024 3.6 to 3.10 and 15.4 with ASCE 7-22 combinations, in ASD or LRFD. One stud is designed for its tributary width (the stud spacing); the studs are pinned at the plates.

Inputs of the stud wall

- **Studs:** species, grade and size (2x4 to 4x8 dimension lumber), plies per stud, spacing, height and the repetitive member factor Cr (three or more studs at 24 in or less with sheathing, NDS 4.3.9).
- **Bracing:** Ke (1.0 for studs pinned at the plates), the weak-axis bracing (sheathing fastened to the studs, blocking at a spacing, or none) and whether both stud edges are braced for bending (sheathing or finish on both faces, CL = 1.0). These are your judgment; the usual values are prefilled.
- **Loads:** axial loads per foot of wall per load type with an eccentricity (its moment adds to the wind moment), and the wind pressure W on the wall (the ASCE 7-22 components and cladding pressure, strength level, acting in both directions).
- **Opening:** optional. The rough opening width, the head height (the height of the jack studs), the jack and king studs each side, how they are held about the weak axis, the header's size, plies, species and grade (for its bearing on the jacks and the room above the head), and concentrated loads on the header as a reaction at this jamb. One jamb is designed. The wall load over the opening goes through the header to the jacks; the king studs take the wall load of half a stud space beside them and the wind on half the opening and half a stud space, over the full height, and the jacks are not counted in bending, the usual conservative assumption. With the jacks not braced, built-up plies take NDS 15.3 (Kf = 0.6 nailed, 0.75 bolted on the weak axis, and not below the plies as single columns). Sheathing broken by the opening braces one face at most: braced or not is your judgment. **The header member is not designed here:** the checks list its span, line loads and reactions; copy them into the sawn lumber or glulam beam with the use Header.
- **Plates:** the species and grade of the plate under the stud and whether Cb applies (the stud bears at least 3 in from the plate end).
- **Wind deflection:** the wind load factor (IBC Table 1604.3 note f permits 0.42 of the components and cladding load) and the limit height / n (360 plaster or stucco, 240 other brittle finishes, 120 flexible finishes).

Auto-design (editor toolbar) varies the stud size, plies and spacing (12, 16, 19.2, 24 in.); grade when unlocked. Objectives: least wood per foot of wall; lowest cost index. Kept unless you select them: grade. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Slenderness	le/d ≤ 50 about both axes (3.7.1.4)
Stud compression	fc ≤ Fc' = Fc* CP, CP of the governing axis (3.7.1)

Check	Basis
Combined bending and compression	Eq. 3.9-3 under each combination with wind, with its CD or λ; Eq. 15.4-1 when the axial load is eccentric; Fb' with Cr and CL
Shear from wind	fv = 3V/(2A) ≤ Fv'
Bearing on the plate	fc⊥ = P/(b d) ≤ Fc⊥' of the plate with Cb (3.10.2, 3.10.4)
Wind deflection	5wℓ⁴/(384 E' I) with the reduced wind load, against height / n
Jack studs (with an opening)	Slenderness; compression with Cp over the head height, built-up per 15.3 if not braced (3.6, 3.7, 15.3)
Jack studs on the plate; header on the jacks	fc⊥ with the plate's Cb; the header end with Cb = 1.0 (3.10.2, 3.10.4)
King studs (with an opening)	Eq. 3.9-3 with their share of the wall load and the wind on half the opening, no repetitive member factor; shear; deflection

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows a representative length of wall: studs (and plies) at the designed spacing, the plate the studs bear on, the stud height, blocking rows when the weak axis is braced by blocking, and the wall loads by type along the top; and the stud section. Wall length, openings, headers, sheathing, the number of top plates and anchorage are not part of the calculation and are not drawn; the wind pressure and the bracing basis are in the sheet notes.

Limitations

- One opening per calculation; the design of the header member, sills, cripples and their fastening, the sheathing and its fastening, and the in-plane shear of the wall are not covered (see the shear wall module for in-plane design). Force transfer around openings is a later extension.
- Checked against the NDS 2024 text (steps cite section and page): Tables 2.3.2, 2.3.3, 2.3.5, 2.3.6, 4.3.1, 4.3.8, N3; 3.3.3 (CL, RB ≤ 50, now checked when the compression edges are not braced), 3.7.1, 3.9.2, 3.10.2, 3.10.4, 4.3.9 (Cr) and 15.4 (Eq. 15.4-1 with Eq. 15.4-2). Corrected: the plate bearing Fc⊥' takes no λ in LRFD (Table 4.3.1).
- Reference values and the Table 4A / 4B size factors were read against the NDS 2024 Supplement (Chapter 4, 2024-07) on 2026-11-08 and agree. Stud grade wider than 6 in is reported as not feasible (Southern Pine Stud takes the Table 4B "No. 3 and Stud" values at every width).

Verification examples

Verification status: 33 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 31 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

NDS 2024 has no worked stud wall example. The module is verified against Aghayere & Vigil, Structural Wood Design 2nd ed., Example 5.9 (published values as tabulated in the free calcs.com validation page; its ASCE 7-16 snow load is entered as S / 0.7 and its plate bearing without Cb, as in the example) and hand calculations in closed form of the NDS 2024 equations (`backend/tests/test_verify_stud_wall.py`):

Example	What it checks	Published / hand value	ILX Calcs
AV-5.9	2x6 DF-L No. 2 at 24 in, 10 ft, per stud D 1440 + L 640 + S 1440 lb, W 25 psf	P = 3000 lb, CP = 0.492, Fc' = 840 psi, utilization 43.3 %, plate Fc⊥' = 625 psi, Fb' = 2153 psi; under D + 0.75(L + S + 0.6W): CP = 0.377, Fc' = 896 psi, Eq. 3.9-3 = 0.49	3000, 0.4915, 839.3, 43.3 %, 625, 2152.8; 0.376, 893.7, 0.491
HUD-5.4	2x4 SPF Stud at 24 in, 8 ft, W 16 psf service (HUD Residential Structural Design Guide pp. 5-59 to 5-61)	fb 1,004 psi, FcE 479 psi, Fc' 426 psi (CD 1.6), Fc' 419 psi (CD 1.25)	Same within 1 %, except Fc' 433 psi at CD 1.6: the guide rounds Cp to 0.35 (documented deviation). The guide's Fb' 1,782 psi uses its system factor 1.5, not in the NDS: ILX reports 1,366 psi (documented deviation)
HUD-5.5	Jack studs under a header: 6.25 ft opening, 2 jacks, 600 plf (pp. 5-63 to 5-64)	Span 6.5 ft, R 1,950 lb	same; bearing stress on 3 in. from the guide's 650/lb: 216.7 psi
HUD-5.6	4x4 SPF No. 2 column, 7.3 ft, 4,800 lb, entered as a jack stud unbraced (pp. 5-65)	Cp 0.44, Fc' 582 psi, capacity 7,129 lb	Cp 0.438, 579 psi, 7,095 lb (within 1 %)
HAND-OPEN	2x6 DF-L No. 2 jamb, 6 ft opening, 2 jack + 2 king, ASD and LRFD	Hand: reaction, Fc', king stud wind width, Eq. 3.9-3, deflection	same
HAND-1	2x6 DF-L Stud at 16 in, 10 ft, D 300 + L 400 plf, W 25 psf	Fc' = 597.6 psi (D + L), Eq. 3.9-3 = 0.330 (D + 0.6W, ASD), 0.433 (1.2D + 1.0W + L, LRFD), plate Fc⊥' = 781 psi, Δ = 0.108 in	same

CHAPTER 56

CLT panel

A cross-laminated timber floor or roof panel spanning one way between two supports, to NDS 2024 Chapter 10 and ANSI/APA PRG 320-19 with ASCE 7-22 combinations, in ASD or LRFD. The design is per foot of panel width in the major strength direction.

Inputs of the CLT panel

- **Layup:** the PRG 320 basic layups E1 to E4 (machine stress rated laminations) and V1 to V3 (visually graded), 3, 5 or 7 plies, and the lamination thickness (PRG 320 Table A2 uses 1-3/8 in). The effective properties are derived from the lamination grades (below). *Values entered* takes (FbS)eff, (EI)eff, (GA)eff and Vs from the PRG 320 table or the manufacturer, with the panel thickness for the self-weight.
- **Span and loads:** the simple span and uniform area loads per load type; the self-weight uses the density.
- **Service conditions:** temperature; PRG 320 panels are for dry service.
- **Deflection limits** with creep (Kcr = 2.0 for CLT in dry service, NDS 3.5.2) and the sustained part of the live load; **load duration** as in the beams.

The effective properties per foot of width (PRG 320 Annex A): $(EI)_{eff} = \Sigma E (b t^3/12 + b t z^2)$ over the parallel layers (the cross layers are ignored); $(GA)_{eff} = a^2 / [t1/(2G1b) + \Sigma ti/(Gi\ b) + tn/(2Gn\ b)]$ with G = E/16 for parallel and E/160 (rolling shear) for cross layers; $(FbS)_{eff} = 0.85\ Fb\ (EI)_{eff} / (E\ h/2)$; $Vs = Fs\ (2/3)\ b\ h$.

Auto-design (editor toolbar) varies the number of plies (3, 5, 7); the PRG 320 layup grade when unlocked. Objectives: thinnest panel (least wood); lowest cost index. Kept unless you select them: layup. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Bending	$M = w\ell^2/8 \le (FbS)_{eff}' = (FbS)_{eff}$ (CD or KF φ λ) CM Ct CL (CL = 1.0)
Rolling shear	$V = w\ell/2 \le Vs' = Fs(Ib/Q)_{eff}' = Vs$ CM Ct; no CD (10.3.2); LRFD KF = 2.00, φ = 0.75, no λ (Table 10.3.1)
Live load deflection	$5w\ell^4/(384\ (EI)_{app})$, $(EI)_{app} = (EI)_{eff} / (1 + Ks\ (EI)_{eff}/((GA)_{eff}\ \ell^2))$, Ks = 11.5 (NDS Eq. 10.4-1)
Total load deflection	The ASD gravity combinations with creep

Drawings

The drawing sheet (PDF / DXF, Drawing sheet button) shows the one-way span of a 1 ft panel strip with the area loads by type (self-weight in D), the panel section across the span with each layer (longitudinal layers hatched, cross layers plain, layer and overall thicknesses dimensioned), and the shear and moment diagrams per unit width of the governing combination. Lamination widths, panel joints, bearing, connections, vibration and fire design are not part of the calculation; the sheet notes say so.

Limitations

- Checked against the NDS 2024 text: Table 10.3.1 and 10.3.2 to 10.3.12, Eq. 10.4-1 with Ks = 11.5 (Table 10.4.1.1, uniform load, pinned), the lamination thickness 5/8 in to 2 in (10.1.3.1), the panel thickness limit of 20 in (10.1.3.2, now checked), dry service only (10.1.5) and Kcr = 2.0 (3.5.2). Corrected: the rolling shear capacity takes no load duration factor and, in LRFD, KF = 2.00 and φ = 0.75 without λ (Table 10.3.1; the module had used the Fv factors CD, KF 2.88 and λ). In ASD the rolling shear capacity no longer rises with CD (for example 13 % lower under snow); in LRFD it is 2.00/(2.88 λ) of the former value (0.87 for λ = 0.8, 0.69 for λ = 1.0).
- ANSI/APA PRG 320 is not on disk: the lamination grades (PRG 320 Table 1), the Annex A equations and the rolling shear capacity formula stay marked "to confirm against the code text" and are listed in `research/DOCUMENTS_NEEDED.md`. NDS 2024 10.2 takes the reference values from the manufacturer's approved recommendations or evaluation report. The derived E1 and V1 3-ply properties agree with the Table A2 values as recalled to within 1 %; the 5- and 7-ply bending values are about 1 % below the recalled table values. The 5-ply V1 and V3 Table A2 values published in the WoodWorks examples ((FbS)eff, (EI)eff, (GA)eff, Vs) are reproduced to within 1 % (V1 (GA)eff 1.06 against the quoted 1.1); the V3 major-lamination Fb was corrected from 1,100 to 750 psi with them (the former value overstated the V3 bending capacity by 46 %).
- One-way simple spans with uniform loads only. Continuous spans, two-way action, concentrated loads, openings, bearing at the supports, the minor strength direction, vibration and fire design are not covered. Wet service is reported as not feasible.

Verification examples

Verification status: 34 values compared in 8 examples (7 published, 1 hand calculations from the cited equations): 34 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The WoodWorks CLT floor design examples (free slides: Breneman 2023-12, Gesa 2025-03, the 2021-11 webinar), which use PRG 320 Table A2 values, hand calculations in closed form, and PRG 320 Table A2 values as recalled (to be confirmed against the published table) in `backend/tests/test_verify_clt_panel.py`:

Example	What it checks	Value	ILX Calcs
WW-V3-5	5-ply V3, 15 ft, D 75 + L 50 psf (built-in layup)	Ma = 3516 lb-ft/ft, (FbS)eff = 4000 lb-ft/ft, Va = 938 lb/ft, Vs = 3025 lb/ft, (EI)eff = 363 × 10^6 lb-in²/ft, (GA)eff = 0.98 × 10^6 lb/ft	3516, 3977, 937.5, 3025, 360.3, 0.981
WW-V3-5d	Same panel with the table values entered	ΔST = 0.177 in, ΔT = 2.0 × 0.266 + 0.177 = 0.709 in	0.1775, 0.710
WW-V1-5	5-ply V1, 16 ft, D 40 + L 40 psf (built-in layup)	Mb = 2560 lb-ft/ft, (FbS)eff = 4800 lb-ft/ft, Va = 640 lb/ft, Vs = 3300 lb/ft, (EI)eff = 415 × 10^6, (GA)eff = 1.1 × 10^6	2560, 4773, 640, 3300, 411.8, 1.06

Example	What it checks	Value	ILX Calcs
WW-V1-5d	Same panel with the table values entered	ΔST = 0.159 in, ΔT = 0.477 in	0.1589, 0.4766
WW-V1-3	3-ply V1 (residual section of the fire example)	(FbS)eff = 2090 lb-ft/ft	2089
HAND-1	E1 5-ply, 15 ft, D 15 + L 50 psf	(EI)eff = 437.5 × 10^6 lb-in²/ft, (GA)eff = 0.925 × 10^6 lb/ft, (FbS)eff = 10,341 lb-ft/ft, Vs = 2475 lb/ft, Δ_L = 0.152 in; LRFD Vs' = 2475 × 2.00 × 0.75 = 3713 lb/ft	same
PRG-E1-3	E1 3-ply properties	4525 lb-ft/ft, 115 × 10^6 lb-in²/ft, 0.46 × 10^6 lb/ft, 1490 lb/ft	4526, 114.9, 0.462, 1485
PRG-V1-3	V1 3-ply properties	2090, 108, 0.53, 1980	2089, 108.1, 0.531, 1980

CHAPTER 57

Wood shear wall

A wood structural panel shear wall to AWC SDPWS 2021 Section 4.3, with the ASCE 7-22 load factors for wind and seismic. One calculation covers one story of a wall line: a segmented wall (one or more full-height segments) or a perforated wall, sheathed on one or both faces, with the overturning of the stories stacked above.

> **Source note.** SDPWS 2021 (Chapter 4) was read against this module on 2026-11-08: every Table 4.3A value it holds, the design factors (4.1.4), the aspect ratio rules (4.3.3), Eq. 4.3-1 (4.3.4), two-sided walls (4.3.5.4), walls in a line (4.3.5.5), perforated walls (4.3.2.3, 4.3.5.6) and force transfer (4.3.2.2, 4.3.3.3). That reading corrected three Ga values and five rules; see Changes below.

Inputs of the wood shear wall

- **Sheathing:** panel grade (Structural I or rated sheathing), thickness and nail (6d, 8d or 10d common), the edge nail spacing (6, 4, 3 or 2 in), one or both faces. The framing species sets G (NDS Table 12.3.3A); for species other than Douglas Fir-Larch and Southern Pine the capacities are multiplied by 1 − (0.5 − G), never more than 1.0. The panel type picks the OSB or plywood column of Ga (4-ply or more plywood and composite panels: Ga × 1.2), and framing wetter than 19 % at fabrication halves Ga. **Entered** replaces the table: vn and Ga of the whole wall (both faces), before the species factor. SDPWS 2021 prints one vn for wind and seismic: type it in the vn box. The second box takes a seismic vs from an SDPWS 2015 table or report only; seismic uses the smaller of vn and 1.4 vs, so with a 2021 value typed in both boxes nothing is raised.
- **Two faces:** both faces of wood structural panels combine as Kmin (Ga1 + Ga2), Kmin the smaller vn / Ga of the two faces, for wind and seismic (SDPWS 2021 4.3.5.4.1). Identical faces simply add.
- **Hold-down on the inside face of the end post:** with 10d nails, Table 4.3A footnote 10 multiplies vn by 0.92. Tick the box under Chords and hold-downs when it applies; a note reminds you on every wall with 10d nails.
- **3/8 and 7/16 in panels:** SDPWS Table 4.3A footnote 2 permits the 15/32 in values when the studs are at 16 in o.c. or the panels run across the studs (used by the AWC and WoodWorks examples below). The module has no switch for it: pick the 15/32 in row of the same grade and nail in that case.
- **Wall:** the wall type, the height h and the length bs of each full-height segment. For a segmented wall you choose how the story shear is shared (4.3.5.5): in proportion to each segment's capacity, where a segment over 2:1 counts with 2bs/h (4.3.5.5.1 Exception 1, instead of 1.25 − 0.125 h/bs), or to its stiffness (equal top deflection by Eq. 4.3-1, with the anchorage slip; segments over 2:1 then take 1.25 − 0.125 h/bs). For a perforated wall enter the total length Ltot and the openings (width × clear height).
- **Force transfer around openings:** the piers left to right as the segments, and one opening between each pair: width, height and the wall above it. Each opening needs wall above and below it, so a door is refused: use a segmented or perforated wall there. One story at a time. Strap capacity is optional. The least depth above and below (1 ft) and the largest length to depth (6.5, advisory) are APA's limits as Malone quotes them, inputs because the code does not set them.
- **Loads:** the wind and seismic story forces at the top of this wall, and the forces of any stories stacked on it (same segments; each acts at the top of its story). ASCE 7-22 forces are strength level (W, QE); choose **ASD level** if

you enter 0.6W and 0.7ρQE. The dead load on the wall resists overturning. ρ (ASCE 7-22 12.3.4) and SDS (vertical seismic Ev = 0.2SDS D, 12.4.2.2) are inputs.

- **Chords and hold-downs:** the chord centroid from each segment end (moment arm d = bs − 2 × offset), the chord E and area A, the anchorage slip Δa (hold-down, rod, shrinkage and crushing, from the manufacturer) and the hold-down capacity in the method's terms (0 reports the force only).
- **Drift and deflection:** Cd and Ie (ASCE 7-22 Tables 12.2-1 and 1.5-2), the allowable story drift ratio (Table 12.12-1, 0.020 prefilled), the serviceability wind factor and an optional wind deflection limit h / n.
- **Code options:** the aspect ratio factor for segments with h/bs > 2: SDPWS 2021 (1.25 − 0.125 h/bs, 4.3.3.2, with 2bs/h where 4.3.5.5.1 or 4.3.3.4 calls for it) or, for a project still under SDPWS 2015, 2bs/h for wind and seismic everywhere (as the AWC 2015/2018 design examples apply it).

Auto-design (editor toolbar) varies the sheathing and nail (Table 4.3A row), edge nail spacing and one or two faces. Objectives: lowest cost index (sheathing + nails). Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Aspect ratio of each segment ≤ 3.5	SDPWS 2021 4.3.3.1, Table 4.3.3 (blocked wood structural panels); piers 4.3.3.3, perforated segments 4.3.3.4
Unit shear, wind and seismic	v = load factor × V × share / bs against vn (Table 4.3A) × species factor (footnote 3) × aspect ratio factor: wind φD vn with φD = 0.80 or vn / 2.0, seismic φD vn with φD = 0.50 or vn / 2.8 (4.1.4); two faces Kmin (Ga1 + Ga2) (4.3.5.4.1); 10d with an inside-face hold-down × 0.92 (footnote 10, an input)
Perforated wall	Co = Awall / (3Ao + Afhs) ≤ 1.0 (Eq. 4.3-6); Σbi with segments over 2:1 at 2bi/h of their length (4.3.3.4); v = V / (Co Σbi) (Eq. 4.3-9); the wall's vn ≤ 2,435 plf, one or two faces; height ≤ 20 ft (4.3.2.3). With the SDPWS 2015 option, Co = r / (3 − 2r) × Ltot / ΣLi with the adjusted ΣLi, as in AWC example E3.4
Force transfer around openings	SDPWS 2021 4.3.2.2, 4.3.3.3 and a rational analysis, the APA simplified Diekmann method: T = Lo Lpier / (Lleft + Lright) at each opening; pier shear (V / L)(Li + T) / Li with the aspect ratio factor, h = the taller opening beside the pier; above and below the corners the pier's force less the strap forces; vv = (Fo − wD L / 2 + wD x) / (ha + hb) above and below an opening; strap force = vv over the tributary width, both load directions; piers at least 2 ft, aspect ratio 3.5:1 on the piers; deflection the average of the piers by Eq. 4.3-1
Hold-down tension	(overturning − 0.9D or (0.9 − 0.2SDS)D resisting) / d for LRFD, 0.6D and (0.6 − 0.14SDS)D for ASD (ASCE 7-22 2.3.6 / 2.4.5; SDPWS 4.3.6.1.2, 4.3.6.4.2); perforated walls T = Vh / (Co Σbi) with no dead load (Eq. 4.3-8); against the entered capacity
Chord compression	Reported (overturning share); the post is checked with the wood column or stud module
Seismic story drift	Δ = Cd δxe / Ie with δxe by Eq. 4.3-1 under QE (ρ = 1.0) against the allowable drift; a perforated wall with v = V / (Co Σbi) and b = Σbi (4.3.4.2)
Wind deflection	Eq. 4.3-1 under the serviceability wind against h / n (when entered)
Detailing notes	3 in framing and staggered nails at 2 in spacing, 10d at 3 in, or over 980 plf in SDC D to F (4.3.7.1); joints offset on two-faced walls (footnote 6); 10d with an inside-face hold-down (footnote 10); nailing assumed by the table; plate washers and sill anchorage not checked

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows the shear wall elevation of the story designed: the full-height segments (marked SW-1, SW-2 ... with the shear wall cross) or the perforated wall with its openings (O-1, O-2 ...), the stories above as a dashed outline, the story forces at each level, the hold-downs at the ends with net uplift labelled with the tension T, the sheathing and edge nailing callout, and the segment lengths and story heights. The unit shear of each segment, the opening sizes and the largest hold-down tension are in the design summary. The distance between segments, the opening locations, the hold-down device, end posts, sill anchorage and collectors are not part of the calculation; the sheet notes say so.

Limitations

- Blocked wood structural panel sheathing of Table 4.3A only. Unblocked walls (Cub), gypsum, fiberboard, particleboard and lumber sheathing (Tables 4.3B to 4.3D) are not offered; use **Entered** with your own values only for wood structural panels.
- Stories above stack on the same segments; a perforated wall is designed one story at a time.
- The anchorage slip Δa is a constant entered value (not computed from the hold-down force).
- Not checked: chord / end-post compression and bearing, sill plate anchor bolts and plate washers (4.3.6.4.3), collectors, the foundation.
- A perforated segment over 3.5:1 fails the aspect check here; SDPWS 4.3.3.4 lets it be left out of Σbi instead: leave it out of the segments.
- Read against SDPWS 2021 on 2026-11-08: every vn, Ga and nail bearing length of Table 4.3A held here, the footnotes used (2, 3, 4, 5, 6, 10), 4.1.4, 4.3.2.2, 4.3.2.3, 4.3.3, 4.3.4, 4.3.5.4, 4.3.5.5, 4.3.5.6, 4.3.6.1 and 4.3.7.1. The SDPWS 2015 option rests on the AWC 2015/2018 examples as Tekla quotes them; SDPWS 2015 is not on file.
- From NDS 2024 (on disk) the module takes only the framing specific gravity G (Table 12.3.3A; Redwood corrected to G = 0.37 at verification, which lowers its species factor from 0.94 to 0.87) and the common nail dimensions (Table L4); both are confirmed. NDS 2024 does not contain the SDPWS provisions.

Force transfer: the method rests on a rational analysis with assumptions the engineer accepts: full-length collectors, and the straps, blocking and their nails are for you to detail, at the head and sill, to develop the force. It cannot take a door or several stories here. Each pier's deflection is taken at the wall height by default, which is larger than APA T555C's method (about 0.48 in. against 0.352 in. by hand on its example); **Deflection of the piers** switches to the APA method: each pier pushed each way, h from the wall below an opening when pushed toward it, Δa in proportion to v h over the hold-down capacity, so enter Δa at that capacity. T555C permits piers down to 18 in. under conditions; this calculation keeps the code's 2 ft.

Verification examples

Verification status: 186 values compared in 27 examples (16 published, 11 hand calculations from the cited equations): 184 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples, all free from their publishers (research/sources/design_examples):

- **WoodWorks, Five-Story Light-Frame Wood Structure over Podium, Design Example, 2nd Ed. (2025)** (SDPWS 2021, ASCE 7-22): Tables 3.5, 3.13C, 3.18, 3.19 and 4.5.

- **WoodWorks, Introduction to Wood: Structural Lateral Framing Design (January 2025 slides)**: two-story shear wall, slides 56-80.
- **AWC Structural Wood Design Examples 2015/2018, E3.1, E3.2, E3.4** (SDPWS 2015), as quoted with inputs, wall figure and results on the free Trimble Tekla Tedds 2026 verification pages (the AWC book itself is not on file).

Example	What it checks	Published value	ILX Calcs
WW5S-SW-Roof	29 × 10 ft wall, 15/32 in OSB, 10d at 6 in: ASD v, vn / 2.8, Ga, Eq. 4.3-1 terms, δSW, Δ = Cd δ	227, 310, 22, 0.002 / 0.147 / 0.045, 0.194, 0.78 in	226.8, 310.7, 22, 0.002 / 0.147 / 0.045, 0.194, 0.78
WW5S-SW-6th	Same, 10d at 4 in	447, 460, 30, 0.003 / 0.213 / 0.069, 0.284, 1.14	447.2, 460.7, 30, 0.003 / 0.213 / 0.069, 0.284, 1.14
WW5S-SW-5th	Same, 10d at 2 in	612, 770, 52, 0.003 / 0.168 / 0.094, 0.265, 1.06	612.5, 769.6, 52, 0.003 / 0.168 / 0.094, 0.265, 1.06
WW5S-SW-4th	Same, 10d at 2 in	723, 770, 52, 0.002 / 0.199 / 0.107, 0.307, 1.23	722.6, 769.6, 52, 0.002 / 0.199 / 0.107, 0.308, 1.23
WW5S-SW-3rd	Same, both faces at 4 in	778, 920, 60, 0.002 / 0.185 / 0.100, 0.287, 1.15	777.7, 921.4, 60, 0.002 / 0.185 / 0.100, 0.287, 1.15
WW5S-SW-W3	Wind chapter, 2 × 29 ft, 8d at 6 in (15/32 in values): ASD v and allowable, seismic / wind	158 / 261, 261 / 365 plf	157.4 / 260.6, 260.7 / 365
WW25-SW-L1	Two-story 12 ft wall, lower story, 15/32 in Structural I 8d at 3 in: v, capacity, T	563 / 375 plf, 770 / 550 plf, 7,500 / 5,417 lb	562.5 / 375, 770 / 550, 7,500 / 5,417
WW25-SW-L2	Upper story	188 / 167 plf, 1,875 / 1,667 lb	187.5 / 166.7, 1,875 / 1,667
WW25-SW-D	Eq. 4.3-1 under wind (v = 563 plf, h = 9 ft, Ga 24, Δa 0.134 in)	0.007, 0.211, 0.101, 0.32 in	0.007, 0.211, 0.101, 0.318
AWC-E3.1	Segmented, wind, 2 × 5 ft + 2 × 3 ft, SDPWS 2015 2bs/h	vw 1,065, 532.5 plf, 7,455 lb, T 3,549 lb	1,065, 532.5, 7,455, 3,549
AWC-E3.2	Segmented, seismic	380 plf, 5,320 lb, T 3,043 lb	380.4, 5,325, 3,043
AWC-E3.4	Perforated, seismic, Ltot 40 ft, ΣLi 18 ft	Co 0.78, 5,335 lb, T 3,034 lb	0.784, 5,370, 3,017

Two fixes came out of these examples (2026-11-01): the seismic capacity now follows SDPWS 2021 4.1.4 (φD = 0.50 on vn, was 0.80 on vs: the LRFD seismic capacity drops by about 12 %; ASD vn / 2.8 is within 1 plf of the former vs / 2.0), and the SDPWS 2015 aspect ratio rule 2bs/h now applies to wind as well as seismic and to ΣLi of a perforated wall (AWC E3.1, E3.4).

Changes from reading SDPWS 2021 (2026-11-08). Ga of three Table 4.3A rows (Structural I 7/16 in 8d plywood at 6 / 4 in, 13 / 16, were 14 / 17; Structural I 15/32 in 8d OSB at 4 in, 18, was 20; rated sheathing 15/32 in 8d plywood at 3 in, 15, was 16). Two faces combine as Kmin (Ga1 + Ga2), where wind used to add the faces and seismic took the larger of the stronger face and twice the weaker. Shear shared by capacity among several segments takes 2bs/h for segments over 2:1. Perforated walls: Co by Eq. 4.3-6 capped at 1.0, Σbi with 2bi/h, b = Σbi in the deflection, and the 2,435 plf limit on the wall's vn, one or two faces. Table 4.3A footnote 10 is an input. Entered values: seismic takes the smaller of vn and 1.4 vs.

Hand calculations of the remaining provisions, derived step by step in `backend/tests/test_verify_wood_shearwall.py`. 15/32 in Structural I, 8d at 4 in, OSB: vn = 1,205 plf, Ga = 18 kips/in (SDPWS 2021 Table 4.3A).

Example	What it checks	Hand calculation	ILX Calcs
SW-1	8 × 9 ft segment, QE = 5 kips, W = 7 kips, LRFD	v = 625 plf vs 0.50 vn = 602.5 plf; T = 7.63 kips (wind); δxe = 0.499 in, Δ = 1.99 in	625, 602.5, 7.63, 0.499, 1.99
SW-1	Same, ASD	v = 437.5 plf vs vn / 2.8 = 430.4 plf; T = 4.53 kips	437.5, 430.4, 4.53
SW-2	h/bs = 3, SPF (G = 0.42)	0.50 vn × 0.92 × 0.875 = 485 plf	485.0
SW-2b	Same, SDPWS 2015 rule	0.50 vn × 0.92 × 2bs/h = 369.5 plf; wind 0.80 vn × 0.92 × 2bs/h = 591.3 plf	369.5, 591.3
SW-3	8 ft + 3 ft by capacity, 3 ft segment at 2bs/h	8 ft share 0.800, v = 600 plf	0.800, 600
SW-3b	8 ft + 3 ft by stiffness (Δa = 0), Ga 18	3 ft share by Eq. 4.3-1	0.256 (v = 558 plf)
SW-4	Perforated, Ltot 20 ft, 8 × 5 ft opening	Co = 0.741, v = 675 plf, T = 5.40 kips	0.741, 675, 5.40
SW-5	Two faces, 4 in and 6 in nailing	vn(c) = min(1,205 / 18, 785 / 14) × 32 = 1,794 plf: seismic 0.50 × 1,794 = 897 plf; wind 0.80 × 1,794 = 1,435 plf	897, 1,435
SW-6	Story above (QE = 3 kips at 19 ft)	T = (75 − 7.84) / 7.5 = 8.95 kips	8.95
SW-7	Perforated, 12 ft segment in 20 ft, 4 × 4 ft opening, W = 6 kips	Co = 180 / 156 = 1.15, capped at 1.0; v = 500 plf; T = 4.5 kips	1.000, 500, 4.50
SW-8	Perforated, 6 + 3 ft segments, h = 8 ft	Co = 160 / 192 = 0.833; Σbi = 6 + 2.25 = 8.25 ft	0.833, 8.25
SW-9	10d at 6 in, hold-down on the inside face	0.80 × 0.92 × 950 = 699 plf	699.2

Force transfer around openings. Published: APA Technical Note T555C, *Design for Force Transfer Around Openings (FTAO)* (March 2024), and WoodWorks, *FTAO Analysis for Complex Shear Wall Openings* (Malone, May 2023).

Example	What it checks	Published value	ILX Calcs
T555C-2024	pp. 7-12: 26 ft wall, 8 ft, windows 6 and 2 ft, 0.7E = 3,750 lb, 15/32 in. sheathing, 8d at 4 in.	H 1,538 lb; 288 plf; corner forces 865, 865, 308, 269 lb; piers 337, 388, 244 plf; 120, 95, 167 plf above; 380 plf capacity; pier deflections 0.680, 0.218, 0.252, 0.252, 0.170, 0.542 in.; 0.352 in. total, with the APA deflection option	Within tolerance. Two differences recorded: T555C's four-term 0.329 in. (ILX has the three-term equation) and its printed percent drift
MALONE-1	Slides 23-32: V = 4,500 lb, 9 ft wall, two openings 3 × 3 and 5 × 5 ft	Fo 2,076.9 lb; vv 346.2, 519.2 plf; corner forces 692.4, 346.2, 690.6, 1,905.5 lb; piers 346.2, 499.7, 384.8 plf; above the head 173.1, 18.7, 38.3 plf; 326.6 plf between the sills	Within tolerance: T3 is 1.333 ft here, 1.33 there, so 692.3 and 1,903.8 lb
MALONE-2	Slides 33-41: the same wall, W = 150 plf factored	614.4, 3,539.4 lb; vv 202.4, 277.4, 491.1, 678.6 plf; corner forces 454.8, 264.9, 686.3, 2,238 lb	Within tolerance
T555-MALONE	Slide 20: APA's T555 example as Malone reproduces it	Piers 336.3, 387.7, 243.5 plf; 288 plf; corner forces 865, 865, 308, 269 lb; above 120.1, 94.4, 166.7 plf	Within tolerance

The slide 20 labels read V = 4,500 lb and Fo = 2,076.9 lb, the values of Example 1; T555C confirms V = 3,750 lb and an 8 ft wall, which is what is entered. Closed-form equilibrium is tested too: the piers carry V, straps add to the vertical shear above and below the opening, and the overturning balances.

CHAPTER 58

Wood diaphragm

A wood structural panel diaphragm to AWC SDPWS 2021 Section 4.2, under a uniform wind or seismic load, with the ASCE 7-22 load factors: one span between two shear wall lines, several spans, or a cantilever with its open-front limits. Unit shear, chord force, collector (drag) force, openings, aspect ratio, deflection and flexibility.

> **Source note.** SDPWS 2021 (Chapter 4) was read against this module on 2026-11-08: every Table 4.2A / 4.2C vn it holds, the design factors (4.1.4), the aspect ratios (Table 4.2.2), the deflection equations (Table 4.2.3), the open-front rules (4.2.6) and the construction rules (4.2.7, 4.2.8). The apparent shear stiffness Ga is an input (transcribe it from Table 4.2A, 4.2B or 4.2C).

Inputs of the wood diaphragm

- **Sheathing and nailing:** panel grade, thickness and nail; blocked or unblocked; for blocked diaphragms the nailing column of Table 4.2A (boundary 6 / 4 / 2-1/2 / 2 in with the other panel edges at 6 / 6 / 4 / 3 in; the boundary spacing also applies along continuous panel edges in cases 3 and 4 and along all panel edges in cases 5 and 6); the layout case 1 to 6 (unblocked: case 1 or cases 2 to 6 of Table 4.2C); the framing width at panel edges (2 or 3 in nominal); the framing species (species factor $1 - (0.5 - G) \le 1.0$, table footnote 2). **Entered** replaces the table: type the one vn SDPWS 2021 prints; the second box takes a seismic vs from an SDPWS 2015 table or report only, and seismic uses the smaller of vn and 1.4 vs. Ga (kips/in) is entered from the table's OSB or plywood column; 0 skips the deflection.
- **Diaphragm:** the span L between the shear wall lines and the depth b between the chords; the length of the shear wall on each support line and its offset from the diaphragm edge (shorter than b: a collector drags the shear into the wall).
- **Support:** one span (the default; a calculation saved before this choice existed reads the same). **Several spans:** up to 12, with your judgment on the model: each span simple and tributary to the wall lines (a flexible diaphragm, the usual), or continuous on rigid wall lines (HUD Example 6.4's model; the middle line takes more, and the example itself warns how much this depends on that assumption). **Cantilever:** one wall line and the far edge free, with a load at the free edge as well as the uniform load.
- **Open front (cantilever):** on by default for seismic load: the L/W limit and the 35 ft limit of SDPWS 2021 4.2.6.1. Torsionally irregular is on by default, the stricter limit (1:1 one story, 0.67:1 above); turn it off, 1.5:1, when your analysis shows no Type 1 irregularity. Enter the elastic drift at the free edge from the walls, with torsion, from your rigid diaphragm analysis to check the drift of 4.2.6.1; the added unit shear from torsion also comes from there. The amplification of the diaphragm's own deflection is an input, 3 by SDPWS Commentary's example (WoodWorks calls Cd or the overstrength factor both acceptable).
- **Openings:** up to 6, each by its position and size and the transfer length lt on each side, where the ties run past the opening. Strips of at least 1 ft.
- **High-load diaphragms:** choose High-load and enter the vn of Table 4.2B (used for wind and seismic); the table is not reproduced here, its construction rules (4.2.8.1.2) are in a note.
- **Roof slope:** nothing to enter. Enter the span and depth of the diaphragm and the load along it; the table capacities and the aspect limits do not depend on the slope (Table 4.2.2 covers flat and sloped diaphragms).

- **Loads:** the uniform wind and seismic loads along the span (strength level, or ASD level 0.6W / 0.7E); the seismic load comes from Fpx (ASCE 7-22 12.10.1.1) and ρ = 1.0 for diaphragms (12.3.4.1). Ω0 for collectors is an input (12.10.2.1; 1.0 where the light-frame exception applies).
- **Chords and deflection:** chord E and area, the chord splice positions (each splice taken in both chords) and the splice slip Δc, optional chord and collector capacities (in the method's terms), the serviceability wind factor and an optional deflection limit (SDPWS 4.2.1 leaves the permissible deflection to the engineer). ADVE, the average drift of the adjoining walls, gives the flexibility classification.

Auto-design (editor toolbar) varies the sheathing and nail (table row), boundary nailing and blocking. Objectives: lowest cost index (sheathing + nails + blocking). Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Aspect ratio L/b	SDPWS 2021 4.2.2, Table 4.2.2: 4:1 blocked, 3:1 unblocked
Unit shear, wind and seismic	v = wL / 2b against the unified nominal capacity vn (Tables 4.2A / 4.2C) × species factor: wind φD vn with φD = 0.80 or vn / 2.0, seismic φD vn with φD = 0.50 or vn / 2.8 (SDPWS 2021 4.1.4, 4.2.4)
Chord force	T = C = M / b with M = wL² / 8, against the entered capacity or reported
Collector force	Along each support line: F(y) = R y / b − (R / Lw) × wall length in 0 to y, largest value × Ω0 (seismic); against the entered capacity or reported
Deflection	Table 4.2.3, Eq. 4.2-1: 5vL³ / (8EAb) + 0.25vL / (1000 Ga) + Σ(x Δc) / 2b, under the strength-level seismic load and the serviceability wind
Cantilever	v = (wL + P) / b, M = wL² / 2 + PL at the wall line; deflection at the free edge by Eqs. 4.2-2 and 4.2-3 (Table 4.2.3); splices by Σ(x' Δc) / b over both chords, x' to the free edge
Open front	SDPWS 4.2.6.1 for seismic: L/W, L at most 35 ft, model note, drift Δ = (Cd δwalls + Cdiaph δdiaph) / Ie at the free edge against the allowable (ASCE 7-22 Table 12.12-1)
Several spans	Largest shear and moment; reactions per wall line; collector with the largest reaction; deflection by Eq. 4.2-1 span by span, a continuous span taken as simple (conservative)
Flexibility	ASCE 7-22 12.3.1, 12.3.1.3: flexible by calculation when the deflection exceeds 2 ADVE; SDPWS 4.1.7: rigid otherwise; 12.3.1.1 stated as a note
Openings	ASCE 7-22 12.10.1, rational analysis: strips share the shear at the opening's centre by depth, vs = V / (d1 + d2); tie F = vs lo / 2 at the corners; chord M / b + F at the edge of the opening; transfer zone: R = F do / b at the chords, added shear max(F − R, R) / lt, on the larger basic shear, conservative
Detailing notes	Nail penetration and edge distance, intermediate nailing, boundary nailing by case, 3 in framing with staggered nails at adjoining panel edges at 2-1/2 in or closer or 10d at 3 in (4.2.8.1.1); an advisory where 2 in framing is chosen with 2-1/2 or 2 in nailing in cases 3 to 6

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows the diaphragm plan with the span running down the sheet: the shear walls along the two support lines (their length and offset as entered), the chords along the other two edges with ticks at the entered splices, the wind and seismic loads on the loaded edge, the sheathing and nailing callout, the unit shear at the supports, the largest chord force and, when the wall is shorter than the depth, the collector force; the span, splice positions, wall and depth are dimensioned. For the governing

case it adds the unit shear and chord force diagrams along the span and the collector force along the support line. The chord and collector members, splice details, panel layout, framing and the shear walls themselves are not part of the calculation; the sheet notes say so.

Limitations

- Uniform load on the same depth over all spans; concentrated loads only at a cantilever's free edge. Offsets and re-entrant corners are not covered.
- The torsion of a cantilever and its distribution to the walls come from your rigid diaphragm analysis (the torsion calculation); the rules for a torsionally irregular structure that is not open front (4.2.5.1: L/W at most 1.5:1 with wood structural panels, drift at each edge) are stated in a note, not checked. The open-front limits are applied also to a cantilever of 6 ft or less, which 4.2.6.1 exempts. Chord forces from walls along the chord lines are not included (the WoodWorks example shows about 365 lb in its case).
- Openings: no published free numbers; the method is hand-checked statics and the strips and zones follow Diekmann and Malone's reasoning, and two openings at one section are refused. Malone's transfer diaphragm is a judgment: ties and zone shears depend on lt.
- Table 4.2B values are entered, not tabulated. Lumber sheathing (4.2.8.2 to 4.2.8.4, Table 4.2D) is not offered.
- The chords are the diaphragm edges (b between them); chord and collector members and their splices are checked only against an entered capacity.
- The distribution to the walls is not done here.
- Read against SDPWS 2021 on 2026-11-08: every vn of Tables 4.2A and 4.2C held here, their footnotes, 4.1.4, Tables 4.2.2 and 4.2.3, 4.2.5, 4.2.6, 4.2.7 and 4.2.8.1. That reading corrected rated sheathing 5/16 in 6d (blocked vn 475 / 630 / 940 / 1,065 plf with 2 in framing and 530 / 700 / 1,065 / 1,205 with 3 in, were 460 / 600 / 910 / 1,035 and 520 / 700 / 1,050 / 1,175; unblocked case 1 with 3 in framing 475, was 460) and rated sheathing 15/32 in 10d unblocked case 1 with 3 in framing (810, was 800): all were low, on the safe side. The cantilever splice term now counts each listed splice in both chords, as the simple span does (it was half). Entered values: seismic takes the smaller of vn and 1.4 vs; High-load takes its one vn for both.
- From NDS 2024 (on disk) the module takes only the framing specific gravity G (Table 12.3.3A; Redwood corrected to G = 0.37 at verification, which lowers its species factor from 0.94 to 0.87) and the common nail dimensions (Table L4); both are confirmed. NDS 2024 does not contain the SDPWS provisions.

Verification examples

Verification status: 71 values compared in 19 examples (13 published, 6 hand calculations from the cited equations): 71 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples, all free from WoodWorks (research/sources/design_examples): *Introduction to Wood: Structural Lateral Framing Design* (January 2025 slides, SDPWS 2021), *Wood Diaphragm Deflections and Flexibility* (March 2024 slides, SDPWS 2021) and *Cantilever Wood Diaphragm Webinar Series Part 3* (slide 16 reproduces SDPWS 2015 Table 4.2A). The Cantilever Webinar slides' cantilever is compared on capacities only. The full cantilever is compared with *Design Example of a Cantilever Wood Diaphragm* (WoodWorks, May 2026), and two spans with HUD's *Residential Structural Design Guide*, Example 6.4.

Example	What it checks	Published value	ILX Calcs
WW25-DIA	36 × 24 ft, unblocked 3/8 in Structural I, 8d at 6 in, ASD wind 250 / seismic 139 plf: v, capacity, M, T	188 / 104 plf, 253 / 180 plf, 40,500 / 22,518 lb-ft, 1,688 / 938 lb	187.5 / 104.2, 252.5 / 180.4, 40,500 / 22,518, 1,688 / 938
WW25-DIA	Eq. 4.2-1 under wind v = 188 plf, A = 8.25 in², Ga = 6.0, splice at 12 ft, Δc = 0.05 in	0.017 + 0.282 + 0.025 = 0.324 in	0.017 + 0.281 + 0.025 = 0.324
WW24-DIA	19/32 in sheathing, 10d at 6 in, blocked: vn, vn / 2.8	895, 319 plf	895, 319.6
WWC3-DIA	15/32 in sheathing, 10d, blocked 6 / 6 in, 2 in: vw = vn, ASD seismic	810, 290 plf (0.5 × 580)	810, 289.3
WWC3-T4.2A	Same panel, other nailing columns and 3 in framing: vw = vn	1,080, 1,610, 1,835; 910, 1,205, 1,820, 2,060 plf	same

| WWDE-ASD | Right cantilever 35 × 40 ft, ASD, Method 2B forces: 185.63 plf and 553.2 lb at the free edge | 176.3 plf, 200.5 plf with 24.3 of torsion, 289.3 plf, chord 3,327 lb, L/W 0.875 | 176.3, 200.6, 289.3, 3,327, 0.875 | | WWDE-STR | Same, strength level, v = 236 plf: Eq. 4.2-2, flexibility, drift | 0.027 + 0.165 + 0.075 = 0.267 in; 2 Δ = 0.428 in; Δ = 2.26 in, limit 2.4 in | same | | HUD-6.4 | Two 24 ft spans continuous, 24 ft deep, 200 plf | V 3,000 lb, v 125 plf, middle line 6,000 lb, M 14,400 ft-lb, chord 600 lb | same |

The example's forces come from its rigid diaphragm analysis, entered here as loads. Its v = 236 plf is applied as printed (w = 236 × 40 / 35), and its 0.075 in of slip, from its spreadsheet, is entered as one splice at the support, with half the slip because each listed splice counts in both chords.

The seismic capacity follows SDPWS 2021 4.1.4 since 2026-11-01 (φD = 0.50 on vn, was 0.80 on vs: the LRFD seismic capacity drops by about 12 %; ASD vn / 2.8 is within 1 plf of the former vs / 2.0).

Hand calculations derived step by step in `backend/tests/test_verify_wood_diaphragm.py` (15/32 in sheathing, 10d, blocked 6 / 6 in, 2 in framing: vn = 810 plf, SDPWS 2021 Table 4.2A).

Example	What it checks	Hand calculation	ILX Calcs
DIA-1	60 × 40 ft, seismic w = 300 plf, wind 400 plf, LRFD	v = 225 plf vs 0.50 × 810 = 405 plf; wind 300 vs 648 plf; M = 135 kip-ft, T = 3.375 kips	225, 405, 300, 648, 135, 3.375
DIA-1	Deflection, Ga = 10 kips/in, splices at 20 and 40 ft, Δc = 0.03 in	0.029 + 0.338 + 0.030 = 0.396 in	0.396
DIA-1	ASD	v = 157.5 plf vs 810 / 2.8 = 289.3 plf; T = 2.36 kips	157.5, 289.3, 2.36
DIA-2	25 ft wall on a 40 ft line, Ω0 = 2.5	F = 225 × 15 = 3.375 kips, × Ω0 = 8.44 kips	3.375, 8.44
DIA-3	Unblocked, 3 in framing, case 3, Hem-Fir	0.50 vn × 0.93 = 0.50 × 600 × 0.93 = 279 plf	279.0
DIA-CANT	Cantilever 30 ft, 200 plf and 2 kips	V = 8 kips, v = 200 plf, M = 150 kip-ft, T = 3.75 kips, Eqs. 4.2-2, 4.2-3 terms	same
DIA-TRIB	Spans 20 + 40 ft, each simple, 300 plf	V = 6 kips, middle line 9 kips, M = 60 kip-ft	same
DIA-OPEN	8 × 8 ft opening in the 60 × 40 ft span	V = 1.8 kips, vs = 56.25 plf, F = 225 lb, chord 3.585 kips, v net = 127.5 plf	same

No free published example of a diaphragm opening with numbers was found, nor of Table 4.2B; the WoodWorks Malone papers describe the method only.

CHAPTER 59

Wood fastener connection

Bolts, lag screws, wood screws, nails and dowels in wood to NDS 2024 Chapters 11 and 12: the yield limit equations, withdrawal, head pull-through, combined lateral and withdrawal loads, the adjustment factors and the spacing, end and edge distance requirements, for a group of fasteners in rows.

> **Source note.** Every provision, equation and tabulated value of this module was checked against ANSI/AWC NDS-2024 (with Commentary); the steps cite the section, table or equation and the printed page. The results are compared with 116 values of the published NDS 2024 design value tables (below).

Inputs of the wood connection

- **Fastener:** the type; bolt or dowel diameter (3/8 to 1 in), lag screw diameter (1/4 to 1 in), wood screw gauge (No. 6 to 24) or nail (common, box, sinker, Table L4); the length of lag and wood screws and the thread length (0 = the standard minimum: lag screws the lesser of 6 in and L/2 + 1/2 in, Table L2; cut thread wood screws 2L/3, rolled thread the greater of 4D and 2L/3, Table L3); the lag screw body (reduced body, the basis of Tables 12J and 12K, or full body) and the wood screw thread (rolled, the basis of Tables 12L and 12M, or cut). The diameter in the yield equations follows 12.3.7: D for nails, bolts and dowels; the root diameter Dr for reduced body and rolled thread screws; for full-body screws D only where the thread bearing length in the main member is at most 1/4 of lm (12.3.7.2), otherwise Dr. Fyb is the NDS default for the fastener (Table I1) or entered.
- **Members:** single shear (two members) or double shear (three members, bolts and dowels); wood or steel side members; species of the main and side members (G, NDS Table 12.3.3A); thicknesses; the load angle to grain in each member (Fe∥ = 11,200 G, Fe⊥ = 6,100 G^1.45 / √D, Hankinson between; Fe = 16,600 G^1.84 below 1/4 in); steel Fu (Fe = 1.5 Fu hot rolled, 1.375 Fu cold formed, Appendix I.2); the member widths and E for the group action factor. Dowel bearing lengths: lm = p − E/2 where the penetration p includes the tapered tip E (12.3.5.3; E from Table L2 for lag screws, 2D for wood screws and nails); toe-nails lm = L cos 30° − L/3 and ls = the lesser of ts and L/3 (12.3.9.2, Commentary C12.5.3).
- **Fastener pattern:** rows, fasteners per row, spacing in a row and between rows, end distance (member in tension or compression), the loaded and the opposite edge distance.
- **Load:** the total connection load P (factored for LRFD, the ASD combination for ASD), its angle α to the wood surface (0 lateral, 90 withdrawal) and the load duration (CD for ASD, λ for LRFD).
- **Service conditions and factors:** moisture at fabrication and in service (Table 11.3.3: dry / dry, fabricated wet / in service dry with the exception of footnote 2, fabricated dry / in service wet, wet / wet), temperature (Table 11.3.4), end grain (Ceg; with D ≥ 1/4 in the main member takes Fe⊥, 12.3.3.4) and toe-nails (Ctn; CM does not apply to toe-nails in withdrawal, 12.5.3.1). NDS 2024 has no diaphragm factor Cdi (NDS 2018 12.5.3 had Cdi = 1.1; it is not in NDS 2024 Table 11.3.1).

Auto-design (editor toolbar) varies the fastener size of the chosen type (bolt, lag, screw or nail) and the number per row and of rows. Objectives: least fastener steel; fewest fasteners, then least steel; lowest cost index. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Lateral design value	Z = least of the yield modes Im, Is, II, IIIm, IIIs, IV (double shear Im, Is, IIIs, IV), Table 12.3.1A with Rd of Table 12.3.1B; Z' = Z × CD (or KF φz λ, KF = 3.32, φz = 0.65, Tables N1 to N3) × CM × Ct × Cg × CΔ × Ceg × Ctn (Table 11.3.1); drift pins (dowels) 75 % of a common bolt (12.3.10); n Z' against P cos α
Withdrawal design value	W = 1800 G^1.5 D^0.75 (lag, Eq. 12.2-1), 2850 G² D (wood screw, Eq. 12.2-2), 1380 G^2.5 D (smooth shank nail, Eq. 12.2-3) per inch of (thread) penetration, the lag screw tip excluded; adjusted; n W'p against P sin α
Head pull-through	Nails and wood screws through a wood side member: WH = 690 π DH G² tns (tns ≤ 2.5 DH) or 1725 π DH² G² (12.2.5, Eqs. 12.2-6a/b), DH from Tables L3 / L4, × CD (KF φz λ) × CM × Ct; n W'H against P sin α
Combined lateral and withdrawal	Lag and wood screws: Z'α = W'p Z' / (W'p cos²α + Z' sin²α) (Eq. 12.4-1); nails: Z'α = W'p Z' / (W'p cos α + Z' sin α) (Eq. 12.4-2); n Z'α against P
End distance, spacing in a row	Tables 12.5.1A and 12.5.1B: below the full value CΔ = actual / full, below the minimum not permitted (D ≥ 1/4 in)
Edge distance, spacing between rows	Tables 12.5.1C and 12.5.1D (minimums); outer rows more than 5 in apart perpendicular to grain are flagged (12.5.1.3)
Lag screws in withdrawal only	Table 12.5.1E: edge 1.5D, end 4D, spacing 4D
Penetration	Lag screws 4D without the tapered tip (12.1.4.6), wood screws (12.1.5.6) and nails (12.1.6.4) 6D including the tip
Group action	Cg by Eq. 11.3-1 for D ≥ 1/4 in (γ = 180,000 D^1.5 wood to wood, 270,000 D^1.5 wood to metal)
Notes	Small fasteners (splitting), local stresses (Appendix E), steel side plate and fastener steel, screw root tension (12.2.1.4, 12.2.2.5)

For a member loaded at an angle to grain (0° < θ < 90°) the parallel and the perpendicular geometry requirements are both applied (the more restrictive governs).

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows the fastener layout on the main member near its end, with the loaded edge at the top: the rows and fasteners at the designed end distance, spacing, edge distances and row spacing (equal spacings as one dimension, 3 @ 4" = 1'-0"), the fastener callout and the load P at its angle to grain; and the section across the members through a fastener column: the side member(s) or steel side plate and the main member with their thicknesses, the fasteners at the rows (a lag screw, wood screw or nail at its length with its penetration into the main member). Washers, the steel plate design and its holes, net section and row / group tear-out are not part of the calculation; the sheet notes say so.

Limitations

- One fastener type, one or two shear planes; lag screws, wood screws and nails in single shear only.
- Not checked: the steel side plate (bearing, tear-out, net section) and the steel strength of the fastener (AISC 360), local stresses in the wood members (NDS 11.1.2, Appendix E: net section, row and group tear-out), eccentric connections, split rings, shear plates, timber rivets, structural screws with evaluation reports.

- Smooth shank carbon steel nails only (no stainless, ring shank, post-frame or metal hardware nails), no wood structural panel or CLT members (Table 12.3.3B, CLT narrow edge), no connections with four or more members (12.3.8) and no load at an angle to the fastener axis other than 90° (12.3.9.1).
- Rows are taken as identical (n = rows × per row) with the same Cg.
- The same end and edge distances apply to the main and the wood side member.
- Pull-through uses the side member thickness as the net thickness tns (enter a reduced ts for countersunk heads).
- Perpendicular to grain the spacing for CΔ = 1.0 is taken as 4D (Table 12.5.1B gives "required spacing for attached members").

Verification examples

Verification status: 232 values compared in 152 examples (143 published, 9 hand calculations from the cited equations): 232 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

The reference values are compared with the published NDS 2024 design value tables (inputs per the table footnotes: one fastener, all factors 1.0, penetration 8D for lag screws and 10D for wood screws and nails, reduced body lag screws and rolled thread wood screws with D = Dr, A36 plates of 1/4 in and A653 Grade 33 below). The Z tables are rounded to 10 lb; every value agrees within 5 lb or 2 % (`backend/tests/test_verify_wood_connection.py`).

Table (NDS 2024, page)	Fastener and connection	Values compared
12A (pp. 101-102)	Bolts, single shear, wood side member; G = 0.37 to 0.67, Z∥, Zs⊥, Zm⊥, Z⊥	31
12B (p. 103)	Bolts, 1/4 in A36 side plate	10
12F (p. 107)	Bolts, double shear	14
12J (p. 112)	Lag screws, wood side member	17
12K (p. 114)	Lag screws, steel side plate (gage and 1/4 in)	9
12L, 12M (pp. 115-116)	Wood screws, wood and steel side members	10
12N, 12P (pp. 117-118)	Common, box and sinker nails, wood and steel side members	14
12.2A to 12.2C (pp. 85-87)	Withdrawal per inch: lag screws, wood screws, nails	9
12.2F (p. 90)	Head pull-through	2
11.3.6A, 11.3.6C (PDF pp. 87-88)	Group action factor Cg, 1 in bolts at 4 in: wood side members (As 5 to 64 in², As/Am 0.5 and 1) and steel side plates (Am 5 to 64 in², Am/As 12); two-decimal tables, within 0.005	19

Published worked examples, mode by mode, from free documents (research/sources/references and design_examples): **AWC Technical Report 12** (General Dowel Equations, 2015) and the **AWC Structural Wood Design Examples 2015/2018** E2.2 and E2.5 as quoted on the free Trimble Tekla Tedds 2026 verification pages. Both take Fe from NDS Table 12.3.3 (rounded to 50 psi); the module computes Fe from G (Table 12.3.3 footnote), so the modes differ by up to 0.6 %.

Example	What it checks	Published (Im / Is / II / IIIm / IIIs / IV; Z) lb	ILX Calcs
TR12-3.1	1/2 in bolt, 1-1/2 in Hem-Fir members, single shear, no gap: main ∥ / side ∥	900 / 900 / 414 / 550 / 550 / 663; 414	903 / 903 / 416 / 551 / 551 / 664; 416
TR12-3.1	main ∥ / side ⊥	720 / 383 / 250 / 380 / 324 / 442; 250	722 / 381 / 250 / 380 / 324 / 441; 250
TR12-3.1	main ⊥ / side ⊥	383 / 383 / 176 / 289 / 289 / 387; 176	381 / 381 / 175 / 288 / 288 / 386; 175
TR12-3.2	3/8 in lag screw, Case 1 (NDS basis, Dr = 0.265 in), ls 1-1/2, lm 3 in, DF-L, side ∥	1,113 / 557 / 420 / 478 / 260 / 201; 201	1,113 / 557 / 420 / 478 / 260 / 201; 201
TR12-3.2	side ⊥	890 / 290 / 304 / 353 / 153 / 143; 143	890 / 290 / 304 / 353 / 153 / 143; 143
TR12-3.4	8d nail through 16 gage A653 Gr. 33 steel (NDS case)	IIIs 97	96.4
AWC-E2.5	1/2 in bolt, Hem-Fir 4x to 4x	Fe 4,800 psi; 2,100 / 2,100 / 966 / 957 / 957 / 663; 663	4,816; 2,107 / 2,107 / 970 / 960 / 960 / 664; 664
AWC-E2.2	10d common nail, DF-L 1x side into 3x main	658 / 235 / 219 / 230 / 105 / 118; 105	656 / 234 / 219 / 229 / 105 / 117; 105

The equations of the yield modes, the adjustment factors and the combined loading are also verified against hand calculations derived step by step (every mode of each case is compared):

Example	What it checks	Hand calculation	ILX Calcs
NDS-B1	1/2 in bolt, single shear, DF-L 1-1/2 to 3-1/2 in, parallel	Im 2,450, Is 1,050, II 913, IIIm 1,103, IIIs 615, IV 716 lb; Z = 615 lb (NDS 2024 Table 12A: 610 lb)	all six within 0.1 %
NDS-B1	LRFD (λ = 0.8)	Z' = 615 × 3.32 × 0.65 × 0.8 = 1,061 lb	1,061
NDS-B2	3/4 in bolt, double shear, 1/4 in A36 plates, perpendicular	Fem = 2,578 psi, Fes = 87,000 psi; Z = 1,354 lb (mode Im)	2,578, 87,000, 1,354
NDS-B3	5/8 in bolt at 45°, Southern Pine	Feθ = 4,249 psi; IIIs 766 lb, IV 942 lb	4,249, 766, 942
NDS-L1	1/2 × 6 in reduced body lag screw (D = Dr = 0.371 in, lm = p − E/2)	IIIs 400, IV 394 lb; Z = 394 lb; W = 378 lb/in; W'p = 1,206 lb; Z'α (30°) = 474 lb	394, 378, 1,206, 474
NDS-N1	16d common nail, 1-1/2 in DF-L side (lm = p − D)	Fe = 4,637 psi; Z = 141 lb, mode IV (NDS 2024 Table 12N: 141 lb)	4,637, 140.7
NDS-N2	16d nail in withdrawal	W = 39.5 lb/in, W'p = 79.0 lb	39.5, 79.0
NDS-N3	16d nail at 40° (Eq. 12.4-2)	Z'α = 73.7 lb	73.7
NDS-S1	No. 10 rolled thread wood screw (D = Dr)	Z = 106 lb; W = 135 lb/in; W'p = 226 lb; W'H = 148 lb	106.3, 135.4, 225.6, 147.5
NDS-G1	Four 3/4 in bolts in a row, wet, 125 °F, CD = 1.6	Cg = 0.977, CΔ = 0.714, Z = 2,404 lb, n Z' = 5,262 lb	0.977, 0.714, 2,404, 5,262

Corrections made when the module was checked against NDS 2024: Redwood G = 0.37 (was 0.44); CM = 0.7 for fasteners under 1/4 in fabricated wet and in service dry (was 1.0); nail withdrawal CM 1.0 when fabricated and in service wet and no CM on toe-nails in withdrawal; the diaphragm factor Cdi removed; lag and wood screws use Dr per 12.3.7 (the default lag screw used the shank D, which overstated Z by up to about 50 %); lm = p − E/2 (was p −

E for lag screws and p for nails and wood screws); the lag screw minimum penetration excludes the tip; the lag screw thread length is capped at 6 in; toe-nail bearing lengths; drift pins at 75 %; nail combined loading by Eq. 12.4-2; head pull-through and the Table 12.5.1E checks added.

CHAPTER 60

Wood bearing and notches

A reaction bearing on a sawn lumber or glulam member, perpendicular, parallel or at an angle to the grain, and a member end notched at its support, to NDS 2024 3.4.3, 3.10, 4.4.3 and 5.4 with ASCE 7-22 combinations, in ASD or LRFD.

Inputs of the bearing check

- **Member:** sawn lumber (species, grade, size, or Fv, Fc⊥ and Fc entered) or glulam (Table 5A stress class, b, d).
- **Bearing:** the reaction per load type, the direction (perpendicular to grain for a beam on a support, parallel to grain for end-grain bearing, or at an angle θ), the bearing length lb and width (0 = the member width), whether the bearing is at the member end (Cb = 1.0 within 3 in of the end, NDS 3.10.4), the deformation limit (0.02 in with 0.73 Fc⊥, NDS 4.2.6) and, for end-grain bearing, whether a metal bearing plate is provided.
- **Notch:** none, on the tension face (bottom) or on the compression face (top) at the support, the depth remaining dn and, for a compression-face notch, the distance e the notch extends past the inner edge of the support.

Checks performed

Check	Basis
Bearing perpendicular to grain	$f_{c\perp} = R/(b_w l_b) \le F_{c\perp}' = F_{c\perp}\, C_M C_t C_i C_b$ (3.10.2)
Bearing parallel to grain	$f_c \le F_c^*$ (Fc with all factors except CP, 3.10.1); a metal plate where $f_c > 0.75\, F_c^*$ (3.10.1.3)
Bearing at an angle	Hankinson, $F_\theta' = F_c^* F_{c\perp}' / (F_c^* \sin^2\theta + F_{c\perp}' \cos^2\theta)$ (Eq. 3.10-1)
Shear at a tension-face notch	$V \le V_r' = (2/3)\, F_v' b\, d_n (d_n/d)^2$ (Eq. 3.4-3)
Shear at a compression-face notch	$V \le V_r' = (2/3)\, F_v' b [d - ((d - d_n)/d_n)\, e]$ for $e \le d_n$ (Eq. 3.4-5); for $e > d_n$, d_n replaces d in Eq. 3.4-2: $(2/3)\, F_v' b\, d_n$ (3.4.3.1(e))
Notch depth	Sawn lumber: d/4 on either face (4.4.3.1); glulam: tension face d/10 and 3 in (5.4.5.1), compression face 2d/5 (5.4.5.2)
Glulam compression-face notch length	$l_b + e \le 3\, d_n$ from the member end (5.4.5.2; the member end is taken at the outer edge of the bearing)

The shear at the support is taken as the reaction. Glulam notches take Cvr = 0.72 on Fvx (5.3.10, 5.4.5.4).

Drawings

The drawing sheet (Drawing sheet / DXF buttons, and the report appendix) shows the member end in elevation with the bearing length, the notch as entered (dn, e), the reaction by type at the angle to grain used by the check, and the bearing area bw x lb on the member (on the end grain for bearing parallel to grain). The support is drawn as a block the bearing length long: its material, size and fastening are not part of the calculation, nor is the bearing's distance from the end when it is not at the end.

Limitations

- Checked against the NDS 2024 text (steps cite section and page): 3.4.3.1 (Eqs. 3.4-3 and 3.4-5), 3.10.1 to 3.10.4 (Eqs. 3.10-1, 3.10-2), 4.2.6 (Eq. 4.2-1, 0.73 Fc⊥), 4.4.3.1, 5.3.10, 5.4.5. Corrected: a compression-face end notch in sawn lumber is limited to d/4 like a tension-face notch (NDS 2024 and 2018 4.4.3.1; the module had allowed 0.4 d, the glulam limit); the glulam notch provisions are 5.4.5 in NDS 2024 (5.4.4 was cited) and the 3 dn length limit of a glulam compression-face notch is now checked; Fc⊥ takes no λ in LRFD (Table 4.3.1).
- A taper cut at the end is treated by NDS 2024 3.4.3.1(c) and (e) like an end notch: on the tension face as a tension-face notch, on the compression face with dn measured at the inner edge of the support (enter it as a compression-face notch). Notches away from the supports (3.4.3.1(f), 4.4.3.2) and SCL members (8.4.1) are not covered. A compression-face notch must also stay out of the middle third of the span (advisory).
- The sawn lumber reference values (NDS 2024 Supplement Tables 4A, 4B, 4D) were read against the 2024 Supplement on 2026-11-08 and agree. The glulam values were read against Supplement Table 5A on 2026-11-12 (pp. 80-82); the bearing values Fc⊥x and Fc⊥y agree.

Verification examples

Verification status: 16 values compared in 9 examples (3 published, 6 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published values of AWC Structural Wood Design Examples 2015/2018 Example E1.3 (as tabulated in the free Tekla Tedds verification example), Breyer Ex. 6-18 / 6-19 and Aghayere & Vigil Ex. 5.9 (as tabulated in the free calcs.com validation pages), and hand calculations in closed form (`backend/tests/test_verify_wood_bearing.py`); the hand cases are a 4x10 DF-L No. 2 with D 1500 + L 2500 lb on 3 in of bearing. No free published notch example was found.

Example	What it checks	Published / hand value	ILX Calcs
AWC-E1.3	5 × 22 24F-1.8E glulam end bearing, 6 in	R = 12,988 lb, Fc⊥' = 650 psi, fc⊥ = 433 psi	12,988, 650, 432.9
BREYER-6-18	2x6 DF-L No. 1 rafter end bearing	Fc⊥' = 625 psi (ASD), 938 psi (LRFD)	625, 939.4
AV-5.9	2x6 stud on a DF-L No. 2 plate, no Cb	P = 3000 lb, Fc⊥' = 625 psi	3000, 625
HAND-1	Perpendicular at the end	Fc⊥' = 625 psi (ASD), 939.4 psi (LRFD, no λ)	same
HAND-2	Perpendicular away from the end	Cb = 1.125, Fc⊥' = 703.1 psi	same
HAND-3	At 45°	Fθ' = 924.7 psi	same
HAND-4	Parallel to grain	Fc* = 1350 psi	same

Example	What it checks	Published / hand value	ILX Calcs
HAND-5	Tension-face notch, dn = 7.5 in	Vr' = 2070.9 lb	same
HAND-6	Compression-face notch, dn = 7.5 in, e = 2 in	Vr' = 3689.0 lb	same

CHAPTER 61

Flitch-plate beam

A flitch-plate beam: two sawn lumber plies with one steel plate between them, or three plies with two plates, bolted through. The wood and its bolts are designed to NDS 2024, the plates to AISC 360-22, with ASCE 7-22 load combinations in ASD or LRFD. The parts share the load in proportion to their stiffness; the bolts that carry the plate's share from the wood (and the reaction share at the supports) are designed too.

Inputs of the flitch-plate beam

- **Wood plies:** the layup (one plate between two plies, two plates between three plies), species and grade, ply size (dimension lumber 2" to 4" thick; every ply the same), or *Other* with reference values you enter (including the size factor) and the specific gravity G used for the bolt bearing strength (NDS Table 12.3.3A). The self-weight uses the wood density entered and 490 pcf for the steel.
- **Steel plates:** thickness and depth (equal to or less than the wood depth), position (flush top, centred, flush bottom), steel (ASTM A36, A572 Gr. 50 or Fy and Fu entered; E = 29,000 ksi) and the lateral bracing of the plate for AISC F11.2: *at the bolts* (Lb = the bolt spacing in a row, the conservative choice) or *continuously by the plies* in contact on both faces (DeStefano 2007: the wood braces the plate against lateral buckling). The choice is yours.
- **Length, supports and bearing:** any number of pins or fixed ends, the bearing length, and what bears at the supports: the wood plies only (the usual detail: bolts at the ends carry the plate's reaction into the wood), wood and plate together (the plate must be flush with the bottom), or the plate only (the bolts carry the wood's reaction into the plate).
- **Loads:** line loads in plf and point loads in lb by load type, and where they bear: on the wood (joists on top or hung from the plies: the plate's share passes through the bolts), on the plate, or on both together (a flush top shared by stiffness: only the self-weights pass through the bolts).
- **Bolts:** diameter, one row or two staggered rows, the spacing s in each row (two rows are staggered at s/2), the distance from the top of the wood to the top row and between the rows, the end distance, the number of bolts and their pitch at each support, the number of bolts that carry each point load (usual practice: bolts at each concentrated load; your choice), and whether special detailing for cross-grain shrinkage is provided (it waives the 5 in. limit between outer rows of NDS 12.5.1.3).
- **Lateral stability:** the unbraced length lu of the compression edge and the load case for le (NDS Table 3.3.3). CL is computed with the total wood width (the plies are bolted through the plates and act together).
- **Service conditions and factors:** wet service, temperature, incising, the repetitive member factor (not usual for a flitch beam; NDS 4.3.9 is your judgment), the shear reduction near the supports (3.4.3.1) and the bearing deformation limit.
- **Deflection and long-term load:** span / n for live and total load, creep of the wood in the total deflection (NDS 3.5.2 applied to the wood part: Itr,LT = Iw / Kcr + n Is), the sustained part of the live load, unseasoned lumber (Kcr = 2.0), and whether the steel and the bolts are designed for the long-term share: under sustained load the wood creeps and sheds load to the steel; the long-term part (D plus the sustained part of L) is then shared with the wood E divided by Kcr. Off by default; your judgment.

- **Load duration (code options):** CD per load type (ASD) and the live load type for λ (LRFD), as for the sawn lumber beam.

Auto-design (editor toolbar) varies the ply size, one or two plates, the plate thickness and the bolt diameter; for each candidate it derives the plate depth (full depth less 1/4 in., or the same fraction of the wood depth as given), the bolt rows within the plate and the edge distances, and the bolt spacing and bolt counts that carry the transfer scaled from the input as given. Species and grade are kept unless unlocked; span, supports and loads are never changed. Objectives: lightest (wood + steel weight); lowest cost index. See Auto-design.

Checks performed

Check	Basis
Load sharing	n = Es / E'w (E of the wood with CM, Ct, Ci; not Emin); each part bends about its own centroid with the same curvature: αs = Es Is / (E'w Iw + Es Is), αw = 1 − αs (DeStefano 2007; FPL Wood Handbook, transformed sections)
Wood bending, every combination	fb = αw M / Sw ≤ Fb' = Fb (CD or KF φ λ) CM Ct CL CF Ci Cr (NDS 3.3, Table 4.3.1)
Wood shear	fv = 3 Vw / (2 b d) ≤ Fv' with Vw = αw V, or the whole V where only the wood bears (the reaction passes through the plies next to the support)
Wood bearing at the supports	fc⊥ = R / (Σb lb) ≤ Fc⊥' (NDS 3.10.2, Cb), with αw R where wood and plate bear together
Plate flexure	AISC F11: Mn = Mp = Fy Z ≤ 1.5 Fy Sx (F11-1); lateral-torsional buckling by F11-3 / F11-4 with Lb = the bolt spacing and Cb = 1.0, or none when braced by the plies; φ = 0.90, Ω = 1.67; demand αs M (with the long-term share when chosen)
Plate shear	AISC G4 (doubly symmetric member): Vn = 0.6 Fy Aw Cv2, Aw = d t, Cv2 of G2.2 with kv = 5; φ = 0.90, Ω = 1.67; demand αs V, or V where only the plate bears
Plate bearing at the supports	AISC J7 (1.8 Fy Apb, φ = 0.75, Ω = 2.00) and local yielding of the plate edge (J10.2 with k = 0: Fy t lb, φ = 1.00, Ω = 1.50) where the plate bears
Bolt value	NDS 12.3 double shear, wood plies as side members and the steel plate as the main member (Fe = 1.5 Fu, Appendix I.2), wood loaded perpendicular to grain (Fe⊥, θ = 90°, Kθ = 1.25); two plates: NDS 12.3.8.2 (five members, Z = Zmin × 2). Z' = Z (CD ≤ 1.6, or KF 3.32, φz 0.65, λ ≤ 1.0) CM Ct Cg CΔ
Bolts: load transfer	q = αs w(on wood) − αw w(on plate) per unit length, each combination; force per bolt q s / rows ≤ Z'; the required spacing is reported
Bolts at the point loads	αs P / (bolts at the load) + q p ≤ Z'
Bolts at the supports	the plate's reaction share αs R (wood bears) or the wood's αw R (plate bears) over the support bolts ≤ Z'
Wood shear at the support bolts	where the plies hang on the bolts (plate bears): NDS 3.4.4, Vr' = (2/3) Fv' b de (de/d)² within 5d of the end
Bolt geometry	NDS Table 12.5.1A end distance (2D minimum, 4D full: CΔ), Table 12.5.1C edges (loaded 4D, unloaded 1.5D; the edge the bolts push the plies toward is loaded), Table 12.5.1D spacing between rows (the rows run across the beam), 12.5.1.3 5 in. between outer rows across the grain; AISC J3.5 / Table J3.4 edge and end distance in the plate, J3.4 spacing 2 2/3 d
Deflection	live (each live type alone) and total (ASD gravity combinations) with E'w Itr; creep: Δ = ΔST + (Itr / Itr,LT) ΔLT

Check	Basis
Beam slenderness	RB ≤ 50 (NDS 3.3.3.7) when the compression edge is unbraced

Drawings

The sheet shows the beam elevation (supports, loads by type, spans, the plate edges hidden), the section (plies, plates, bolts with heads and nuts, widths and depths), the bolt pattern at the beam end (end distance, the support bolts and the first transfer bolts in their rows, spacings and edge distances), and the shear, moment and deflection diagrams. Washers, hole sizes, hangers and the supports themselves are not part of the calculation.

Limitations

- Each part bends about its own centroid (curvature compatibility through the bolts); no horizontal shear connection between wood and steel is relied on, so a plate flush with the top or bottom adds no composite (parallel axis) stiffness. This is the usual practice (DeStefano 2007) and is conservative for stiffness.
- The plates run the full length; a plate only where the moment needs it is not covered.
- Dimension lumber plies only (2" to 4" thick); timbers, glulam and SCL plies are not covered (LVL plies can be entered as reference values but the SCL volume factor is not applied).
- NDS does not tabulate bolts with a steel centre plate and wood side members (Tables 12F / 12G cover wood main members); the module uses the yield limit equations of Table 12.3.1A. The reference design values are those of NDS 2024 Supplement Tables 4A, 4B and 4D, read on 2026-11-08 (as the sawn lumber beam).
- Friction between the plates and the plies is neglected (DeStefano 2007). Bolt bearing on the plate (AISC J3.11) is not checked separately: the NDS yield modes include the bearing of the plate (mode Im, Fe = 1.5 Fu).
- The plate edge local yielding check is J10.2 applied to a plate without flanges (k = 0): a module decision.

Verification examples

Verification status: 43 values compared in 9 examples (5 published, 4 hand calculations from the cited equations): 41 within tolerance, 2 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

`backend/tests/test_verify_flitch_beam.py`: the published values of DeStefano, "Flitch Plate Beams Design Guide", STRUCTURE magazine, June 2007 (Table 1, p. 56, DF-L No. 2 plies, A36 plates, E = 1,600,000 psi; the design example, p. 57), and hand calculations in closed form.

Example	What it checks	Published / hand value	ILX Calcs
DESTEFANO-T1-2x10	(2)-2x10 + PL 1/2 x 9	n = 18.1, Itr = 748 in⁴, Str = 161.7 in³, M = 13,340 lb-ft, 74 %, V = 3,330 lb	18.12, 748.4, 161.8, 13,350, 73.6 %, 3,330
DESTEFANO-T1-3x2x10	(3)-2x10 + (2) PL 1/2 x 9, Cr	Itr = 1,397, Str = 302.1, M = 28,649, 79 %, V = 4,995	1,398, 302.2, 28,680, 78.8 %, 4,995
DESTEFANO-T1-2x12	(2)-2x12 + PL 1/2 x 11	Itr = 1,361, Str = 241.9, M = 18,142, 74 %, V = 4,050	1,361, 242.0, 18,150, 73.9 %, 4,050

Example	What it checks	Published / hand value	ILX Calcs
DESTEFANO-T1-2x8	(2)-2x8 + PL 3/8 x 7	Itr = 289, Str = 79.7, M = 7,173, 67 %, V = 2,610	289.5, 79.9, 7,189, 67.1 %, 2,610
DESTEFANO-EX	12 ft, 700 plf, 1/2 in. bolts	plate load 518 plf, plate reaction 3,108 lb, Z (mode Is) = 945 lb, spacing 21.9 in., 3.29 end bolts	514.9 plf, 3,090 lb, 947 lb; 20.3 in. and 3.54 bolts (deviation: mode IIIs, 872 lb, governs)
HAND-1	Plate F11 (Lb = 24 in.) and G4	Mn = 298.0 kip-in (F11-3), Mp = 364.5 kip-in, Vn = 97.2 kips	same
HAND-1B	Plate braced by the plies	φMp = 328.1 kip-in	same
HAND-2	Bolt with two plates (12.3.8.2)	Z = 2 × 473.6 = 947.3 lb; LRFD Z' = Z × 3.32 × 0.65 × 0.8	same
HAND-3	Deflection with creep, long-term share	ΔL = 0.195 in, ΔT = 0.280 in, αs = 0.736, αs,LT = 0.807	same

Deviation DESTEFANO-2007-BOLT: the example's bolt value is yield mode Is only; with a steel centre plate mode IIIs gives 872 lb and governs, so the module's spacing is 8 % closer (conservative). The free worked examples found cover section properties and the bolt rational method; Breyer, Design of Wood Structures (flitch beam example) is not free (research/DOCUMENTS_NEEDED.md).

CHAPTER 62

CLT shear wall

A stacked cross-laminated timber shear wall of equal rocking panels with the prescribed nailed angle connectors, by SDPWS 2021 Appendix B, story by story from the top: connector shear, panel shear, hold-down strength and deformation, compression zone, deflection and drift. The default is the wall of FEMA P-2192-V1, Chapter 6. SDPWS 2021 Appendix B and 4.6 were read against this module on 2026-11-08; the Commentary equations (C-B.2 to C-B.5) are not in the copy on file and follow FEMA's use of them.

Inputs of the CLT shear wall

- **System.** Cross-laminated timber shear walls, h/bs from 2 to 4, R, Ω, Cd of 3, 3, 3; or high aspect ratio panels only, h/bs of 4, with 4, 3, 4 (ASCE 7-22 Table 12.2-1). R is not used here: forces are entered. Cd is.
- **Panels.** Panel length bs and the number in the wall; every story has the same panels.
- **Stories, top first, up to 6.** F at strength level, added at the top of that story. wD and wL on the panel; FEMA enters each story's own load, not the accumulation. Connectors at the bottom of a panel (the top matches), hold-downs at each end, what it bears on.
- **Added to h.** FEMA's text writes the panel height, but its Tables 6-6 to 6-8 only come out with the 6.875 in. floor panel above added for stories with a floor over them. So it's an input, prefilled as those tables: 0 reads h as the equation writes it, and gives less overturning. Your call.
- **CLT panel.** Thickness (at least 3.5 in., B.3.1), the layers along the height, G, Ft, Fc. In-plane Fv(tv), EIeff and GAeff are from the panel's report and the SDPWS Commentary for your panel length; they're prefilled for the example's 4 ft 9 in. panel, so **change them with the panel length**.
- **Hold-downs.** Position, capacity and deformation from the device's report, at the design method's level; the factor of B.3.4, 2.0 for seismic (B.3.4 permits 1.5 for wind; the prefilled 2.0 is on the safe side); the end of the panel developing them, 1 ft in the example.
- **Compression bearing.** Floor panel Fc⊥, thickness and E across the grain. End grain bearing takes 0.75 Fc unless on stiff metal.
- **Add the gravity of the stories above to the compression.** On by default, off in FEMA's tables. The load from the panels above reaches this end panel too, so the default is the longer zone; the hold-down never counts it, which is the conservative side there.
- **Ie, allowable drift.** 0.025 is the example's case, ASCE 7-22 Table 12.12-1: risk category II, four stories or less, finishes that accommodate drift.

Auto-design varies the connectors per panel and the hold-downs at each end, top story first, then more connectors if deflection asks for them. Panels, the device and loads stay yours.

Checks performed

Check	Basis
Panel aspect ratio, each story: 2 to 4, or 4 for high aspect panels	SDPWS 2021 B.3.1, B.3.7
Panel thickness at least 3.5 in.; at least two connectors at the top and bottom of each panel	SDPWS 2021 B.3.1, B.3.2
Wall height at most 65 ft, seismic	ASCE 7-22 Table 12.2-1
Connector shear: vs = n × 2,605 CG / bs (Eq. B-2), then 0.5 or vn / 2.8 for seismic, 0.8 or vn / 2.0 for wind	SDPWS 2021 B.5, B.6; the factors of 4.1.4.1 and 4.1.4.2, which B.3.4 names
Panel in-plane shear, and enough that the nails govern	NDS 2024 Table 10.3.1
Hold-down strength at the entered factor (2.0 seismic, 1.5 permitted for wind) times the connectors' design capacity, accumulating down	SDPWS 2021 B.3.4 item 3; Commentary Eq. C-B.2 as FEMA uses it
Hold-down deformation at most 0.185 in. per story under strength forces	SDPWS 2021 B.3.4 item 2
Panel tension over the length developing the hold-downs	NDS 2024 Table 10.3.1
Compression zone within the end panel	SDPWS 2021 B.3.5; Commentary Eqs. C-B.3 to C-B.5 as FEMA uses them
Deflection against drift limit × h × Ie / Cd	SDPWS 2021 B.4, Eq. B-1 (h the panel height), ASCE 7-22 12.8.6

Limitations of the CLT shear wall

- Walls not meeting Appendix B (the R = 1.5 route of SDPWS 4.6.3.2) are not covered, nor are diaphragms.
- Stated in a note, not checked: moisture content under 16 %, platform construction, connectors within 12 in. of each panel end, the anchorage designed for the connectors' concurrent tension and shear, and B.3.6 load path connections.
- Not checked: the panel's column stability, net sections at holes and cuts, row and group tear-out at the hold-downs, screws of the device. Use the CLT panel type for the panel in compression.
- One panel length per wall. Panels of different lengths or a wall set up at "T" intersections are not modelled.
- Appendix B applies to wind as well as seismic (4.6.3.2). B.6 and 4.6.3.1 send the design values to 4.1.1, the general requirements; the factors used are those of 4.1.4, which B.3.4 names for the hold-down: an interpretation, stated here.
- EIeff and GAeff aren't calculated; PRG 320 and SDPWS Commentary C-B.4 aren't on hand.
- Connector layout is by count: drawings do not place them.

Verification examples

Verification status: 55 values compared in 7 examples (6 published, 1 hand calculations from the cited equations): 51 within tolerance, 4 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

FEMA P-2192-V1, Chapter 6, pp. 229-246: Tables 6-4 to 6-9 by strength design, with the compression option off as FEMA has it. Differences, plainly:

- Table 6-9's shear and total at stories 2 and 1 follow the height plus the floor panel, where Eq. B-1 says panel height. ILX uses panel height: 0.319 and 0.338 in. against 0.33 and 0.35. That is about 0.01 in. on the low side of the printed value; recorded as a deviation.
- FEMA's printing has small slips, not differences in method: 20,849 plf where its factors multiply to 20,952; 17,687 and 17,678 lb for one device; 35,540 for 34,540 lb on p. 237.

Allowable stress design and wind have no published example: hand calculations. Appendix B was read on 2026-11-08; PRG 320 and the Commentary were not.

CHAPTER 63

Continuous rod tie-down

One rod run at one end of a stacked wood shear wall, restrained by a bearing plate at every level, story by story from the top: overturning and uplift, rod tension, bearing plates, compression posts, shrinkage and take-up device travel, the vertical chord deformation, wall deflection and story drift. It follows WoodWorks' Five-Story Light-Frame Wood Over Podium example; that wall is the default input.

Inputs of the rod tie-down

- **Wall length b, lateral load, SDS.** Seismic takes the vertical effect 0.2 SDS D from ASCE 7-22 12.4.2.2, so uplift uses (0.6 − 0.14 SDS)D + 0.7E (allowable) or (0.9 − 0.2 SDS)D + E (strength). Wind uses 0.6D + 0.6W or 0.9D + W. The load factor on L with the lateral load is yours (ASCE 7-22 2.3.1 exception).
- **Stories, top first, up to 12.** F, dead wD and live wL are what is added at the top of that story on this wall; wD includes the story's own wall. Lever arm d runs from the rod to the centre of the posts at the other end, per story, because the posts change. Chord length is the wall tributary to the posts for gravity. Rod length is restraint to restraint, empty for the story height. Wood across grain is the wood in the shrinkage zone at that level, for example 4.5 in. for a double 2x4 top plate and a 2x4 bottom plate, 15.75 in. for platform framing with 2x12 joists. Ga comes from SDPWS Tables 4.3A to 4.3D for the nailing you use.
- **Posts and wood plates.** Species, grade and post size; the posts do not act compositely. The plate's species sets Fc⊥ under the plates and posts.
- **Bearing plates.** Fy, and the hole sizes over the rod: WoodWorks uses 3/16 in. in the wood so the rod moves freely and 1/16 in. in the steel. Cb applies unless a plate is within 3 in. of the end of the wood member.
- **Take-up device.** On by default, as WoodWorks recommends. Travel, slack and elongation come from the evaluation report of the device you choose: **ILX has not read ICC-ES AC316 or any device's report.** The prefilled slack and elongation are the example's; the travel is a placeholder to replace.
- **Shrinkage.** S = C D (Mi − Mf) plus construction gaps, with C, the moisture contents and the gaps as inputs. The fuller formula of the Wood Handbook is not offered: WoodWorks shows it within 0.1 % of the simple one on its plates.
- **Forces for deformation.** WoodWorks departs from ASCE 7-22 12.8.6.1, whose 1.0D + 0.5L gives less uplift, and takes the strength uplift combination instead. That is the default; 1.0D + 1.0 lateral or your own factors are the alternatives.
- **Wood-on-wood crushing factor.** 1.75 by default, as the example. WoodWorks calls this the engineer's judgment: the SDPWS Commentary example leaves it out, and a factor of 1.0 gives less drift.
- **Cd, Ie, allowable drift, local limit on rod elongation.** ASCE 7-22 Tables 12.2-1, 1.5-2, 12.12-1. The limit on rod elongation is a note only: some jurisdictions set one.

Auto-design varies the rod size and the bearing plate length and thickness at each story, smallest steel first, and larger rods if drift asks for them. Posts, lever arms, plate width and grades stay yours.

Checks performed

Check	Basis
Rod tension, each story	AISC 360-22 J3, Table J3.2: 0.75 Fu on the gross area, φ 0.75, Ω 2.00
Plate bearing on the wood, each story, under the difference in tension from the story above	NDS 2024 3.10.2, 3.10.4, net of the hole
Plate bending, each story, as a cantilever from the rod on the net width, the worse direction	AISC 360-22 F11.1
Compression posts, each story, every combination with its CD or λ	NDS 2024 3.6, 3.7, 3.10; ASCE 7-22 2.3, 2.4
Take-up device travel against the accumulated movement at the top level	Device report; IBC 2304.3.3
Story drift, each story: rod elongation, plate crushing, slack and device elongation (or shrinkage), post crushing on the plates and sheathing	SDPWS 2021 Eq. 4.3-1, ASCE 7-22 12.8.6, Table 12.12-1, the curve of WoodWorks p. 43
Rod elongation against the local limit, a note	Local requirement

Limitations of the rod tie-down

- Every level has a bearing plate. A run that skips levels is not covered.
- Not designed: sheathing and edge nailing (use the wood shear wall), the anchorage at the base (use concrete anchors), couplers, which must develop the rod, the podium and its overstrength under discontinuous walls, the other end's different chord, multi-story wall effects.
- Tension is on the gross area with Fnt, as WoodWorks does. Rod elongation uses the threaded tensile area for every rod length, also conservative where a rod is not threaded through.
- Dead load resisting overturning is $w\ b^2 / 2$ on the wall length b, not the lever arm d, as the example.
- Compression for crushing is the lateral force only, as the example says; gravity preload is left out. WoodWorks calls this somewhat unconservative for the crushing itself and offsets it elsewhere.
- Deformation curves are limit-state values, not adjusted for load duration.

Verification examples

Verification status: 186 values compared in 13 examples (11 published, 2 hand calculations from the cited equations): 185 within tolerance, 1 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

WoodWorks DE-2025, 2nd Ed., pp. 14-15, 31-50: Tables 2.1, 3.6, 3.8 to 3.19 for its 29 ft wall, allowable stress and the drift forces. One value differs, plainly: Table 3.17B prints 0.466 in. at the 4th level, where its own columns give 0.446 in. and its accumulated column follows 0.446 in. On p. 15 the prose says 0.063 in. per floor of shrinkage where its own p. 14 and Table 2.1 use 0.090 in.; ILX follows the table.

The strength side has **no published example**: hand calculations of the top story for strength design and of wind for allowable stress. The comparison with a device's report, the checks of a real device's travel, and anything on AC316 are not done.

CHAPTER 64

Wood ledger

A deck or floor ledger fastened to a wood rim (band) joist, a concrete wall or a fully grouted masonry wall with through bolts, lag screws or anchors: the adjusted lateral design value of one fastener (NDS 2024 Chapters 11 and 12, with the gap of the sheathing and washers by the general dowel equations of AWC Technical Report 12), the required on-centre spacing along the ledger under every ASCE 7-22 combination, the check of the spacing used, the fastener pattern (edge, end and row distances) and, as a comparison, the prescriptive IRC deck ledger table.

> **Source note.** The NDS provisions were checked against ANSI/AWC NDS-2024 (scanned, with Commentary); the steps cite the section, table or equation and the printed page. The gap equations are those of AWC TR12 (2015), Table 1-1. The masonry anchor bolt follows TMS 402-22 as TMS's cross-reference, 2022 handouts and sample Guide examples state it (the code text is not on hand); its masonry equations were checked against TMS 402-13. One value is unconfirmed: φ of the anchor steel in shear (see the checks). The IRC table values are those of AWC DCA 6-09 Table 5 (the IRC 2009 to 2021 ledger table); the IRC 2021 text itself is not freely downloadable as a file (ICC read-only access) and the placement table was read from state-adopted copies (to confirm against the code text: IRC 2021 Table R507.9.1.3(2)).

Inputs of the wood ledger

- **Support and fastener:** a wood rim joist (through bolts or lag screws), a concrete wall or a fully grouted masonry wall (anchor bolts or post-installed anchors). Diameters 1/2, 5/8 and 3/4 in (bolts and anchors) and 1/2 and 5/8 in (lag screws, reduced body: D = Dr in the yield equations, NDS 12.3.7.1 c, with the thread length of Table L2 or entered). Fyb = 45,000 psi (Table I1). A ledger shall not be attached to brick, stone or masonry veneer (IRC R507.9.1.1); lag shields and lead anchors are not covered.
- **Concrete wall:** f'c and the anchor embedment. The dowel bearing strength of concrete is Fe = 3 f'c, at most 7,500 psi (AWC TR12 Table A1; the basis of NDS Table 12E, which assumes f'c ≥ 2,500 psi and 6 in embedment); the embedment is the bearing length lm. The anchor's own strength in the concrete (ACI 318-19 Chapter 17, for example from the Anchors in concrete calculation, or the anchor's ICC-ES report) is the engineer's entered design value per anchor (φVn for LRFD, the allowable load for ASD), for this spacing and edge distance.
- **Grouted masonry:** f'm, the masonry dowel bearing strength (no published value exists for masonry: the concrete relationship 3 f'm ≤ 7,500 psi is prefilled as the engineer's judgment, or the engineer enters Fe), and the anchor: a cast-in headed or bent-bar anchor bolt designed by TMS 402 (embedment, fy and fu, the edge distance lbe in the direction of the load and the wall thickness), or a post-installed anchor with its entered design value.
- **Ledger:** 2x6 to 2x12 sawn lumber; species G from NDS Table 12.3.3A, or G entered; the ledger length (for the elevation and the fastener count); E for the group action of a vertical pair. IRC R507.9.1.1 asks for a 2x8 or larger No. 2 preservative-treated Southern Pine, incised hem-fir or naturally durable ledger.
- **Rim (band) joist:** sawn lumber (species G) or an engineered rim board with the manufacturer's equivalent specific gravity for dowel bearing; its thickness and depth; the distance from the top of the rim to the top of the ledger (the rows' position in the rim); E for a vertical pair.

- **Between the ledger and the support:** the sheathing and the stacked washers (or a spacer). Their total is the gap g of the TR12 equations; the sheathing's own bearing is neglected (conservative). Without a gap the equations are NDS Eqs. 12.3-1 to 12.3-6.
- **Fastener pattern:** staggered (one fastener per station, alternately in the top and the bottom row, IRC Figure R507.9.1.3) or pairs (two rows aligned); the on-centre spacing s along the ledger; the distances of the two rows from the top and the bottom of the ledger; the end distance.
- **Loads:** from the deck joists (D, L, Lr, S in psf on half the joist span: 10 psf dead and 40 psf live are the IRC deck table basis) or line loads along the ledger by load type; optional point loads spread over a length of ledger (the fasteners there carry them with the line load). Every ASCE 7-22 combination is evaluated with its own CD (ASD; at most 1.6 for connections, NDS 11.3.2) or KF φz λ (LRFD; λ at most 1.0 for connections, Table N3 footnote 1); the load duration factors of the code options section apply.
- **Service conditions:** moisture (CM, Table 11.3.3: an exterior deck ledger is usually in service wet, CM = 0.7; the engineer decides) and temperature (Ct, Table 11.3.4); the IRC comparison on or off.

Auto-design (editor toolbar) varies the fastener type (bolt or lag screw, on a wood rim only) and its diameter, the largest on-centre spacing that passes (36 in down to 2 in), and, when unlocked, the ledger size (locked by default: the ledger usually matches the joist depth for the hangers). A lag screw's length follows the candidate (through the ledger, the gap and the rim with the tip past its inside face). Objectives: least fastener steel per foot; fewest fasteners per foot; lowest cost index. The support, the gap, the loads and the pattern's edge distances are kept; see Auto-design.

Checks performed

Check	Basis
Lateral design value per fastener	Z = least of the yield modes (NDS Table 12.3.1A, or with a gap TR12 Table 1-1: P = (−B + √(B² − 4AC)) / 2A) with Rd of Table 12.3.1B at θ = 90°; Fe⊥ = 6,100 G^1.45 / √D in the ledger and a rim, Fe of concrete or masonry as above; Z' = Z × CD (or KF φz λ, KF = 3.32, φz = 0.65) × CM × Ct × Cg × CΔ (Table 11.3.1); against w s / n per fastener under the governing combination
Fastener spacing ≤ required spacing	s_req = n Z' / w (n = 1 staggered, 2 pairs), the least over the combinations and the point-load locations
Group action	A row is two or more fasteners aligned with the (vertical) load (11.3.6.2): a vertical pair is a row of 2 (Eq. 11.3-1, members loaded perpendicular to grain with the area thickness × s, 11.3.6.3; a wall taken as rigid); staggered columns are rows of one (Cg = 1.0) unless they are closer than 1/4 of the distance between the rows (then one row)
End distance of the ledger	Table 12.5.1A, perpendicular to grain loading: minimum 2D, full design value at 4D, CΔ = a / 4D between; the least CΔ applies to every fastener (12.5.1.2)
Spacing in a vertical pair	Table 12.5.1B: minimum 3D, full at 4D (CΔ)
Spacing between fastener columns	Table 12.5.1D, perpendicular to grain loading: 2.5D, (5l + 10D)/8 or 5D by l/D
Edge distances	Table 12.5.1C, perpendicular loading: loaded edge 4D, unloaded edge 1.5D. The ledger bears up on the fastener (its top edge is loaded); the fastener bears down on the rim (its bottom edge is loaded)

Check	Basis
Ledger depth for the pattern	4D + 1.5D (+ 4D for a vertical pair)
Lag screw penetration	4D of thread in the rim without the tapered tip (12.1.4.6): a 1/2 in lag screw does not meet it in a single 2x rim (1-1/2 in); a double rim does
Anchor in concrete (entered)	The engineer's φVn / allowable value per anchor against the load per anchor (NDS 11.2.4)
Anchor bolt in grouted masonry	TMS 402-22 shear: masonry breakout 4 Apv √f'm, crushing 1050 (f'm Ab)^0.25, pryout 8 Apt √f'm (402-13 Eqs. 9-6 to 9-8), steel 0.6 Ab fu (Eq. 9-7) (strength, 9.1.6.3.2, φ 0.50 masonry, 0.65 steel (to confirm against the code text)) or the allowable stress forms 1.25 Apv √f'm, 350 (f'm Ab)^0.25, 2.5 Apt √f'm (402-13 Eqs. 8-6 to 8-8), 0.25 Ab fu (Eq. 8-7) (8.1.4.3.2); fu ≤ min(1.9 fy, 125 ksi), Ab the effective tensile area π/4 (do − 0.9743/nt)² (6.3.8); Apt = π lb² less half the overlaps with the nearest anchors, Apv = π lbe²/2 within the wall thickness and the wall length per anchor (6.3.2, 6.3.3); lb ≥ max(4db, 2 in) (6.3.6)
IRC prescriptive spacing (advisory)	IRC 2021 Table R507.9.1.3(1) when its conditions hold: 1/2 in lag screw or bolt (with up to 1/2 in stacked washers), sheathing ≤ 15/32 in, a 2 in nominal band joist with G ≥ 0.42, a Southern Pine, Douglas fir-larch or hem-fir ledger 2x8 or larger, staggered, 40 psf live and 10 psf dead load by joist span up to 18 ft, no point loads, the lag screw tip past the band joist. Otherwise a note says which condition is not met
IRC placement (advisory)	IRC 2021 Table R507.9.1.3(2): ledger top 2 in, bottom 3/4 in; band joist top 3/4 in, bottom 2 in; ends 2 to 5 in; rows 1-5/8 in apart
Notes	Ledger bending between fasteners (negligible), the deck's lateral load connection (IRC R507.9.2), the rim joist and its support, flashing and corrosion protection, anchor tension

The IRC table is not an NDS design: it rests on load tests of ledger connections (Virginia Tech and Washington State University, Carradine, Bender, Loferski and Woeste) with test-based allowable loads per fastener of roughly 385 lb (1/2 in lag screw), 725 lb (1/2 in bolt) and 615 lb (bolt with stacked washers), back-calculated from the table at 50 psf on half the joist span. The NDS yield model is two to three times more conservative (JLC, "Load-tested deck ledger connections", 2004), and the gap and wet service lower it further, so the NDS spacing is much closer than the table's. The calculation designs by NDS and shows the table beside it.

Limitations

The drawing sheet shows the section at the ledger (the support, the sheathing and washers, both fastener rows with their washers, nut, head or embedment, the thicknesses, the rim's edge distances and the line load) and the ledger elevation near its end (the pattern as designed: staggered or paired, the spacing as equal spaces, the end distance and the edge distances). Flashing, the joists and their hangers, the lateral load connection, the rim's support and the anchor's strength in the wall are not part of the calculation; the sheet notes say so.

- Vertical load only (perpendicular to grain in the ledger and in a horizontal rim, θ = 90°). Withdrawal and the deck's lateral load connection (IRC R507.9.2) are notes, not checks.
- Not checked: the rim joist's bending, bearing and its connection to the house; the ledger's bending between fasteners (negligible); the joist hangers; local stresses of NDS Appendix E; hollow (ungrouted) masonry; brick veneer (not permitted); a ledger carrying girders or beams (IRC R507.9.1.1).
- The anchor's strength in concrete and the strength of post-installed anchors in masonry are the engineer's entered values. No published dowel bearing strength exists for grouted masonry: the 3 f'm relationship is the engineer's judgment.
- Structural screws with an evaluation report (proprietary ledger screws) are not covered: use their report.

- One ledger ply; a built-up ledger and its ply fastening are the engineer's design.
- Point loads are spread uniformly over the entered length; overlapping point loads are not combined.
- φ of the anchor bolt steel in shear is 0.65, which is not confirmed. TMS 402-22 moved the steel strength from fy to fu, and the code text of 9.1.4.3 is not on hand. The 402-16 value of 0.90 goes with fy, and it does not fit fu: the Designers' Guide 2022 prints a steel design tension of 6,390 lb for a 1/2 in. A307 bolt, which is 0.75 × 0.142 in² × 60 ksi. No shear value is printed. The allowable forms against the nominal forms suggest about 0.65 in shear and 0.75 in tension, and 0.65 is used. Masonry crushing governs at usual sizes, so the value rarely decides a design.
- The TMS 402 projected areas use pairwise overlaps of the nearest anchors (conservative where three overlap) and assume the masonry extends at least lb beyond the anchors on the wall face.

Verification examples

Verification status: 76 values compared in 46 examples (41 published, 5 hand calculations from the cited equations): 76 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published tables and examples (`backend/tests/test_verify_wood_ledger.py`; tables rounded to 10 lb, compared within 5 lb or 2 %; the module computes Fe from G, the tables round Fe to 50 psi):

Source (page)	What it checks	Published	ILX Calcs
NDS 2024 Table 12E (p. 106; NDS 2018 p. 99)	1-1/2 in ledger to concrete, Fe = 7,500 psi, 6 in embedment, Z⊥ for 1/2, 5/8, 3/4 in, G = 0.55 / 0.50 / 0.46	410, 580, 660 / 380, 530, 590 / 360, 470, 520 lb	410, 579, 666 / 382, 530, 580 / 360, 469, 514 lb
NDS 2024 Table 12A (p. 101)	Bolts, ledger to rim, Z⊥: 1-1/2 in rim G 0.50 and 0.37; 3-1/2 in rim 1/2 in G 0.50, 3/4 in G 0.55 and 0.43; 2-1/2 in rim 5/8 in G 0.46	220, 140, 330, 580, 400, 300 lb	218, 141, 331, 579, 405, 303 lb
NDS 2024 Table 12J (p. 112)	Lag screws, p = 8D, G 0.50, Z⊥, 1/2 and 5/8 in	200, 280 lb	205, 278 lb
AWC TR12 Example 3.1 (p. 17)	1/2 in bolt, Hem-Fir 1-1/2 in members, both perpendicular to grain, gap 0 / 1/4 / 1/2 in: Im, Is, II, IIIm, IIIs, IV and Z	g = 0: 383, 383, 176, 289, 289, 387; Z 176. g = 1/4: 383, 383, 157, 258, 258, 349; Z 157. g = 1/2: 383, 383, 142, 231, 231, 315; Z 142 lb	381, 381, 175, 288, 288, 386; 175. 381, 381, 156, 257, 257, 348; 156. 381, 381, 141, 230, 230, 314; 141 lb
IRC 2021 Table R507.9.1.3(1) / AWC DCA 6-09 Table 5 (p. 12)	Spacing by joist span 6 to 18 ft: lag screw; bolt; bolt with stacked washers	30, 23, 18, 15, 13, 11, 10; 36, 36, 34, 29, 24, 21, 19; 36, 36, 29, 24, 21, 18, 16 in	the same (table lookup)

Hand calculations (closed form, written out in the test file):

Example	What it checks	Hand calculation	ILX Calcs
HAND-LEDGER-DEFAULT	The default design: 2x10 Southern Pine ledger to a 2x spruce-pine-fir rim through 15/32 in sheathing, 1/2 in bolts at 5 in staggered, 10 ft joists, 10 + 40 psf, in service wet, ASD	Z = 170.3 lb (TR12 mode II, g = 0.469 in); Z' = 119.2 lb; load per bolt 104.2 lb; s_req = 5.72 in	170.3, 119.2, 104.2, 5.72

Example	What it checks	Hand calculation	ILX Calcs
HAND-LEDGER-LRFD	The same in LRFD, 1.2D + 1.6L (λ = 0.8)	Z' = 205.8 lb; s_req = 6.50 in	205.8, 6.50
HAND-LEDGER-CONC2000	1/2 in anchor in 2,000 psi concrete (Fe = 3 f'c = 6,000 psi), mode IIIs	396.7 lb	396.7
HAND-TMS-ANCHOR	1/2 in headed A307 anchor bolt, fu = 60 ksi, f'm = 2,000 psi, lb = 4 in, lbe = 6 in, 24 in apart: ASD Bvc, Bvs = 0.25 Ab fu, Bvpry, Bvb; strength φBvn with 0.65 × 0.6 Ab fu	1,437, 2,129, 5,620, 3,161 lb; 2,155, 3,321, 8,992, 5,058 lb	the same
HAND-LEDGER-PAIRED-CG	Vertical pair, rim E 1.0e6, ledger E 1.6e6 psi	Cg = 0.993	0.993

No free, complete NDS worked example of a deck ledger connection was found (research/DOCUMENTS_NEEDED.md): the AWC Structural Wood Design Examples and the DCA 6 Commentary would add one.

CHAPTER 65

Wood stair

One straight flight on sawn lumber stringers with its treads, to NDS 2024 with ASCE 7-22 loads and the stair limits of IBC 2021 1011. LRFD or ASD.

Inputs of the wood stair

- **Flight:** the stair's use sets the code limits checked; risers, riser and tread, clear width and headroom. The span on plan is (risers − 1) × tread plus the span beyond the treads, which you enter.
- **Stringers:** how many, evenly spaced; solid, with the treads on ledgers or housed, or cut in a sawtooth. Species, grade, size, plies.
- **Cut stringers and the notch limit:** NDS 4.4.3.2, as written, limits interior notches to one-sixth of the depth in the outer thirds and permits none in the middle third. A cut stringer fails that, so the check is on by default. AWC's own deck guide (DCA6) prescribes cut stringers with a 5 in. throat. If you accept that, turn the limit off; the calculation then checks the net section at the throat and says the decision is yours.
- **Treads:** flat boards of the same species and grade, spanning between stringers. Uniform load is shared by the boards in a tread; the concentrated load is on one board.
- **Loads:** dead and live load on plan, the concentrated load on a tread, a guard's dead load on an outside stringer. ASCE 7-22 Table 4.3-1 gives 100 psf, 40 psf in one- and two-family dwellings, and 300 lb on a tread, not at the same time as the uniform load. An entered value below the table gets a warning.
- **Service conditions:** wet service and incising (exterior stairs usually have both), temperature, deflection limits. The tread's L/288 is AWC DCA6's basis, not an IBC limit.

Auto-design (editor toolbar) varies the stringer size, the number of stringers and the tread board. Objective: the least wood across the stair.

Checks performed

Check	Basis
Riser, tread, width, headroom, flight rise	IBC 2021 1011.2, 1011.3, 1011.5.2, 1011.8
Stringer bending, shear, bearing	NDS 3.3, 3.4, 3.10 by the beam engine of the wood beam, on the horizontal projection: the worst of an inside and an outside stringer. Every combination of ASCE 7-22 2.3 or 2.4
Stringer deflection	NDS 3.5 with creep, against your limits; the horizontal plane result times $1/\cos^2\theta$ for the load and length on the slope
Cut stringer	Section at the throat $d - R\,T / \sqrt{(R^2 + T^2)}$; the notch limit of NDS 4.4.3.2 as an input; 5 in. throat (DCA6) as a warning
Tread bending, shear, deflection	NDS 3.3, 3.4, 3.5, flat use, uniform and concentrated load separately

The drawings are the flight elevation, the stringer and tread sections, and the stringer's shear and moment diagrams.

Limitations

- One straight flight, simply supported on plan. Landings, winders and connections are not designed. The guard is a dead load only.
- Axial force in the stringer is not calculated: with two vertical seats it is zero at midspan, where the moment is largest. A stringer that thrusts against its base needs that checked separately.
- Vibration is not checked.
- A cut stringer is checked braced at every tread.

Verification examples

Verification status: 14 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 14 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No free worked example of a wood stair exists. Everything here is a hand calculation from the cited equations, listed on the Verification page, and two comparisons with AWC DCA6-2015, p. 21, which publishes rules and not calculations:

- Table 6's minimum tread of a 2x8 at 36 in., wet and incised, No. 2 Douglas Fir-Larch, 300 lb and L/288. It passes bending at 993 psi against 993.6 and deflection at 0.105 in. against 0.125 in., and a 2x6 fails: the table reads as this calculation does for Douglas Fir-Larch. By hand, Hem-Fir's lower E gives 0.129 in., a little over the limit, though the table lists it with the same 2x8.
- Figure 28's two stringer spans, cut 6 ft with a 5 in. throat and solid 13 ft 3 in., come out at the same bending stress within 5 % on the net section. That is why the net section is offered once the notch limit is turned off.

Neither is a published result to the same basis, so this type should be read as hand verified.

CHAPTER 66

Wood I-joist

A prefabricated wood I-joist to NDS 2024 Chapter 7 with ASCE 7-22 combinations, in ASD or LRFD. An I-joist is a proprietary product: the NDS gives no values (7.2), so the calculation works from the design resistances of the product, not from stresses. It holds the APA Performance Rated I-joists (PRI) of APA Z725H-2022 Table 9, and takes values entered from a manufacturer's report.

Inputs of the I-joist

- **Joist:** the APA depth and series, or the moment Mr, shear Vr, intermediate and end reactions, EI, K, the vertical load capacity, depth, flange width and flange Fc⊥ from the manufacturer. Values are at normal duration of load. K is as APA defines it: a simple span under a uniform load deflects $5w\ell^4/384EI + w\ell^2/K$. A manufacturer who writes the shear term as $w\ell^2/8GA$ has K = 8 GA. The APA values come from a copy of the 2022 revision (APA's current one is March 2024): confirm them, and the product you specify.
- **Spacing and loads:** area loads per square foot over the spacing and the full length, plus line and point loads with their load types. Own weight is entered (about 2.5 to 5 plf).
- **Partial loading:** on by default. L, Lr and S also act on part of the spans (ASCE 7-22 4.3.3, 7.5), as on the sawn lumber beam. The APA span tables were not calculated this way: read their note on it.
- **Bearing:** the lengths at the ends (at least 1-3/4 in.) and at intermediate supports (at least 3-1/2 in.; the support next to a cantilever counts as intermediate), bearing stiffeners at the ends, an end in a hanger whose sides do not reach the top flange (it needs a stiffener), load stiffeners at point loads, and the Fc⊥ of the plate. Table 9 note i limits a reaction to the bearing on the flange or the plate, whichever is smaller.
- **Lateral stability:** whether the ends are held against rotation at the bearings (required, NDS 7.3.5.2), and the unbraced lengths of the top and bottom flanges. 0 = held throughout, CL = 1.0. An unbraced flange is designed as a column by itself, buckling sideways (7.3.5.3), which needs the flange's Fc and Emin from the manufacturer: the prefilled values are sawn No. 2 values and matter only where a length is entered.
- **Deflection and service:** limits as span / n (APA's tables use 480 for live load; the IBC floor value is 360), creep (Kcr = 1.5, NDS 3.5.2), temperature, and the APA cantilever limits, which you can clear where you engineer the cantilever.
- **LRFD format conversion:** NDS 7.3.8 takes KF from the manufacturer. 2.54 and 2.88 are prefilled, the Table 2.3.5 values for bending and shear: check them against the product report. A reaction takes the shear value, as Table 7.3.1 has φ = 0.75 for it.

Auto-design varies the depth and series from the APA table (shallowest then lowest series, or lowest stiffness that passes); the spacing when unlocked. Entered values stay as entered.

Checks performed

Check	Basis
Moment, positive and negative, with CD or KF φ λ, Ct and CL	NDS 7.3, Table 7.3.1; 7.3.5.2, 7.3.5.3

Check	Basis
Shear, no reduction near supports	NDS 7.3, 7.4.7
End reaction, interpolated by bearing length, with or without stiffeners, limited by flange or plate bearing	NDS 7.4.1; APA Table 9 notes f, i
Intermediate reaction at 3-1/2 in.	NDS 7.4.1; APA Table 9 note e
Minimum bearing lengths	APA p. 8 note 7
Stiffener at a low hanger; load stiffener above 1,500 lb on the top flange, adjusted for duration	NDS 7.4.2; APA p. 16
Vertical load in the plane of the joist against VLC, no duration increase	APA Table 9 note a, p. 8 note 10
Ends held against rotation	NDS 7.3.5.2
Cantilever at most 4 ft and 1/4 of the adjacent span (option)	APA p. 17
Live and total deflection with bending and shear deformation, creep	NDS 7.4.5, 3.5.2; APA Table 9 note h; IBC Table 1604.3 limit entered

Limitations

- Web holes (7.4.3), notches (7.4.4), wet service (7.1.4: dry use only), pressure treatment and floor vibration are not checked.
- Composite action with glued sheathing is not used: Table 9 is the bare joist, so spans come out shorter than the APA span tables, which assume a glued-nailed floor.
- On a continuous joist the moments and reactions are those of a bending analysis; shear deformation is added to the deflection but does not redistribute them.
- A wall or roof load at the end of a cantilever usually needs reinforcement: APA Tables 5 and 6 are not applied; the module warns.
- Web and bearing values with stiffeners at intermediate supports are the manufacturer's: only the Table 9 intermediate value is used.
- NDS 7.3.5.3 calls the flange column method "one acceptable method"; the manufacturer may have another.

Verification examples

Verification status: 22 values compared in 9 examples (5 published, 4 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No free worked example of an I-joist calculation was found. The NDS has none; the APA guide gives design values and span tables only. So the module is checked three ways, and the reader should weigh them as such:

Example	What it checks	Published / hand value	ILX Calcs
HAND-1	11-7/8 in. PRI-40 at 16 in., 16 ft, D 15 + L 40 psf, ASD and LRFD	Hand: M' 3,545 lb-ft, V' 1,420 lb, R' 1,200 lb from Table 9; Δ by Table 9 note h; LRFD with KF φ λ	same
HAND-2	Snow duration, 2-7/8 in. bearing	Hand: R' = (1,200 + 1,420)/2 × 1.15 = 1,506.5 lb	same
HAND-3	Plate bearing governs (Table 9 note i)	Hand: 708.8 lb	same

Example	What it checks	Published / hand value	ILX Calcs
HAND-4	Bottom flange unbraced 6 ft (7.3.5.3)	Hand: CL = Cp of the flange	same
APA-T1-1 to 3	Table 1 spans less 12 in. (note b, nailed-only floor), where L/480 governs	Ratio of deflection to limit 1.0, 5 % band: a reverse reading, since the tables are for a composite floor and print clear spans	within the band
APA-T1-4	Table 1, 11-7/8 in. PRI-40 at 24 in., 16 ft 8 in., moment governs	Ratio 1.0, 5 % band	within the band
APA-T2-1	Table 2, 14 in. PRI-50 at 24 in., two spans of 16 ft 1 in., interior reaction governs	Ratio 1.0, 5 % band	within the band

The module's table of APA values is checked by test against the printed rows. Open: the current APA revision and a product evaluation report with its KF values.

CHAPTER 67

Rafters and ceiling ties

A sloped sawn lumber rafter with the tie that resists its thrust: a pair of rafters on two walls, a ridge board with ceiling joists or rafter ties at the plate or raised above it, or a structural ridge beam or wall with no thrust. The rafter, the tie and the nails at their joint are designed to NDS 2024 with ASCE 7-22 load combinations in ASD or LRFD, and the nailing is compared with IRC Table R802.5.2.

With a ridge board the two rafters and the tie form a three-hinged frame. The walls give vertical reactions only, the ridge carries a horizontal thrust H and the tie pulls the rafters together. Moments about the heel give $H = M0 / (Hr - Hc)$, with M0 the moment of the loads on one rafter about the heel, Hr the height of the ridge and Hc the height of the tie above the plate. A uniform load w on the plan, with no overhang, gives $H = w L^2 / (2 Hr)$ at the plate; raising the tie multiplies it by $1 / (1 - Hc/Hr)$ and bends the rafter at the tie.

Inputs of the rafters and ties

- **Rafter:** species and grade, size (dimension lumber 2" to 4" thick, NDS Supplement values), or *Other* with the reference values you enter (including the size factor) and the specific gravity G for the nails; spacing; the repetitive member factor (three or more rafters at 24 in. or less joined by the sheathing, NDS 4.3.9: your call); the self-weight and the wood density.
- **Roof geometry:** the slope as a rise in 12, the horizontal span L of one rafter (wall to ridge, half the building span for a gable), the overhang beyond the wall on the plan, and the seat length lb: the horizontal length of the birdsmouth seat cut on the plate. The notch depth at right angles to the rafter is $lb \sin \theta$.
- **Ridge:** *ridge board* (the tie takes the thrust) or *structural ridge beam or bearing wall* (a sloped simple beam; the reaction at the ridge is reported for you to design that support, for example with the wood beam or glulam beam type).
- **Tie:** at the top plate or raised, with its height Hc above the plate. IRC R802.5.2 keeps ceiling joists in the bottom third of the rafter height unless the ridge is designed as a beam: a higher tie here is your engineered design.
- **Ridge connected for tension:** under net uplift the rafters pull apart at the ridge. On: a strap, gusset or collar tie carries the force, which is reported for you to design it (IRC R802.4.2). Off, with a raised tie: the ridge opens, the tie balances the moment about the heel ($T = M0 / Hc$) and the wall takes the horizontal balance, which is reported. Off, with the tie at the heel: nothing else can hold the rafter, so an advisory tells you the force the ridge must carry.
- **Roof loads:** dead load along the slope or on the plan (your choice of how it is measured), roof live load Lr and snow S on the plan, wind toward and away from the roof, normal to it, as strength-level components and cladding pressures, and vertical point loads at a distance from the wall (negative on the overhang). Both rafters are loaded alike: for drifts or unbalanced snow enter the heavier side.
- **Ceiling joist / rafter tie:** species, grade and size, one at every rafter; the ceiling dead load and attic live load it carries in bending (0 for a rafter tie with no ceiling), its span between supports and the unbraced length of its top edge; the nail and the number of nails at the rafter-to-tie joint (face nailed through the rafter, single shear); an optional lap splice and its nails.

- **Lateral bracing:** the unbraced top edge (0 = sheathed, CL = 1.0), and how the bottom edge is braced: at the supports and the tie, continuously, or at a spacing. The bottom edge is in compression under uplift and over the wall with an overhang.
- **Service conditions:** wet service, temperature, incising.
- **Deflection limits:** span / n for live load and wind and for total load (IBC Table 1604.3; 0 skips the check), and how they are measured: perpendicular to the rafter over the span along the slope, or the horizontal projection used by the AWC span tables and the IRC (the same stiffness; the deflection is multiplied by $\cos^2\theta$ and the span is the plan span). The wind factor for deflection (IBC Table 1604.3 note f), creep on the dead load (NDS 3.5.2) and unseasoned lumber.
- **IRC comparison:** the table of the edition your jurisdiction adopted (2021 or 2018) and the ground snow load for its column. The table does not govern the design; it is compared with your nail count.
- **Load duration (code options):** CD per load type (ASD) and the live load type for λ (LRFD).

Auto-design (editor toolbar) varies the rafter size and the tie size and derives the number of nails for each candidate from the tie force of the input as given (5 % more, for a heavier rafter's self-weight). Species and grades are kept unless unlocked; slope, span, overhang, loads, seat, nail size and code options are never changed. Objectives: least wood; lowest cost index. See Auto-design.

Checks performed

Check	Basis
Statics, every combination	Load components perpendicular and parallel to the rafter; moment, shear and axial force along it, overhang as a cantilever. The frame that holds is chosen per combination: H = M0 / (Hr − Hc) and T = H − ΣFx, or the open ridge H = 0 and T = M0 / Hc
Bending	fb = M / S ≤ Fb' with CL of the edge in compression: top edge lu as entered, bottom edge as braced (NDS 3.3, Table 4.3.1)
Beam slenderness	RB ≤ 50 (NDS 3.3.3.7)
Shear	fv = 3V / (2bd) ≤ Fv' with V the full shear at the support perpendicular to the rafter (no reduction for loads near it)
Column slenderness	le/d ≤ 50 over the rafter length along the slope, Ke = 1.0; the weak axis is braced by the sheathing (NDS 3.7.1.4)
Bending and compression	NDS Eq. 3.9-3, CP over that length, CL of the compression edge
Bending and tension	NDS Eq. 3.9-1 and 3.9-2
Deflection	Double integration of M / EI, relative to the chord from the wall to the ridge (to the tie when the ridge is open), against span / n; an overhang against 2 × overhang / n (IBC Table 1604.3 note h); Kcr on the dead load in the total
Seat bearing	fθ = R / (b lb) against Hankinson's Fθ' with θ = 90° minus the slope (NDS 3.10.3), Cb when the seat is away from the end
Birdsmouth depth	lb sin θ ≤ d/4 at the end of a rafter with no overhang (NDS 4.4.3.1), ≤ d/6 with an overhang (4.4.3.2); a note for 4 in. rafters, whose tension-side interior notches are not permitted
Notch shear	Vr' = (2/3) Fv' b dn (dn/d)² (NDS Eq. 3.4-3), also applied with an overhang
Bending at the notch	With an overhang, on the net depth at the wall (NDS 3.1.2)
Tie	Tension plus the bending of the ceiling load on its own span, Eq. 3.9-1 and 3.9-2 (NDS 3.8, 3.9.1)

Check	Basis
Rafter-to-tie nails, lap splice	Yield limit equations, single shear, lm = min(L − ls, tie thickness) (NDS 12.3), Z' = Z CD ≤ 1.6 or Z KF φ λ with λ ≤ 1.0, CM, Ct; penetration ≥ 6D (12.1.6.4)
IRC comparison	Table R802.5.2: next lower slope, next higher spacing, ground snow and span; the Hc/Hr factor; 10d common at 1.2 times in the 2021 table
Uplift, ridge tension, wall horizontal force, tie compression	Advisories with the force for the connection or bracing you design
Ridge drop and wall spread (report only)	Short-term service: (δtie + v sin θ) / ((1 − Hc/Hr) tan θ), with δtie the stretch of half the tie and v the rafter's bending at a raised tie; wall spread = drop × tan θ. No code limit: your judgment for the finishes

Limitations

- Both rafters are loaded alike. A drift, unbalanced snow or wind on one side is entered as the heavier side's load; the asymmetric frame (shear at the ridge) is not analysed.
- The ridge beam or wall, the ridge board's own design, collar ties (other than as the ridge connection you say exists), purlins and struts, hips and valleys are not designed.
- The rafter-to-plate connector, the ridge strap or gusset and the wall under the horizontal force are not designed; their forces are reported. Nail spacing is yours (nails under 1/4 in. take no geometry factor; NDS 12.1.6, 12.5.1.1).
- The tie's shear, bearing and deflection under the ceiling loads are not checked (use the wood beam type); its bending is a simple span.
- The strong-axis column length is the whole rafter along the slope. A raised tie or an overhang does not shorten it (conservative); the sheathing is taken to brace the weak axis.
- Rafter shortening and nail slip are left out of the ridge drop. The deflection check is relative to the chord: the rigid rotation of the rafter is the ridge drop, reported separately.
- A rafter with no overhang notched at the seat is treated as an end notch; the code limit that applies with an overhang is the interior notch limit.
- Wind on the soffit of the overhang is not added.
- Reference values come from the NDS 2024 Supplement Tables 4A and 4B, read on 2026-11-08 (as the wood beam). The model IRC is sold: its tables were read in the 2021 and 2018 Seattle Residential Codes. The 2024 IRC has not been read.

Verification examples

Verification status: 79 values compared in 35 examples (33 published, 2 hand calculations from the cited equations): 78 within tolerance, 1 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Example	Published	ILX
AWC STJR-2024 Example 2 and Table R-1 (p. A-4, p. 13): 2x6 at 16 in, 20 + 10 psf, L/240, Fb 1400 psi	span 13 ft 3 in, E 1.35 million psi	13 ft 3.4 in, 1.35

Example	Published	ILX
Same, seat 3.5 in, 14 ft (Tables 9.1, 9.2, p. 3)	Fc⊥ 53 psi	53.3 psi
Tables R-1, R-5, R-13 (pp. 13, 17, 25): 2x8, 2x10, 2x6 at 12 to 24 in	spans and E of five more cells	within the print rounding
Table 9.1: 66.67 plf, 12 ft on 1.5 in, 20 ft on 2.5 in	178 psi, 178 psi	177.8, 177.8
ClearCalcs gravity printout (Aug 2020): 8.5 ft run, 6 ft rise, tie 3 ft below the ridge	D: T 321 lb, M 723 lb-ft, V 185 lb; S: 562, 1,260, 324	321.1, 722.5, 185.2; 561.9, 1,264, 324.1
ClearCalcs uplift printout: 10:12, tie 2 ft below the ridge, overhang, 40 psf	D: 576 lb, 1,080 lb-ft, 369 lb; W: 697 lb, 474 lb-ft; 0.6D + 0.6W: 309 lb, 115 and 210 lb-ft, 124 lb	574.6, 1,081, 367.8; 697.4, 473.9; 309.6, 115.5 and 210.4, 124.2
IRC 2021 Table R802.5.2(1), p. 8-25, five rows; IRC 2018 Table R802.5.2, p. 488, five rows	3 to 39 nails	equal
IRC Table R802.5.2(2), Hc/Hr 1/3, 1/4, 1/5, 1/6, 1/10	1.5, 1.33, 1.25, 1.2, 1.11	from the statics: 1.500, 1.333, 1.250, 1.200, 1.111
NDS 2024 Table 12N, p. 117: 16d at G 0.55, 0.50, 0.43, 0.42; 10d at 0.50, 0.43, on a 1-1/2 in side member	154, 141, 122, 120; 118, 102 lb	within the table rounding
HUD Residential Structural Design Guide (2000), Example 7.3, p. 7-33	reaction 652 lb; joint tension 1,337 lb	653 lb; **653 lb (documented deviation)**
Hand calculations, ASD and LRFD	thrust, moments, stresses, Hankinson, Eq. 3.4-3, nails, deflection, raised tie	equal

The deviation. The HUD example resolves the bearing reaction through the rafter axis as a pinned truss joint. That treats the rafter as carrying its load only at its ends; the moment balance of the rafter gives half that force. The AWC span tables have no thrust, but the ClearCalcs printouts and the module agree on this statics, and the IRC table's own counts match it. The HUD guide itself says that its 12 nails exceed conventional practice.

No free worked example of the full set of NDS checks on a rafter pair was found: the checks of bending with axial force, Hankinson at the seat and the birdsmouth are hand calculations from the cited equations (research/DOCUMENTS_NEEDED.md).

CHAPTER 68

Wood truss

Wood truss: pick a layout, span and pitch, and get chords and webs sized and checked to ANSI/TPI 1-2022 (the default, for metal-plate-connected trusses) or to NDS 2024 alone (heavy timber), for roof and floor trusses, with ASCE 7-22 load combinations.

Inputs of the wood truss

- **Roof or floor, truss type:** roof trusses offer 18 layouts, floor trusses the parallel-chord ones.
- **Geometry:** span between the outer bearing lines, pitch (rise in 12), heel height, depth on parallel chords, flat length on a hip, rise on a bowstring, room width and height on an attic, overhangs, panels (Auto: about 6 ft on a pitched truss, about 1.5 depths on parallel chords; a count the layout cannot have snaps to the nearest one and the results say so).
- **Supports:** pin, roller or none at each end, intermediate supports (moved to the nearest chord joint, and the results say where), bearing on the bottom or top chord of a parallel-chord truss.
- **Loads:** area loads in psf times the truss spacing: top chord dead, roof live, snow, unbalanced snow, floor live, bottom chord dead and live. Live load on half the span as well (ASCE 7-22 4.3.3, on by default, because web forces reverse under it). Wind rows (pressures normal to the top chord from your wind analysis, each checked from both directions), total horizontal seismic, SDS for the vertical effect, and point loads anywhere on a chord, also between panel points.
- **Members:** Size the members automatically is on by default: at every calculation the lightest section of each group that passes is chosen. Turn it off to enter the sections and have them checked as they are.
- **Bracing:** what holds each chord out of plane, and the webs: as needed (a compression web that fails unbraced gets a lateral brace at mid-length before anything is upsized, and the results, schedule and sheet name it), none, or every web. These are your judgment as engineer of record, and the braces themselves are not designed.
- **Members with their own section:** a mark from the schedule and a section, for one heavy web or chord panel. The rest keep the section of their group.
- **Serviceability:** live and total deflection limits as span / n, and the horizontal movement limit at a roller.

Auto-design varies the top chord, bottom chord and web sections, lightest first, within the given families, and fixes the result (it turns automatic sizing off). It never changes the geometry, loads, bracing or code options.

Analysis. A plane frame, first-order, with chords continuous over the panel points and pinned at heels and pitch breaks, and webs pinned at both ends, or every panel point pinned if you choose. Loads between panel points are carried in chord bending, so a chord has axial force and bending together. Every load case is solved once and combined with ASCE 7-22 2.3.1 or 2.4.1 (and 2.3.6 or 2.4.5 with seismic). Deflections always use the ASD combinations.

- **Design standard:** ANSI/TPI 1-2022 for metal-plate-connected trusses, or NDS 2024 alone. Design values and load duration come from the NDS and its Supplement in both.
- **Sections:** visually graded sawn lumber and timbers by species and grade; plies on chords; chords laid flat for the 4x2 floor truss. Bearing length.

- **Heavy timber truss:** choose NDS 2024 alone with sawn timbers, or Material = Glulam with a class and sections listed as width x depth. Glulam members use Fbx+ for sagging and Fbx- for hogging, and the lesser of CL and CV in bending, with L taken as the member's length (the conservative end of Eq. 5.3-1). A balanced lay-up suits a chord. Bolted joints, steel gussets, and heavy timber as a construction type (IBC 602.4, fire) are not checked here: the forces at every joint are listed, for Wood connection or your own design.
- **Repetitive member increase, Kcr, person load:** on by default where the standard sets them; yours to change.

Checks performed

Check	ANSI/TPI 1-2022	NDS 2024 alone
Effective lengths	7.2: in-plane K of a chord from its neighbours, overhangs, webs at 0.8	K entered
Repetitive increase	6.4.2, chords	4.3.9, bending
Tension	7.3.1	3.8.1
Compression	7.3.2	3.7
Bending	7.3.3	3.3
Axial + bending	7.3.4, 7.3.5, panel point checks without the buckling terms	3.9.1, 3.9.2
Slenderness	7.3.6	3.7.1.4
Shear	7.3.7	3.4.2
Bearing	7.3.9, with its buckling limit	3.10.2
Person load	6.2.2.5	Not applied
Deflection	7.6: live, live + creep, horizontal movement, narrow-face floor multiplier	Your limits

The report prints the steps of the governing member of each group and lists every member in the schedule. Turn on Report every member step by step, under Serviceability, to print every member.

Limitations of the wood truss

- **Connections are not designed.** Metal plates (ANSI/TPI 1-2022 Chapter 8) are normally the truss manufacturer's delegated design with that plate maker's tested values; gussets, bolts, heel joints and anchorage. The forces in every member at every joint are listed for that design.
- One section each for the top chord, bottom chord and webs, apart from the members you give their own.
- One lateral brace per web at most, at mid-length.
- Work lines meet at the joints: joint eccentricity and secondary moments are not modeled. First-order analysis, with moments amplified member by member where the standard does so.
- Gable-end, gambrel, piggyback and unequal-pitch layouts are not generated. Use the 2-D truss analysis for any other geometry.
- Ponding, fire, floor vibration, system bracing design, wind pressure calculation and trusses carrying other trusses (use point loads) are outside this calculation.
- The factors Km (6.4.3), CT (6.4.5) and Cs (7.3.2) stay 1.0, which gives up a benefit the standard permits.
- Not applied from ANSI/TPI 1-2022: rolling shear, heel shear and moment rules, extended chord bearing, reaction limits for top chord bearing, girders, and the deflection limits for a top chord panel and for overhangs.

Verification examples

Verification status: 18 values compared in 18 examples (7 published, 11 hand calculations from the cited equations): 18 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

These results carry the label "Preview: not yet verified, do not rely on these results" on the screen, in the report and on the sheet. The frame solver is checked against closed-form beams and the pin-jointed solver. Published sawn-member values are read through a chord for compression and bending (AWC E1.4 and Aghayere & Vigil through secondary tabulations, WoodWorks DE-2025, HUD). The ANSI/TPI 1 rules are hand calculations only: no published wood truss or TPI example is on file. ANSI/TPI 1 was read in place in part, and several of its provisions are not applied: see the limitations.

CHAPTER 69

Fire design of exposed wood

The structural fire resistance of an exposed wood member by NDS 2024 Chapter 16: the char depth for the rating, the residual section, and the member checked on it with the fire member strengths of Table 16.3.3. Sawn timbers, glulam, SCL and CLT panels, up to 2 hours.

Inputs of the fire design

- **Fire exposure:** the material, the required fire resistance in hours (NDS 16.3 calculations stop at 2 hours), the faces the fire reaches, and the nominal char rate βn. NDS 16.2.1 permits 1.5 in./hr for sawn lumber, glulam, SCL and CLT; a lower rate gets a warning because it needs published char data for the species.
- **Exposure:** four sides; three sides with the top protected (a beam under a deck or slab); or three sides with one side protected (a column against a wall). The protected face's own protection is yours: it must last the rating. A CLT panel is exposed on its bottom face.
- **Member:** sawn timbers by species, grade and size (NDS Supplement Table 4D); glulam by stress class, width and depth; SCL with the manufacturer's values and its volume factor exponent; CLT by layup, plies and lamination thickness (equal laminations: unequal ones need the FDS).
- **Span and loads:** a simple span (or a column height) with a uniform dead and live load and an axial dead and live load. For CLT the uniform load is per foot of panel width (psf × 1 ft).
- **Loads in the fire case:** this is your choice, so it is an input. AWC's method (Technical Report 10) compares the full allowable stress design loads, D + L, with the fire member strength, and that is prefilled. ASCE 7-22 2.5.2 gives 1.2D + 0.5L for extraordinary events. The building code and you decide.
- **Bracing and bearing:** the unbraced length of the compression edge (0 when a deck braces it, as long as the deck lasts the rating), the column effective lengths about the strong and the weak axis, and a bearing length with the number of exposed edges at the bearing (0 skips the bearing check).

The LRFD / ASD switch is hidden: the fire check has one format.

Auto-design (editor toolbar) varies the sawn timber size, or the glulam or SCL width and depth, or the CLT plies. Objective: the least wood. The rating, exposure, grade and loads stay yours.

Checks performed

Check	Basis
Char depth	βt from βn at one hour (Eq. 16.2-1), achar = βt t^0.813 (Eq. 16.2-2); CLT by Eq. 16.2-3, laminations falling off at each glue line
Effective char depth and residual section	aeff = 1.2 achar on each exposed face (Eq. 16.3-1), section properties of the reduced section (16.3.1.2, Table C16.3-1)
Bending	fb = M / S on the residual section against 2.85 Fb with CF or CV on the initial section and CL on the residual one, FbE from 2.03 × 1.20 Emin (Table 16.3.3); glulam takes the lesser of CV and CL

Check	Basis
Shear	1.5 V / A against 2.75 Fv
Compression	P / A against 2.58 Fc CP, about both axes, FcE from 2.03 × 0.822 Emin; le/d ≤ 50
Bending and compression	NDS Eq. 3.9-3 with the fire values
Bearing	the bearing width reduced by 1.5 achar at each exposed edge (Eq. 16.3-2, new in NDS 2024), against 1.67 Fc⊥
CLT bending	the commentary's conservative approach (C16.3.1.3): the panel of the full strong-axis laminations that remain, (FbS)eff by PRG 320 Annex A, times 2.85
Glulam laminations	a note: one outer tension lamination substituted for a core lamination up to 1 hour, two up to 2 hours (16.3.4)

No load duration, wet service or temperature factor applies in the fire (Table 16.3.3 footnote 2). The drawing is the section after the fire: the initial outline, the char layer and the residual section, or the CLT panel with its char layer.

Limitations

- The member at normal temperature is not designed here. Design it with the beam, column or CLT type.
- Protected members and membrane protection (FDS, TR 10 Part II), connections and their protection, char at intersections and abutting edges (16.2.3), lumber decking (16.3.5), thermal separation and burn-through are not covered.
- One simple span with uniform load and an axial load. Other load patterns need their moments entered as an equivalent uniform load.
- CLT rolling shear in the fire is not calculated: check it with the manufacturer's values. The PRG 320 values behind the CLT check are still to be confirmed against the standard.
- Glulam stress class values were read against Supplement Table 5A on 2026-11-12 (p. 80). The 28F and 30F SP classes' Ex,min was corrected to 1.09 x 10^6 psi.

Verification examples

Verification status: 19 values compared in 10 examples (8 published, 2 hand calculations from the cited equations): 19 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

The char depths are NDS 2024's own tables, printed to 0.1 in.: Table 16.3.1 at 1, 1-1/2 and 2 hours (1.5 / 1.8, 2.1 / 2.5, 2.6 / 3.2 in.) and six entries of Table 16.3.2 for CLT. One CLT entry (1-1/2 hours, 1-1/4 in. laminations) prints 2.9 in. where Eq. 16.2-3 gives 2.85 in.; it is not used.

The member checks are hand calculations. No worked example of NDS Chapter 16 is on disk. AWC's Technical Report 10 and its Design for Code Acceptance 2 have them and are listed as needed. An 8x14 beam exposed on three sides for an hour and an 8x8 post exposed on four are checked by hand on the Verification page.

CHAPTER 70

Wood deck framing

A single-level residential deck: joists with an overhang, a built-up beam on posts, the posts, their footings, the ledger and a guard post. Each member is checked by NDS 2024 with ASCE 7-22 loads and compared with the prescriptive tables of AWC's deck guide, DCA 6-2015, which follows the IRC R507. LRFD or ASD.

Inputs of the deck

- **Lumber and loads:** Southern Pine, Douglas Fir-Larch, Hem-Fir or Spruce-Pine-Fir, and the grade (DCA 6 assumes No. 2). Wet service and incising are inputs: DCA 6 assumes wet service throughout and incising for Douglas Fir-Larch, Hem-Fir and Spruce-Pine-Fir. The dead and live load on the deck (DCA 6 and IRC: 10 and 40 psf).
- **Joists:** size, spacing, span L from the face of the ledger to the face of the beam, the overhang LO past the beam, and the concentrated load at the end of the overhang with its deflection limit (DCA 6: 220 lb, LO/180).
- **Beam:** two or three plies, the ply size, the span LB between posts and an overhang past the end post. The beam carries half the joist span plus the overhang, and the overhang's moment.
- **Posts and footings:** 6x6 or 8x8, the height from the top of the footing to the underside of the beam, Ke (1.0 pinned both ends), round or square footing size and thickness, depth of its bottom below grade, the frost depth, and the allowable soil bearing (DCA 6: 1,500 psf).
- **Ledger and guard:** a ledger deck with 1/2 in. lag screws, bolts, or bolts with stacked washers at an on-centre spacing, or a deck on its own beam at the house; a guard post (4x4 or 4x6) with its height and the 200 lb guard load.
- **Compare with the DCA 6 tables:** on by default. Spans, overhangs, post heights, footing sizes and ledger spacing beyond the tables then fail. Turn it off for an engineered deck and they are shown as notes. That choice is yours.

Auto-design (editor toolbar) varies the joist size and spacing, the beam ply size and plies, the post size and the footing size. Objective: the least wood.

Checks performed

Check	Basis
Joist bending, shear	NDS 3.3, 3.4 with CM, Ci, CF, Cr and the time factor of each combination; the overhang's moment with CL for its free bottom edge
Joist deflection	live load on the span against L/360; the overhang tip under the concentrated load, P LO² (L + LO) / (3 E' I), against LO/180
Beam bending, shear, deflection	NDS 3.3, 3.4, 3.5 on the plies acting together
Post	NDS 3.7 with CP over Ke times the height; le/d ≤ 50; the larger of an interior post and an end post with the beam overhang

Check	Basis
Footing	(P + footing weight) / area against the allowable bearing, D + L; bottom at least 12 in. below grade and below the frost line (IRC R403.1.4)
Guard post	200 lb at the top, bending (IRC Table R301.5); the 1,800 lb hold-down of DCA 6 Figure 26 as a note
DCA 6 comparison	Table 2 (joist span, overhang and L/4), Table 3A (beam span, overhang LB/4), Table 4 (post height, footing size and thickness), Table 5 (ledger spacing), 2x8 minimum for ledgers and guard-post joists

The drawings are the deck section along a joist (ledger, joist, beam, post, footing, guard post) and the beam elevation on its posts.

Limitations

- Decking, joist hangers, post caps, anchors, lateral load devices (IRC R507.2.4), stairs, and the house band joist are not designed. Design an engineered ledger with the wood ledger type and stairs with the wood stair type.
- Redwood, Western Cedars, Ponderosa Pine and Red Pine are not offered: their NDS values are not built in.
- Simple spans only. A beam continuous over posts, joists from both sides of a beam, hot tubs, concentrated loads and snow over 40 psf are outside the scope.
- DCA 6 tables are read for No. 2 lumber, 40 + 10 psf. Other inputs keep the comparison, with a note that it is a reference only.

Verification examples

Verification status: 22 values compared in 8 examples (7 published, 1 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

The table values are DCA 6-2015's own, as the type reads them. The member checks of the default deck are hand calculations. DCA 6 publishes rules and not calculations, so they are also read backward in the behaviour tests:

- Table 2's Southern Pine 2x10 at 16 in., 14 ft 0 in., sits at 0.996 of the NDS bending value (wet, repetitive, CD 1.0). The table is bending-controlled at its limit.
- A Douglas Fir-Larch No. 2 4x4 guard post, wet and incised, reaches 918 psi adjusted bending against 1,007 psi under 200 lb at 36 in. It misses the 1,100 psi DCA 6 asks for, and the check fails. A Southern Pine No. 2 4x4 passes at 1,100 psi.

No worked example of a deck by NDS is on disk, so this type should be read as hand verified against published tables.

CHAPTER 71

Split-ring and shear-plate connections

A bolted joint with split rings or shear plates in the side grain of the members, by NDS 2024 Chapter 13 with the adjustment factors of Chapter 11. LRFD or ASD.

Inputs of the connection

- **Connectors:** 2-1/2 in. split rings on 1/2 in. bolts, 4 in. split rings on 3/4 in. bolts, 2-5/8 in. shear plates on 3/4 in. bolts, or 4 in. shear plates on 3/4 or 7/8 in. bolts. Only the 4 in. shear plate offers a choice of bolt; it sets the metal limit of Table 13.2B footnote 2 (4,400 or 6,000 lb). Bolts, or lag screws with their penetration into the main member in shank diameters (Table 13.2.3 sets Cd).
- **Connectors in a row and rows:** a row runs along the load (11.3.6.2). Each connector on each bolt in each shear plane is one unit.
- **Members:** two members (one shear plane) or three (the main member with connectors on both faces). The side members are wood, or steel plates with shear plates. The species gives G, and G gives the species group of Table 13A. Net thickness and width set the tabulated value and the gross areas of the group action factor; E is for the group action factor.
- **Load angle to grain:** in the main member, and in a wood side member at its own angle.
- **Spacing, end and edge distances:** spacing in a row and between rows, the end distance from the square-cut end of the main member, whether the main member is in tension or compression at that end, and the unloaded and loaded edge distances. The values entered are for the main member; if a side member is tighter, enter its values.
- **Loads:** dead, live, snow and wind along the connection, combined by ASCE 7-22. Service conditions: moisture (Table 11.3.3) and temperature (Table 11.3.4). Load duration factors and the live-load time effect factor are in the code options.

Auto-design (editor toolbar) varies the connectors in a row and the number of rows, and the connector when you unlock it. Objective: the fewest connectors.

Checks performed

Check	Basis
Member thickness at least the tabulated minimum	NDS 13.2.2.1 (not feasible below it)
Reference P and Q per unit	Tables 13.2A, 13.2B by group, thickness and faces with connectors, interpolated between rows (13.2.2.2); the lower of the two members joined
Lag screw penetration	Table 13.2.3, Cd interpolated between 0.75 and 1.0
Steel side plates	Cst on P of 4 in. shear plates (Table 13.2.4); the metal limits of Table 13.2B footnote 2
Load at an angle to grain	Hankinson, Eq. 13.2-1, in each member at its own angle

Check	Basis
Geometry factor CΔ	Table 13.3, interpolated between the minimum and the full value; at an angle, separate factors on the parallel and perpendicular components and the spacing of Eq. 13.3-1 with Table 13.3.2.2; the least for any connector applies to all (13.3.2)
Group action Cg	Eq. 11.3-1 with γ = 500,000 lb/in. (4 in.) or 400,000 lb/in. (2-1/2 in. rings, 2-5/8 in. plates), gross areas; a member loaded across the grain by its thickness times the width of the group (11.3.6.3)
Connection capacity	units × N' against the load, every combination with its CD (≤ 1.6) or KF φz λ (λ ≤ 1.0 for connections)

The drawings are the joint elevation (connectors, bolts, end distance, spacing, edge distances, the side member behind) and a section through a connector row.

Limitations

- Connectors in end grain or on sloping cuts (13.2.6, 13.3.3) are not offered.
- The members are not checked: net section at the connector grooves (3.1.2, Appendix K), local stresses (13.3.4.3, Appendix E), and the members themselves. Neither is a steel side plate or the bolt bearing on it.
- The geometry entered applies to the main member; a side member with tighter distances is your check.
- The one-third increase of the shear plate metal limits for wind and seismic (footnote 2) is not applied. For LRFD, the limit is converted with KF φz λ; that form is to be confirmed against the code text.
- Two grooves cut concentric in one surface (13.3.4.2) are not offered.

Verification examples

Verification status: 21 values compared in 12 examples (9 published, 3 hand calculations from the cited equations): 21 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

The tabulated values and group action factors are NDS 2024's own tables: Table 13.2A and 13.2B rows (including the 4,400 lb limit of a 4 in. shear plate on a 3/4 in. bolt), and four entries of Table 11.3.6B. Thickness interpolation, the load at 45° and the geometry factor between the minimum and the full distances are hand calculations. No free worked example of a split-ring or shear-plate connection is on disk. AWC's Structural Wood Design Examples are listed as needed.

CHAPTER 72

Wood design values

The reference design values of every species and grade in the NDS 2024 Supplement, with every adjustment factor of NDS 2024 and the condition that sets it. The adjusted values are shown in ASD and LRFD side by side, with the specific gravity G for fastener design. Use it to look up a grade, to see what a service condition does to it, or to check actual stresses against the adjusted values.

Inputs of the explorer

- **Supplement table:**
  - Table 4A: visually graded dimension lumber, all species but Southern Pine (35 species groups, pp. 33-38).
  - Table 4B: Southern Pine and Mixed Southern Pine, with the dense and non-dense grades, by width (pp. 40-41).
  - Table 4C: MSR and MEL lumber (pp. 43-44). The species group sets G, Fv and Fc⊥ by footnote 2 (p. 45).
  - Table 4D: timbers, beams and stringers or posts and timbers by size (pp. 47-52).
  - Table 4E: decking (pp. 54-56).
  - Table 4F: non-North American dimension lumber (pp. 58-60).
  - Glulam: the Table 5A stress classes (p. 80), the Table 5A Expanded combinations (pp. 81-82) and the Table 5B axial combinations (p. 84).
  - Values entered: SCL from its evaluation report, Table 4G multi-species lumber, or any other product.
- **Species, grade and size:** the lists hold every species and every grade printed in the table. A grade the species does not have is reported, with the grades it has. Where two grading agencies print different grade lists for one species (Table 4D Douglas Fir-Larch and Sitka Spruce), the species appears once for each agency. The nominal size gives the dressed size (Supplement Table 1A) and, for timbers, the size classification (NDS 4.1.3). Glulam and entered values take the actual width and depth.
- **Service conditions:**
  - Load duration CD (ASD, NDS Table 2.3.2) and time effect factor λ (LRFD, Table N3).
  - Wet service: moisture content over 19 % for sawn lumber, 16 % or more for glulam.
  - Temperature, sustained exposure (Table 2.3.3).
  - Incising (Table 4.3.8), repetitive member use (4.3.9), and edgewise or flatwise bending.
- **Stability and bearing:**
  - The unbraced length of the compression edge and the load case of Table 3.3.3 for CL.
  - For glulam, the length between points of zero moment for CV.
  - The column effective lengths about each axis for CP.
  - The sheathed 2x4 truss chord option for CT.
  - The bearing length and whether it is at the member end, for Cb.
- **Actual stresses (optional):** fb, ft, fv, fc⊥ and fc. Each is compared with the adjusted value of the chosen method; 0 means not checked.

There is nothing to size, so the type has no auto-design. It is a reference calculation: its status reads "Calculated".

Checks performed

Value	Factors applied	Basis
F'b	CD or KF φ λ, CM, Ct, CL, CF, Cfu, Ci, Cr	NDS Table 4.3.1; CL by Table 3.3.3 and Eq. 3.3-6 with E'min. Glulam (Table 5.3.1): the lesser of CL and CV (5.3.6), Cfu on Fby
F't	CD or KF φ λ, CM, Ct, CF, Ci	Table 4.3.1 / 5.3.1
F'v	CD or KF φ λ, CM, Ct, Ci	Table 4.3.1 / 5.3.1
F'c⊥	KF φ, CM, Ct, Ci, Cb	No CD and no λ; Cb by Eq. 3.10-2
F'c	CD or KF φ λ, CM, Ct, CF, Ci, CP	CP by Eq. 3.7-1, c = 0.8 sawn, 0.9 glulam and SCL; the lower of the two axes
E', E'min	CM, Ct, Ci; CT on E'min; KF φ on E'min (LRFD)	Eq. 4.4-1 for CT
Size factor CF	Supplement Table 4A / 4F (pp. 32, 57) by grade, width and thickness, including Stud, Construction, Standard and Utility; Table 4B (p. 39); Table 4D (12/d)^(1/9) (p. 46); Table 4E by thickness (p. 53); none for Table 4C	Supplement adjustment factor pages
Flat use Cfu	Tables 4A, 4B, 4F by size; 4C; beams and stringers of Table 4D on Fb, E and Emin; glulam Fby by width	NDS 4.3.7, Supplement pp. 32, 39, 42, 46, 57, 79, 83
Wet service CM	Each table's factors and thresholds; Southern Pine timbers and green Southern Pine decking take no further factor	Supplement pp. 32, 39, 42, 46, 53, 57, 79
Column slenderness	le/d ≤ 50	NDS 3.7.1.4
Beam slenderness	RB ≤ 50	NDS 3.3.3.7
Specific gravity G	The table's G column; Table 4C footnote 2 for MSR and MEL by species and E; Tables 5A and 5B for glulam	For the dowel bearing strength; NDS Table 12.3.3A lists G by species combination

Every value lists its reference value and table page, each factor with its condition, and the ASD and LRFD results. The drawing is the member section with its size, species, grade and G.

Limitations

- **Not built in:**
  - Table 4G, multi-species and country grademarked lumber. Its rotated pages print the species groups as running text, which could not be attributed to their rows reliably. Enter those values instead.
  - Tables 5C and 5D (hardwood glulam), and Tables 6A and 6B (round timbers).
  - SCL. Enter it from the evaluation report; its volume factor comes from the report.
- **Glulam factors not computed:** the curvature factor Cc, the stress interaction factor CI and the shear reduction factor Cvr. The Table 5A footnote 4 and 5 shear reductions are not applied either.

- **Our readings:**
  - Table 4D gives flat use factors for Select Structural, No. 1 and No. 2 only, so the dense grades take the factor of their base grade.
  - A Table 4D species with one value set for both size classes ("5 in. x 5 in. and larger") takes the beams-and-stringers flat use factor when its size is a beam size.
  - The glulam flat use factor of a width between two table rows is that of the next smaller tabulated width.
- **A note from the reading:** Table 5A prints Ex,min = 1.09 x 10^6 psi for the 28F-2.1E and 30F-2.1E SP stress classes. Table 5A Expanded prints 1.11 x 10^6 psi for the same classes. The stress classes here use 1.09, the lower value.
- **Effect on other wood types:** reading the full tables corrected values that other wood types use (in their own commits):
  - Table 4B Southern Pine No. 2, 5-6 in. wide, is Ft = 600 psi. It was 575 psi.
  - The 28F and 30F glulam stress classes now use Ex,min = 1.09 x 10^6 psi.
  - 24F-V8 DF/DF glulam is Fby = 1,550 psi. It was 1,450 psi.

Verification examples

Verification status: 43 values compared in 15 examples (6 published, 9 hand calculations from the cited equations): 43 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

The published examples are AWC Structural Wood Design Examples E1.4 and E1.3, as tabulated in the free Trimble Tekla Tedds verification examples:

- **E1.4:** 4x4 and 6x6 Southern Pine No. 2 posts, 10 ft long: Fc*, CP and F'c.
- **E1.3:** a 5 x 22 24F-E1 SP/SP glulam: RB, Fbx*, CV and F'bx. The example uses NDS 2018 Ex,min for CL, while NDS 2024 uses Ey,min (5.2.7). CV governs here, so F'bx is the same either way.

Table values (Tables 4B, 4C with footnote 2, 5A Expanded) are compared as published. The adjustment chains (wet, incised, repetitive, CL, Utility size factors, MSR flat use, timber CF, decking, glulam Cfu, Cb and CT) are hand calculations from the cited factors. No published worked example covers those combinations on disk.

CHAPTER 73

Wood shrinkage and moisture

How much wood framing shrinks as it dries in service, piece by piece and level by level up a multi-story wood frame, by the method of the USDA Forest Products Laboratory Wood Handbook (FPL-GTR-282). The shrinkage of each story and the cumulative shrinkage at the top are checked against the movements you allow. US and metric units.

The Wood Handbook is not in our reference library yet. Its coefficients are built in from knowledge and marked "to confirm against the code text" wherever they appear.

Inputs of the shrinkage calculation

- **Species:** the FPL species (Douglas-fir, larch, hemlock, the true firs, the southern pines, lodgepole and ponderosa pine, the spruces, redwood, western redcedar), or **Other** with your own values. The commercial groups map to them: Douglas Fir-Larch is Douglas-fir and western larch, Hem-Fir is western hemlock and the true firs, SPF is the spruces, lodgepole pine and subalpine fir, and Southern Pine is loblolly, longleaf, shortleaf and slash.
- **Coefficients from:**
  - the dimensional change coefficients CR (radial) and CT (tangential) per percent of moisture content (Handbook Table 13-5), or
  - the total shrinkage green to oven-dry, SR and ST (Table 4-3), spread evenly below the fiber saturation point: C = S / (100 fs).
- **Across the grain:** framing lumber comes with any ring orientation.
  - The **average** of radial and tangential is common practice.
  - **Tangential** is the upper bound (flat-sawn).
  - **Radial** is the lower bound (edge-grain).
- **Longitudinal shrinkage SL:** green to oven-dry along the grain. The Handbook gives 0.1 to 0.2 % for normal wood; juvenile and compression wood shrink more. It sets the coefficient of studs and posts, CL = SL / (100 fs).
- **Fiber saturation point fs:** about 30 %. Wood does not shrink above it, so the moisture contents are capped at fs.
- **Moisture content at installation Mi:** the lumber as delivered.
  - S-DRY: at most 19 %.
  - KD-15 or MC15: at most 15 %.
  - S-GRN: above 19 %; take fs.
  - Rain during framing raises it.
- **Equilibrium moisture content in service Mf:** the Handbook's values for the interior of heated buildings (Table 13-1) are about 8 % in most of the United States, 6 % in the dry southwest and 11 % in damp, warm coastal areas. Framing commonly ends between 8 and 12 %.
- **Framing stack:** every piece in the vertical load path, from the foundation up, one row per piece. Each row gives:
  - the level (1 is the lowest);

    - the piece: sill, bottom or top plate, rim or band joist, floor joist bearing on the wall, beam or header, blocking, studs, a post, or engineered wood;
    - the number of pieces (two top plates: 2);
    - its vertical dimension (a plate's thickness, a rim's depth, a stud's length);
    - optionally its own coefficient C, required for engineered wood (from the manufacturer; LVL, LSL and I-joist rims installed near their equilibrium moisture content are often taken near zero), and its own initial moisture content.

  A floor's rim or joists belong to the level below them.
- **Movement limits:** both are your criteria and are checked by default.
    - **Allowed shrinkage of one story:** what the connections at each floor take (slip joints in risers and ducts, cladding joints).
    - **Allowed cumulative shrinkage:** the differential movement allowed where the frame meets something that does not shrink: a concrete or masonry core, a steel column or frame, an elevator, a podium stair, or a take-up device's travel.

There is no auto-design: the calculation reports movements. The pieces and the moisture contents are the design, and you change them.

Checks performed

Check	Basis
Dimensional change coefficients	Handbook Table 13-5 (CR, CT) or Table 4-3 (SR, ST, SV) by species; across the grain by the orientation chosen; along the grain from SL
Moisture content change	ΔM = min(Mi, fs) − min(Mf, fs); a piece with its own Mi uses it; swelling (Mf above Mi) is flagged
Shrinkage of each piece	ΔD = n D C ΔM (the Handbook's dimensional change coefficient form, Ch. 13)
Shrinkage of one story	The largest story against your limit (serviceability)
Cumulative shrinkage	The largest cumulative value, at the top of the stack, against your limit (serviceability)
Engineered wood	Not feasible without its coefficient
IBC 2304.3.3	A note from three levels up: walls supporting more than two floors and a roof need a shrinkage analysis

The drawing is a schematic section of the framing stack. It shows each level's true height, its shrinkage and the cumulative shrinkage at its top. It is not to scale: the levels are drawn equal and the pieces across the grain are drawn thicker.

Limitations

- **Coefficients from knowledge:** the Handbook values (Tables 4-3, 13-1 and 13-5) and the form of the method are built in from knowledge, to be confirmed against FPL-GTR-282. So is the IBC 2304.3.3 wording.
- **Coefficient range:** the Table 13-5 coefficients are stated for 6 to 14 % moisture content. Used from 19 % down, as is common practice, they are an approximation. The total shrinkage form spreads the green to oven-dry shrinkage evenly below fs.

- **Not included:**
  - elastic shortening and creep under load;
  - crushing and seating at plates and bearings;
  - construction gaps;
  - thermal movement;
  - the drying time.
- **Shrinkage only:** the movement of the non-shrinking element is not computed. The cumulative value is the wood's movement relative to the foundation.
- **Uniform moisture:** one species and one equilibrium moisture content for the whole stack. A piece of another species can be entered with its own coefficient.

Verification examples

Verification status: 10 values compared in 8 examples (0 published, 8 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example is on disk: the Wood Handbook itself is listed as needed. Every value is a hand calculation of $\Delta D = n\ D\ C\ \Delta M$ with the coefficients the module carries:

- one block, radial and tangential;
- the total-shrinkage form;
- green lumber capped at fs;
- the default three-story stack, level by level and cumulative;
- an engineered rim with its own coefficient and moisture content;
- a Southern Pine rim;
- swelling.

CHAPTER 74

Steel side and knife plate connections

Bolts or drift pins through a wood member with steel plates. The plates are two side plates, one side plate, or one knife plate let into a kerf in the wood. The calculation covers the fasteners in the wood (NDS 2024 Chapters 11 and 12), the local stresses in the wood around the group (11.1.2 and Appendix E) and the steel plate (AISC 360-22 J3 and J4). LRFD or ASD.

Inputs of the connection

- **Arrangement:**
  - Two steel side plates on a wood main member: double shear.
  - One steel side plate: single shear.
  - One knife plate in a kerf: double shear, with the plate as the main member and the wood on each side of the kerf as the side members.
- **Fastener:**
  - Bolts, with the bolt grade for the steel shear check, or drift pins at 75 % of a bolt (12.3.10).
  - Diameter 1/2 in. to 1 in., and the bending yield strength Fyb (45,000 psi for bolts).
- **Plate:** thickness, width, kerf width (usually the plate plus 1/8 in.), Fy and Fu. Fu sets the dowel bearing strength of the steel, Fe = 1.5 Fu.
- **Pattern:**
  - Rows across the member and fasteners in each row along it, with the spacing in a row and between rows.
  - Staggered rows: alternate rows have one fewer fastener, offset by half the spacing.
  - The end distance in the wood, whether the member is in tension or compression at that end, the edge distance of the outer rows, and the plate end distance.
- **Wood member:**
  - Sawn lumber (Douglas Fir-Larch, Hem-Fir, Spruce-Pine-Fir or Southern Pine, Select Structural to No. 2).
  - A glulam stress class (Table 5A) or axial combination (Table 5B), with its thickness and width.
  - Or Ft, Fv, E and G entered.
  - For glulam, the shear reduction factor Cvr = 0.72 at connections (NDS 5.3.10) is on by default.
- **Loads:** dead, live, snow and wind along the member, combined by ASCE 7-22, and the load angle to grain.
- **Service:** moisture (Table 11.3.3), temperature (Table 11.3.4), and the local stress check (NDS 11.1.2, on by default). Load duration factors and the time effect factor are in the code options.

Auto-design (editor toolbar) varies the fastener diameter, the fasteners per row and the rows. The arrangement, plate, member and spacings stay as entered. Objective: the least fastener steel, or the fewest fasteners.

Checks performed

Check	Basis
Kerf at least the plate, wood left on each side	Input
Rows fit the member and the plate	Edge distances and row spacing
Reference Z of one fastener	NDS Table 12.3.1A yield modes (single or double shear), steel Fe = 1.5 Fu, wood Fe by Hankinson, Rd with Kθ (Table 12.3.1B)
Adjusted group capacity n Z'	CD (≤ 1.6) or KF φz λ (λ ≤ 1.0), CM, Ct, Cg (Eq. 11.3-1, γ = 270,000 D^1.5 wood to metal), CΔ (Tables 12.5.1A, 12.5.1B), drift pins × 0.75; every load combination
End distance, spacing, edge distance, row spacing	Tables 12.5.1A to 12.5.1D; the 5 in. limit between outer rows (12.5.1.3) as a note
Net section tension of the wood	Appendix E, Eq. E.2-1, every row deducted, net of the kerf
Row tear-out	Eqs. E.3-1 to E.3-3: n F'v t s_critical per row, s_critical the lesser of the end distance and the spacing
Group tear-out	Eq. E.4-1: half of each outer row's tear-out plus F't on the net area between the outer rows
Bolt shear	AISC 360-22 J3.7, Table J3.2, per shear plane
Bolt bearing and tear-out in the plate	J3.11(a)(1): 1.2 lc t Fu ≤ 2.4 d t Fu per bolt; each of two side plates takes half the load
Plate tension yielding and rupture	J4.1 (holes 1/16 in. over the standard hole, U = 1.0)
Plate block shear	J4.3 along the two outer bolt lines

The drawings are the joint elevation (the member end, the plate, the fasteners, end distance, spacing, edge distances and the load) and a section through a row (plates, wood, kerf and fastener).

Limitations

- **Not designed:**
  - Two or more knife plates (multiple shear planes, 12.3.8). Lag screws and dowels driven into blind holes.
  - The plate's connection beyond the joint (welds, other bolts) and the plate in compression (buckling between bolts).
  - The member away from the joint.
- **Local stresses** (Appendix E) are for load parallel to grain. With a load at an angle, check the member's shear at the connection (3.4.4) and splitting in the member's own calculation.
- **Our readings:**
  - With staggered rows, every row is deducted for the net section, as Appendix E sample E.7 does.
  - For Cg, Am and As are divided by the number of rows.
  - Staggered rows closer than a quarter of the distance between their nearest fasteners act as one row (11.3.6.3).
  - The member's F't and F'v use the connection's CD (at most 1.6) or λ (at most 1.0): conservative where a member could take more.

- **Edge distance:** checked with the parallel-to-grain requirement for loads along the member. The loaded edge requirement (4D) applies when the load has a perpendicular component.

Verification examples

Verification status: 16 values compared in 4 examples (1 published, 3 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not yet recorded for this module.

The published example is NDS 2024 Appendix E sample solution E.7 (pp. 176-177): staggered 1 in. bolts through a combination 2 Douglas Fir glulam with 1/4 in. steel side plates. It checks Z, the row tear-out of each row and in total, the net section and the group tear-out. The example rounds F'v to 191 psi, 0.1 % above the unrounded 190.8 psi.

The default knife plate's yield modes, a sawn member with two side plates, and the default plate's AISC checks are hand calculations. No published knife-plate example is on disk.

CHAPTER 75

Moment and eccentric fastener groups

A group of bolts or lag screws joining a side member (wood or steel) to a main member under moment and eccentric shear: knee braces, portal frame knees, brackets. It uses the elastic method with each fastener checked at its own angle to the grain of each member, by NDS 2024 Chapters 11 and 12. LRFD or ASD.

Inputs of the moment group

- **Fasteners:** bolts (single or double shear) or lag screws (single shear) with their diameter and, for lag screws, their length.
- **Pattern:** fasteners along the main member's grain and rows across it, with their spacings.
- **Members:**
  - **Shear planes and side members:** single or double shear; wood side members, or steel plates (Fe = 1.5 Fu, Appendix I).
  - **Species and thicknesses:** G gives the dowel bearing strengths.
  - **Angle β:** the side member's grain to the main member's grain (45° for a knee brace to a post).
- **Loads:**
  - **P by type** (dead, live, snow, wind) at its direction α from the main member's grain, acting at the eccentricity ex, ey from the group centroid.
  - **Moments by type** at the centroid.
  - Combined by ASCE 7-22.
- **End and edge distances:** the main member's end distance (0 for a continuous post) and edge distance, and the side member's end distance along its grain and its edge distance.
- **Service conditions:** moisture (Table 11.3.3) and temperature. Load duration is in the code options.

Auto-design (editor toolbar) varies the fasteners along the grain, the rows and the diameter. Objective: the fewest fasteners, then the smaller diameter.

Checks performed

Check	Basis
Fastener forces	Elastic method: P/n plus M r / J perpendicular to each radius, J = Σ(x² + y²)
Each fastener's Z	Yield limit equations (Table 12.3.1A) with Feθ by Hankinson (Eq. 12.3-11) at the fastener's own angle to each member's grain, Rd with Kθ at the larger angle (Table 12.3.1B)
Governing fastener	The largest load over adjusted capacity among all fasteners, not only the most loaded one
Adjustment	CD ≤ 1.6 or KF φz λ (λ ≤ 1.0), CM, Ct, CΔ by Tables 12.5.1A and 12.5.1B in each member for the governing fastener's direction, the least applied to all
Group action	Cg = 1.0 (see Limitations)

Check	Basis
Edge distance and row spacing	Table 12.5.1C (4D at a loaded edge across the grain), Table 12.5.1D
Lag screw penetration	At least 4D without the tip (12.1.4.6)

The drawings are the joint (members, pattern, governing fastener, the load at its eccentricity) and a section through the joint.

Limitations

- **Group action:** Cg = 1.0 is our reading. Eq. 11.3-1 is derived for rows of fasteners sharing an axial load through stretching members, while the forces of an elastic moment group follow its rotation. For a group loaded mainly in direct shear along a row, use the wood fastener connection calculation with Cg.
- **Direction for CΔ:** parallel below 5°, perpendicular above 85°, the lesser of both between; our reading.
- **Side member spacing:** the side member's grain is not aligned with the pattern, so its spacing is the least center-to-center distance of the pattern; our reading.
- **Not designed:**
  - the members' net section and moment capacity at the joint;
  - local stresses (Appendix E);
  - the steel plates and the fasteners' steel strength (AISC 360);
  - withdrawal (lag screws loaded laterally only);
  - the plastic (instantaneous center) distribution.

Verification examples

Verification status: 10 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values: NDS Table 12A reference values (a single bolt along and across the grain of each member). The elastic method on a single fastener reduces to the table's connection.

Hand calculations: a 2 × 2 group under a pure moment (every bolt at 45° to the grain) and the default knee brace group.

No worked example of an eccentric wood fastener group is on disk. Breyer's Design of Wood Structures and AWC's Structural Wood Design Examples are listed as needed.

CHAPTER 76

Timber rivet connections

Timber rivets (glulam rivets) driven through steel side plates into structural glued laminated timber, by NDS 2024 Chapter 14 with the adjustment factors of Chapter 11. LRFD or ASD.

Inputs of the rivet connection

- **Rivets and plates:**
  - **Rivet length:** 1-1/2, 2-1/2 or 3-1/2 in., the lengths of Tables 14.2.1A to 14.2.1F. The penetration is p = length − plate thickness − 1/8 in. (14.2.1).
  - **Rows across the grain** and **rivets per row along the grain:** for a load along the grain these are the code's nR and nC. For a load across the grain the roles swap: the rows parallel to the load run across the grain.
  - **Spacing along the grain sp:** 1 or 1-1/2 in., the values the tables give. The code minimum is 1 in. (14.3.1).
  - **Spacing across the grain sq:** 1 or 1-1/2 in. The code minimum is 1/2 in., but Tables 14.2.1A to 14.2.1F are for 1 in. and Table 14.2.2A for 1 or 1-1/2 in.
- **Side plates:** one plate, or plates on opposite faces. The plate thickness is ASTM A36, at least 1/8 in. A plate thinner than 1/4 in. reduces the rivet capacity by Cst (Table 14.2.3).
- **Member:** Douglas Fir-Larch or Southern Pine glulam, the only members Chapter 14 covers (14.1.1).
  - **Thickness:** with plates on both faces, the tables are entered with the thickness; with one plate, with twice the thickness.
  - **Depth at the joint:** across the grain. With the unloaded edge distance and the rows, it gives the far edge distance.
- **Load angle to grain.**
- **End distance:** from the member end to the nearest row.
- **Unloaded edge distance ep:** for a load across the grain it also sets the geometry factor CΔ (Table 14.2.2B).
- **Loads:** dead, live, snow and wind on the connection, combined by ASCE 7-22.
- **Service conditions:**
  - **Moisture:** CM = 0.8 in service wet (Table 11.3.3). Table 11.3.3 gives no factor for rivets fabricated wet, so that case is not feasible.
  - **Temperature:** Ct of Table 11.3.4.
  - **Load duration:** the load duration factors and the live-load time effect factor are in the code options.

Auto-design (editor toolbar) varies the rivets per row, the rows and the rivet length. Objective: the fewest rivets, then the shorter rivet. The member, plates, spacings, distances and loads stay yours.

Checks performed

Check	Basis
Rivet capacity parallel to grain	Pr = 188 p^0.32 nR nC (Eq. 14.2-1), per plate
Wood capacity parallel to grain	Pw from Table 14.2.1A to 14.2.1F by rivet length and sp, member dimension (thickness, or twice it with one plate), rivets per row and rows; linear interpolation as the tables' note permits
Rivet and wood capacity perpendicular to grain	Qr = 108 p^0.32 nR nC (Eq. 14.2-2); Qw = qw p^0.8 CΔ (Eq. 14.2-3) with qw from Table 14.2.2A and CΔ from Table 14.2.2B at ep / ((nC − 1) sq), interpolated
Metal side plate factor	Cst (Table 14.2.3) only when the rivet capacity controls; CΔ only on the wood capacity (Table 11.3.1 footnotes 4 and 5)
Load at an angle to grain	N' = P'Q' / (P' sin²θ + Q' cos²θ) (Eq. 14.2-4)
Connection capacity	Plates × N' against the load, every combination with CD (≤ 1.6) or KF φz λ (λ ≤ 1.0); no group action factor (Table 11.3.1)
Penetration	At most 70 % of the member thickness (14.1.3.2); with plates on both faces the points must not overlap
Plate thickness	At least 1/8 in. (14.1.2.2)
Spacing	Outermost rows at most 12 in. apart across the grain (14.3.1)
End and edge distances	Table 14.3.2: end distance ap (parallel) or aq (perpendicular) by the number of rows, unloaded edge ≥ 1 in., loaded edge ≥ 2 in.

The drawings are the joint elevation (the plate over the rivet group, the rows, end and edge distances, the load) and a section through the member with the plates and the rivet penetration.

Limitations

- **Not designed:**
  - the steel side plates and their connection to the rest of the structure (14.2.6; AISC 360);
  - the member's net section and local stresses (Appendix E);
  - the member itself.
- **Not offered:** rivets in end grain (14.2.5) and rivets from opposite faces whose points overlap (14.1.3.3).
- **Tabulated values only:** spacings, rivet lengths and group sizes outside the tables are not feasible. This includes a load across the grain with sp other than 1 in. (Table 14.2.2A is for sp = 1 in.). The tables are not extrapolated.
- **Geometry factor:** CΔ is interpolated linearly between the rows of Table 14.2.2B. This is our reading: the code gives the table without saying how to read between rows.
- **Scope of Chapter 14:** Douglas Fir-Larch and Southern Pine glulam only. Sawn lumber and CLT are outside it.

Verification examples

Verification status: 17 values compared in 8 examples (5 published, 3 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values: the NDS 2024 tables themselves, read from the page images of the 2024 scan:

- Pw of Tables 14.2.1A and 14.2.1C, including one plate entered with twice the member thickness;
- qw of Table 14.2.2A;
- CΔ of Table 14.2.2B;
- Cst of Table 14.2.3;
- the end distance of Table 14.3.2.

Hand calculations: the rivet capacity Pr, the wood capacity Qw, interpolation between member dimensions, a 3/16 in. plate, a load at 45° and the default connection in ASD and LRFD.

Neither the NDS nor its Commentary has a worked timber rivet example. AWC's Structural Wood Design Examples are listed as needed.

CHAPTER 77

Notched and tapered member ends

The end of a simple-span sawn lumber or glulam bending member, by NDS 2024 3.2.3, 3.4.3, 3.4.4, 4.4.3 and 5.4.5. The end is either notched or taper cut over its bearing, or hung on a connection. An interior notch can be added. The shear at the notch, the bearing, the bending at the reduced section and the code's notch limits are checked under the member's own reactions. LRFD or ASD.

Inputs of the notched end

- **Member:** sawn lumber (species, grade, nominal size from the Supplement Table 4A, 4B and 4D values) or glulam (stress class, width and depth; Supplement Table 5A values, read 2026-11-12). SCL, I-joists and CLT follow their manufacturers' notch rules and are not offered.
- **Span and loads:**
  - **Span:** centre to centre of the bearings, the clear span plus half the bearing at each end (3.2.1).
  - **Loads:** uniform dead, live, roof live and snow loads in plf, including the member's weight, combined by ASCE 7-22. The reaction at each end is w L / 2.
- **Member end:**
  - **No notch:** bearing at the end.
  - **Notch on the tension face:** the bottom cut over the bearing, so the member bears at dn.
  - **Notch on the compression face:** the top cut at the end, running e past the inner edge of the support.
  - **Taper cut on the compression face:** the top cut from the member end down to full depth at e past the support. Its depth is the depth removed at the member end; dn is taken at the inner edge of the support (3.4.3.1(e)).
  - **Hung on a connection:** bolts, lag screws, split rings or shear plates (3.4.4), with the effective depth de and the distance from the member end. de is the depth less the distance from the unloaded edge to the nearest bolt or lag screw centre, or to the nearest edge of a ring or plate (Figure 3E).
  - A taper cut on the tension face is entered as a tension-face notch, as 3.4.3.1(c) treats it.
- **Notch depth (d − dn):** your input, set by the support detail (a seat or ledge height, a flush top). Auto-design keeps it.
- **Bearing length lb:** at the member end, so Cb = 1.0 (3.10.4).
- **Interior notch:** sawn lumber only.
  - Its face: tension, only for members thinner than 3-1/2 in.; or compression.
  - Its distance from the bearing centre to its near edge, its depth and its length.
- **Service conditions:** wet service and temperature. The load duration factors and the live-load time effect factor are in the code options.

Auto-design (editor toolbar) varies the sawn size, or the glulam width and depth. Objective: the least wood. The notch depth, its length, the bearing, the connection, the span and the loads stay yours. A member too shallow for the notch fails the code limit, and a deeper one is found.

Checks performed

Check	Basis
Shear at a tension-face end notch	Vr' = (2/3) F'v b dn (dn/d)², Eq. 3.4-3 (3.4.3.1(a))
Shear at a compression-face end notch or taper cut	Vr' = (2/3) F'v b [d − ((d − dn)/dn) e] for e ≤ dn, Eq. 3.4-5; (2/3) F'v b dn for e > dn (3.4.3.1(e))
Shear at a connection	(2/3) F'v b de (de/d)² within 5d of the end; (2/3) F'v b de beyond (3.4.4.1, Eqs. 3.4-6 and 3.4-7)
Shear in glulam	Cvr = 0.72 at notches, taper cuts and connections (5.3.10 items 3 and 4, 5.4.5.4)
Bearing	fc⊥ = R / (b lb) against F'c⊥ (3.10.2), Cb = 1.0 at the end
Bending at the reduced section	6M / (b dn²) at the end of a compression-face notch, along a taper, and at an interior notch (3.1.2), against Fb' with CL = 1.0 (CV for glulam)
Sawn notch limits	End notches and taper cuts ≤ d/4 (4.4.3.1); compression-face end notches and tapers out of the middle third (Figure 4B)
Glulam notch limits	Tension-face end notch ≤ d/10 and ≤ 3 in. (5.4.5.1); compression-face end notch ≤ 2d/5, ≤ 3 dn from the end and out of the middle third (5.4.5.2, Figure 5F); compression-edge taper ≤ 2d/3 deep and ≤ 3d long (5.4.5.2 Exception)
Interior notch	Sawn only, outer third of the span, ≤ d/6, not on the tension face of members 3-1/2 in. or thicker (4.4.3.2); none in glulam (5.4.5.1, 5.4.5.2); shear by Eq. 3.4-3 (tension face, as Commentary C3.4.3.1(a) permits) or 3.4.3.1(f) (compression face); stiffness note (3.2.3.2)

The shear is taken at the support and equals the reaction, without the reduction for loads near supports (3.4.3.1 and 3.4.2.2 apply that reduction only to members without notches). The drawings are:

- the end elevation (the notch or taper, the bearing block, or the connection);
- the interior notch, when there is one;
- the member section.

Limitations

- **The member away from its ends:** flexure with CL, unnotched shear and deflection are the member's own calculation (wood beam, glulam beam).
- **Simple span only:** uniform loads by type; no point loads.
- **Bending at the notch:** taken with CL = 1.0. The member's lateral stability is checked in its own calculation.
- **Measurements:**
  - The middle-third limit and the glulam 3 dn and 3d lengths are measured from the member end (lb + e), as Figure 5F draws them. This is on the safe side of the middle-third rule.
  - The sawn taper limit (d/4) is checked on the depth removed at the member end. This is also on the safe side: Figure 4B shows dn at the bearing.
- **Not offered:** circular members (Eq. 3.4-4), notches in SCL, I-joists and CLT, notches on both faces at one section (prohibited for glulam by 5.4.5.3).
- **Glulam values:** the Table 5A values (Fvx, Fc⊥x, Fbx) were read against the Supplement on 2026-11-12 (p. 80).

- **Equation numbers:** the NDS 2024 scan numbers two equations 3.4-6 (in 3.4.3.1(f) and in 3.4.4.1(a)). The steps name both the equation and its clause.

Verification examples

Verification status: 15 values compared in 8 examples (0 published, 8 hand calculations from the cited equations): 15 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

Neither the NDS 2024 nor its Commentary works a notched member example, and AWC's Structural Wood Design Examples are listed as needed. Every value is a hand calculation of the code's equations with the Supplement values (Douglas Fir-Larch No. 2) and, for glulam, Table 5A 24F-1.8E (to be confirmed against the code text). They cover:

- a tension-face notch in LRFD and ASD;
- the bearing;
- a compression-face notch with e ≤ dn and with e > dn, and its net section;
- a glulam tension-face notch with Cvr;
- a connection near and far from the end;
- interior notches on both faces.

CHAPTER 78

Wood floor vibration

Walking vibration of a one-way wood joist floor (sawn joists, I-joists or floor trusses) against the criterion you choose. US and metric units.

- **Live load deflection:** limit L/Δ.
- **Onysko / CCMC:** the 1 kN point-load deflection.
- **Hu and Chui (FPInnovations):** the frequency-deflection ratio.
- **Frequency:** an optional minimum.

Only the APA L/480 basis is from a document on disk. The Onysko and Hu-Chui criteria and the frequency minimum are built from knowledge and marked "to confirm against the code text".

Inputs of the floor

- **Joists:**
  - sawn lumber (species, grade, size; E from Supplement Table 4A or 4B); or
  - I-joists or floor trusses, with the EI of one joist from the manufacturer's evaluation report.
- **Span:** simple span.
- **Spacing:** joist spacing.
- **Floor weight:** the actual dead load for the frequency, not the design value.
- **Live load:** for the deflection.
- **Vibration criterion:** your choice.
  - **Onysko / CCMC:** the deflection under 1 kN (225 lb) at midspan must not exceed 8 / L^1.3 mm (L in m) and 2 mm.
  - **Hu and Chui:** f1 / d1^0.44 ≥ 18.7, with f1 in Hz and d1 the 1 kN deflection in mm.
  - **Deflection only.**
- **Share of a point load on one joist:** your judgment. 1.0 is a joist alone. About 0.3 to 0.5 applies where glued sheathing, bridging or strongbacks spread the load. The published methods take the floor system from tests and their own load-sharing models, which this calculation does not reproduce.
- **Live load deflection limit:** L/480, the APA Performance Rated I-Joist basis for better floor performance (APA Z725H), or L/360 (IBC Table 1604.3).
- **Minimum frequency:** optional; 15 Hz is commonly recommended.

Auto-design (editor toolbar) varies the sawn joist size and the spacing (only the spacing for I-joists and trusses). Objective: the least wood per foot of floor.

Checks performed

Check	Basis
Live load deflection	$5wL^4/(384EI)$ against L/limit
1 kN deflection	$k P L^3/(48EI)$, with k the load share, against min(8/L^1.3, 2) mm (Onysko / CCMC)
Frequency-deflection	$f1 = (\pi/2L^2)\sqrt{(EI\ g/w)}$ with the floor weight on one joist; f1/d1^0.44 against 18.7 (Hu and Chui)
Frequency	f1 against your minimum (optional; otherwise reported)

The drawings are a framing plan of a bay (the joists, the spacing and span, the 1 kN load) and the joist section.

Limitations

- **Criteria from knowledge:** the Onysko and Hu-Chui criteria and their constants are to be confirmed against their sources (Onysko 1985 and the CCMC / NBC span tables; Hu and Chui 2004, FPInnovations).
- **Load sharing:** the share of a point load is your input, not the methods' own models.
- **Not covered:**
  - two-way action;
  - composite toppings and finishes;
  - continuous joists;
  - damping;
  - the AISC Design Guide 11 method (the steel floor vibration type);
  - I-joist shear deflection beyond the entered EI.
- **Strength:** the joists' strength is checked in the wood beam or I-joist calculation.

Verification examples

Verification status: 8 values compared in 4 examples (1 published, 3 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not yet recorded for this module.

The default deflection limit is APA's published L/480 basis. Everything else is a hand calculation with the criteria as built:

- a 2x10 Douglas Fir-Larch No. 2 floor: deflection, 1 kN deflection, Onysko limit, frequency and the Hu-Chui ratio;
- an I-joist with its entered EI.

No published wood floor vibration example is on disk. The FPInnovations and CCMC documents are listed as needed.

CHAPTER 79

Built-up and spaced columns

Wood columns made of several pieces, by NDS 2024 15.3 and 15.2, with ASCE 7-22 combinations. LRFD or ASD. A built-up column is two to five laminations nailed or bolted together. A spaced column is two or three members held apart by blocks and joined at the ends by split rings or shear plates in end blocks.

Inputs of the column

- **Column:** nailed built-up (15.3.3), bolted built-up (15.3.4) or spaced (15.2).
- **Laminations or members:** sawn lumber of one species, grade and size (Supplement Tables 4A, 4B, 4D), and their number: 2 to 5 laminations, or 2 or 3 spaced members. If the pieces differ, enter the weakest (15.3.1, 15.2.3.4).
- **Unbraced lengths l1 and l2:**
  - Built-up column: d1 is the face width of a lamination and d2 the thickness of the assembly (n t).
  - Spaced column: d1 is a member's thickness (perpendicular to its wide faces) and d2 its width.
  - **Ke** (Appendix G) gives le = Ke l. For a spaced column, le is not taken less than l (15.2.3.1).
- **During construction:** raises the le/d limit of a built-up column from 50 to 75 (15.3.2.3).
- **Nailing or bolting:** the nail (common wire nails) or the bolt diameter, the rows, the spacing in a row and between rows, the end distance and the edge distance. They are checked against the rules of 15.3.3.1 or 15.3.4.1.
- **Spaced column end blocks:**
  - **Connector position:** the centroid of the end-block connectors from the column end. Within l1/20 is condition a (Kx = 2.5); within l1/10 is condition b (Kx = 3.0).
  - **l3:** from the spacer block to the end connectors.
  - **Connector capacity:** the end-block connectors' capacity at each contacting surface, from your timber connector calculation.
  - **Kx in FcE:** see Limitations.
- **Loads and service:** axial dead, live, roof live and snow loads; wet service; temperature. The load duration factors are in the code options.

Auto-design (editor toolbar) varies the lamination or member size and the number of laminations or members. Objective: the least wood. Species, grade, lengths, fasteners, end blocks and loads stay yours.

Checks performed

Check	Basis
Built-up column stability	CP by Eq. 15.3-1 on le1/d1 (Kf = 1.0) and on le2/d2 (Kf = 0.6 nailed, 0.75 bolted); the smaller CP, but not below that of a lamination as a solid column (15.3.2.2)
Built-up slenderness	le/d ≤ 50, or 75 during construction (15.3.2.3)

Check	Basis
Nailing (15.3.3.1)	15D ≤ end distance ≤ 18D; 20D ≤ spacing in a row ≤ 6 tmin; 10D ≤ spacing between rows ≤ 20D; 5D ≤ edge distance ≤ 20D; two or more rows where d > 3 tmin; nails through all laminations and 3/4 of the outermost
Bolting (15.3.4.1)	7D ≤ end distance ≤ 8.4D (softwoods); 4D ≤ spacing in a row ≤ 6 tmin; 1.5D ≤ spacing between rows ≤ 10D; 1.5D ≤ edge distance ≤ 10D; two or more rows where d > 3 tmin; washers and tight nuts (note)
Spaced column stability	CP by Eq. 15.2-1 on le1/d1 with Kx, not more than the member as a solid column on l2/d2 (15.2.3.5)
Spaced column limits	l1/d1 ≤ 80, l2/d2 ≤ 50, l3/d1 ≤ 40 (15.2.3.2); end-block connectors within l1/10 of the end (15.2.2.1)
End spacer block constant	Ks by species group (Table 13A from G): connector capacity at each contacting surface ≥ member area × Ks (15.2.2.5)
Compression	fc = P / A against Fc' = Fc* CP for every combination

The drawings are the column elevation (length, axial load, mark) and the column section (the laminations and fastener rows, or the spaced members).

Limitations

- **Kx in FcE:** the NDS 2024 printing of 15.2.3.3 writes FcE = 0.822 Emin' / (le/d)² and still defines Kx (2.5 for condition a, 3.0 for condition b) without using it. NDS 2018 writes FcE = 0.822 Kx Emin' / (le/d)². The calculation applies Kx by default (the 2018 form). You can clear the box to follow the 2024 text as printed, which is conservative. This is to be confirmed against the code text and AWC errata.
- **Axial load only:** bending of built-up and spaced columns (3.9, 15.2.3.7) is not covered.
- **Sawn lumber only:** one species, grade and size for all pieces.
- **End blocks:** the split rings or shear plates in the end blocks are designed in the timber connector calculation; here their capacity is an input. Spacer block details (15.2.2.2, 15.2.2.4) are notes.
- **Built-up fastener layout:** the fastener rules are checked as entered. The calculation does not lay out the nails or bolts.

Verification examples

Verification status: 14 values compared in 6 examples (2 published, 4 hand calculations from the cited equations): 14 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: not yet recorded for this module.

The code's own Figures 15C and 15D (typical nailing and bolting schedules) pass every fastener rule. The column capacities are hand calculations of Eqs. 15.3-1 and 15.2-1 with the Supplement values (Douglas Fir-Larch No. 2):

- three 2x6 nailed and bolted, in ASD and LRFD;
- two 3x6 spaced, with and without Kx, with the spacer block constant Ks.

No worked example of a built-up or spaced column is on disk. AWC's Structural Wood Design Examples are listed as needed.

CHAPTER 80

Round timber poles and piles

A round timber pole (ASTM D3200) or pile (ASTM D25) standing as a cantilever from the ground, as in a pole building or a sign. It is designed by NDS 2024 Chapter 6 with the tapered and round column rules of 3.7.2 and 3.7.3. Its embedment is by IBC 2024 1807.3. LRFD or ASD.

Inputs of the pole

- **Member and species:**
  - **Construction pole:** Supplement Table 6B (p. 91), eight species.
  - **Pile:** Table 6A, Pacific Coast Douglas Fir, Red Pine and Southern Pine.
  - The values are for normal load duration and wet service, so CM = 1.0 (6.3.3).
- **Conditioning before treatment:** Cct of Table 6.3.5 (air dried 1.0, kiln dried 0.90, Boulton 0.95, steaming 0.80 or 0.74 marine).
- **Diameters at groundline and at the tip:** from the pole class (ASTM D3200 / ANSI O5.1 circumferences, not built in) or measured.
- **Height above grade and Ke:** Appendix G gives 2.1 for a free-standing cantilever and 1.0 when the roof braces the top.
- **Critical section factor Ccs:** permitted by 6.3.9, so it is an option, off by default. Fc is multiplied by 1 + 0.004 Lc, at most 1.10, with Lc the pole's height above grade.
- **Piles in a capped group:** Cls of Table 6.3.11.
- **Loads:** axial dead, live and snow at the top; lateral wind and live loads at a height above grade. They are combined by ASCE 7-22: the strength combinations for the pole, the ASD combinations for the embedment.
- **Embedment:**
  - **Restraint at grade:** nonconstrained (Eq. 18-1) or constrained by a slab or pavement (Eq. 18-2).
  - **Backfill:** concrete, where b is the footing diameter; or compacted soil or sand, where b is the pole diameter.
  - **Soil:** an IBC Table 1806.2 class or your geotechnical values.
  - **Isolated pole:** the doubling of 1806.3.4 for isolated poles that tolerate 1/2 in. of motion at grade (your judgment, off by default).
  - **Embedment depth:** provided, checked against the required depth.

Auto-design (editor toolbar) varies the diameter at groundline (the tip keeps the entered taper) and the embedment depth. Objective: the smallest pole, then the shallowest embedment.

Checks performed

Check	Basis
Column stability	Representative diameter by Eq. 3.7-2 with a = 0.70 (large end fixed, small end free); the equal-area square for le/d (3.7.3); CP with c = 0.85; le/d ≤ 50

Check	Basis
Compression at the tip	fc at the tip ≤ Fc* (3.7.2)
Bending at groundline	fb = M / (π d³/32) against Fb' with CF above 13.5 in. diameter; no CL (6.3.6)
Shear at groundline	Vr' = (3/4) F'v A for a circular section (3.4.2.1)
Combined	Eq. 3.9-3 with the column's FcE
Embedment	IBC Eq. 18-1 (S1 at d/3) or Eq. 18-2 (S3 at d), solved for the depth that returns itself; the depth for the pressure at most 12 ft; the pressure at most 15 times the tabular value
Vertical bearing	P over the footing or pole bottom against Table 1806.2 (the footing's weight not included)

The drawings are the pole elevation (taper, grade, footing, loads, height and embedment) and the section at groundline.

Limitations

- **Factors not in NDS 2024 Chapter 6:** an untreated factor and a form factor for round sections are not in the 2024 edition (Table 6.3.1), so none is applied.
- **Ct:** taken for wet service, our reading for poles and piles.
- **Combined stress:** Eq. 3.9-3 uses the stress on the representative section and the moment at groundline. This is our reading for a tapered cantilever.
- **Not covered:**
  - pole classes by circumference;
  - the concrete footing's design and weight;
  - soil-structure analysis beyond IBC 1807.3;
  - the pole's connections and bracing;
  - decay and marine borers;
  - CD above 1.6 for water-borne treated poles (6.1.4.1).

Verification examples

Verification status: 15 values compared in 9 examples (6 published, 3 hand calculations from the cited equations): 15 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values:

- the Supplement's Table 6B values and NDS Tables 6.3.5 and 6.3.11;
- two embedment examples on disk: PDHonline S235 (nonconstrained, Eq. 18-1, 7.31 ft) and CalcBook's IBC 2021 example (constrained, 6.60 ft).

Hand calculations: the column, bending and Ccs values. No worked NDS example of a round pole is on disk.

CHAPTER 81

Curved and tapered glulam

A simple-span glulam beam of one of two kinds, by NDS 2024 5.3.8 to 5.3.10, 5.2.8, 5.2.9, 5.4.1 and 5.4.2 under uniform loads. LRFD or ASD.

- a curved beam of constant section;
- a double-tapered curved (pitched-tapered) beam, Figure 5C.

Inputs of the curved beam

- **Beam:** curved with constant section (5.4.1) or double-tapered curved (5.4.2).
- **Stress class:** the Supplement Table 5A values in the library (read against the 2024 Supplement, p. 80).
- **Width and depth:** for a pitched-tapered beam, the depth at the ends de and at the peak dc, the roof slope φT and the soffit slope at the ends φB.
- **Lamination thickness t and radius R:** t is the lamination thickness in the curved part, R the radius of its inside face. Curved members usually use 3/4 in. laminations; t/R is limited to 1/100 for Southern Pine and hardwoods and 1/125 for other softwoods (5.3.8).
- **The moment** (curved beam):
  - **Flattens the curve:** radial tension, checked against Frt'. Gravity on a crowned or pitched beam does this.
  - **Sharpens the curve:** radial compression, checked against Frc' = Fc⊥y'.
- **Span:** span of the beam.
- **Loads:** dead, roof live, snow, live and downward wind in plf. Wind matters for radial tension: with wind or earthquake, Douglas Fir-Larch and the western species take (1/3) Fvx Cvr instead of 15 psi (Table 5.2.8).
- **Deflection limits:** L/ for live load and total load (IBC Table 1604.3 roof values prefilled; your choice).
- **Service conditions:** wet service and temperature. Load duration is in the code options.

Auto-design (editor toolbar) varies the width and the depth in lamination steps (the peak depth of a pitched-tapered beam). Objective: the least wood.

Checks performed

Check	Basis
t/R	≤ 1/100 Southern Pine, ≤ 1/125 other softwoods (5.3.8)
Bending	Curved: 6M/(bd²). Pitched: Kφ 6M/(b dc²), Kφ = 1 + 2.7 tan φT (Eq. 5.4-2). Both against Fb' with Cc (Eq. 5.3-3) and CV (CL = 1.0 with the top braced, 5.3.5)
Radial stress	Curved: fr = 3M/(2 Rm b d) (Eq. 5.4-1). Pitched: fr = Krs Crs 6M/(b dc²) (Eq. 5.4-3) with Krs and Crs (uniform load, dc/Rm ≤ 0.3). Against Frt' (Table 5.2.8, with CD, CM and Ct of Fv) and never more than F'v/3, or against Frc' = Fc⊥y' (5.2.9)

Check	Basis
Straight-tapered part	The largest 6M/(bd²) along the taper against Fb' with the lesser of CI (Eq. 5.3-4, taper φT − φB on the compression face) and CV
Shear	(2/3) F'v b d at the support (de for a pitched beam) with Cvr = 0.72, without the reduction for loads near supports
Deflection	Curved: as a straight beam (5.4.1.4). Pitched: Eq. 5.4-4 with dequiv. Dead load × Kcr = 1.5 (3.5.2) in the total
Horizontal movement	Pitched: ΔH = 2hΔc/ℓ (Eq. 5.4-5), a note for the support detail; curved: a note (5.4.1.4)

The drawings are the beam elevation (the arc, or the Figure 5C shape with its depths and slopes) and the section with the laminations.

Limitations

- **Glulam reference values:** Supplement Table 5A (p. 80), read 2026-11-12. Table 5A Expanded combinations are not offered here.
- **Lateral stability:** the compression edge is taken as braced (CL = 1.0).
- **Not covered:**
  - unbraced beams;
  - Tudor and three-hinged arches (5.4.3);
  - radial reinforcement design (only its F'v/3 cap is checked);
  - point loads and unsymmetrical members.
- **Pitched beam geometry:** ℓc is taken as 2R sin φB, the horizontal length of a circular soffit between the tangent points. The bending stress along the straight-tapered part uses the vertical depth.

Verification examples

Verification status: 17 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No worked example of a curved or pitched-tapered beam is on disk. The values are hand calculations of the NDS equations with the library's 24F-1.8E values:

- **Curved beam:** Cc, CV, bending, radial stress against 15 psi × CD, shear and deflection.
- **Pitched-tapered beam:** Kφ, Krs, Crs, the peak bending and radial stresses, dequiv, Δc and ΔH.

The AITC / APA glulam design manuals are listed as needed.

CHAPTER 82

Concrete beam

A nonprestressed reinforced concrete beam, rectangular or T-shaped, designed to ACI 318-19 (Chapters 9, 21, 22, 24 and 25) by strength design with the ASCE 7-22 load combinations. One model covers simple spans, cantilevers, overhangs and continuous beams; the factored moments and shear can also be entered directly.

Inputs of the concrete beam

- **Materials:** f'c (at least 2,500 psi, 19.2.1.1), Grade 60 or 80 bars, normalweight or lightweight concrete (λ by Table 19.2.4.1(b), or from the equilibrium density wc by Table 19.2.4.1(a)), the modulus Ec by 19.2.2.1(b) (57,000√f'c) or 19.2.2.1(a) (wc^1.5 × 33√f'c, always used for lightweight concrete), and the stirrup grade. The stirrup yield strength used for shear and torsion is limited to 60 ksi (Table 20.2.2.4(a)).
- **Section:** web width bw and overall depth h. A T-beam adds the slab thickness hf and the effective flange width: from Table 6.3.2.1 (slab on both sides: overhang the least of 8hf, sw/2 and ℓn/8 each side; slab on one side: 6hf, sw/2 and ℓn/12) with the clear distance sw to the next web and the shortest clear span, or entered. An isolated T-beam uses the entered width and is checked against 6.3.2.2 (hf ≥ bw/2, bf ≤ 4bw). The clear cover to the stirrups (1.5 in. prefilled, Table 20.5.1.3.1) and the maximum aggregate size (for 25.2.1) complete the section.
- **Reinforcement:** bottom bars in one or two layers (the second layer one bar diameter or 1 in. clear above the first, 25.2.2), top bars, stirrups (size, legs, spacing) and the bar coating. The bars are taken as continuous over the beam; the top bars act as compression steel at positive moment and as tension steel at negative moment.
- **Loads and supports:** the beam length, pins or fixed supports anywhere, line and point loads per load type (see Loads and combinations) and the self-weight bw × h × the unit weight entered (150 pcf prefilled; the slab weight of a T-beam belongs in the line loads). The support width sets the face of the support; the shear is taken at d from the face when *Design shear at d* is ticked (9.4.3.2: the reaction compresses the end region and the loads act at or near the top). A point load between the face and d is detected and the face value is used instead. Instead of loads, the factored Mu+, Mu− and Vu at the critical sections can be entered with the clear span and its support condition.
- **Torsion:** the factored torsion Tu at the critical section (0 = none), whether it is equilibrium torsion (22.7.3.1) or compatibility torsion that may be reduced to φTcr (22.7.3.2), the area of longitudinal torsion bars provided in addition to the flexural bars, and the angle θ of the compression diagonals: 45° for nonprestressed members (22.7.6.1.2(a), prefilled) or an angle from analysis (R22.7.6.1.2, 30° to 60°), used in both Eq. 22.7.6.1a and 22.7.6.1b. For a T-beam the slab overhangs of 9.2.4.4(a) (each side the slab exists, the lesser of h − hf and 4hf, and no more than the entered flange) are counted in Acp and pcp unless they lower Acp²/pcp (9.2.4.4(b)); both values are shown.
- **Deflection:** span / n for the immediate live load deflection (Table 24.2.2: 360 for floors, 180 for flat roofs) and for the deflection after attachment of nonstructural elements (480 where they are likely to be damaged, 240 otherwise), the sustained part of the live load and the load duration (ξ of Table 24.2.4.1.3). These are the engineer's choices; 0 turns a limit off.

Auto-design (editor toolbar) varies the bottom and top bars (size and count), stirrup size and spacing; width and depth when unlocked. Objectives: lowest cost index (concrete, formwork, reinforcement); least reinforcement; shallowest, then cost. Kept unless you select them: overall depth h, web width bw. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Positive and negative moment strength	Strain compatibility (22.2, εcu = 0.003, β1 of Table 22.2.2.4.3) with every bar layer; φ from εt (Table 21.2.2)
Tension-controlled section	9.3.3.1: εt ≥ εty + 0.003 (Table 21.2.2)
Minimum flexural reinforcement	9.6.1.2, bw = min(bf, 2bw) for a determinate beam with the flange in tension; waived by 9.6.1.3 when As ≥ 4/3 As,required
Shear strength	Vc of Table 22.5.5.1 (the larger of (a) and (b) with Av ≥ Av,min, (c) with λs otherwise), √f'c ≤ 100 psi unless Av,min is provided (22.5.3), Vs = Av fyt d/s (22.5.8.5.3), φ = 0.75; d is the depth to the top bars where negative moment acts (cantilevers, continuous spans; the smaller of the two depths)
Section size for shear	Eq. 22.5.1.2
Minimum shear reinforcement	9.6.3.1 where Vu > φλ√f'c bw d (φVc for the shallow and slab-integral beams of Table 9.6.3.1); Av,min/s of Table 9.6.3.4
Stirrup spacing	Table 9.7.6.2.2 along the beam and across the width
Torsion (when Tu ≥ φTth)	Acp, pcp with the slab overhangs of 9.2.4.4; threshold Table 22.7.4.1(a); compatibility reduction to φTcr (22.7.3.2); closed stirrups; section limit Eq. 22.7.7.1a; Av/s + 2At/s with At/s by Eq. 22.7.6.1a (θ = 45° or entered, Ao = 0.85Aoh); with more than two legs, the outer closed leg for At/s plus its share of Av/s; Al by Eq. 22.7.6.1b (cot²θ) and 9.6.4.3; (Av + 2At)/s by 9.6.4.2; spacing 9.7.6.3.3
Bar spacing	Clear spacing 25.2.1; crack control Table 24.3.2 with fs = 2/3 fy (24.3.2.1)
Minimum depth (advisory)	Table 9.3.1.1 with 9.3.1.1.1 (fy) and 9.3.1.1.2 (lightweight)
Immediate live load deflection	Ie of Table 24.2.3.5 at midspan (at the support for a cantilever, 24.2.3.7) under D and under D + each live load type
Long-term deflection after attachment	λΔ = ξ/(1 + 50ρ') (Eq. 24.2.4.1.1) times the sustained deflection, plus the immediate live load deflection
Development lengths (advisory)	ℓd by Eq. 25.4.2.4a (Ktr = 0) and standard hooks by 25.4.3.1
Skin reinforcement (advisory)	9.7.2.3 for h > 36 in.

The diagrams show the loading, the section, the shear and moment of the combination that governs the positive moment, and the immediate deflection under the dead and live loads (with Ie of the critical span).

Limitations

- Nonprestressed beams only; deep beams (clear span ℓn ≤ 4h between supports, 9.9.1.1) are reported as not feasible (design them by Chapter 23).
- The bars are continuous over the beam: cutoffs, bar development at the cutoffs (9.7.3) and the anchorage at the supports are left to the engineer (the development lengths are listed as an advisory).
- One effective depth d (to the bottom bars) is used for shear; ρw uses the smaller of the top and bottom steel where the beam has negative moment.
- Torsion: solid section with closed stirrups around the web (Aoh and ph from one clear cover all round); the entered Tu is applied at the critical section for shear (no torsion diagram from distributed torques); all stirrup legs resist shear and the outer closed leg torsion. Al is entered in addition to the flexural bars; the reduction of 22.7.6.2 in the flexural compression zone is not taken. Hollow sections (22.7.4.1(b), Eq. 22.7.7.1b) and prestressed members are not covered.
- Torsion is an option of the concrete beam rather than a type of its own: the stirrups, the effective depth, Vc and the longitudinal bars are shared by flexure, shear and torsion (Av/s + 2At/s, Eq. 22.7.7.1a uses Vc), so the combined design belongs to one calculation.
- Deflection: one Ie per span from the maximum positive service moment of that span (24.2.3.7), no deduction of the deflection that occurs before the nonstructural elements are attached, and the whole live load added after attachment (both conservative). The crack-control check uses fs = 2/3 fy.
- The tension-controlled requirement is applied as printed in 9.3.3.1 of the ACI 318-19 text on hand (εt ≥ εty + 0.003) (to confirm against the code text: against the latest errata).
- Skin reinforcement and flange distribution of negative-moment bars (24.3.4) are advisories, not designed.

Verification examples

Verification status: 174 values compared in 32 examples (26 published, 6 hand calculations from the cited equations): 161 within tolerance, 13 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The beam is verified against the free StructurePoint design examples (spBeam verification, written to ACI 318-14; the flexure, shear, torsion and spacing provisions used are unchanged in ACI 318-19) and against hand calculations in closed form derived step by step in the test file. ACI 318-19 replaced the Branson effective moment of inertia with the expression of Table 24.2.3.5, so the published 318-14 deflections are deviations (ACI318-14-IE: the program gives 5 to 16 % larger, more conservative, deflections; Mcr, Icr and Ig agree).

Example	Source	What it checks	Published	ILX Calcs
SP-SSB	StructurePoint, Simply Supported RC Beam (ACI 318-14), pp. 2–14, 33	12 × 20 in., 3 #9, 25 ft span, #3 @ 8.3 in.	Mu = 201.88 kip-ft, As,req = 2.872 in², As,min = 0.702 in², Vu = 28.52 kips, φVn = 41.79 kips, s ≤ 8.78 in., crack spacing 10.31 in., Mcr = 32.98 kip-ft, Icr = 3,759 in⁴, Ie(D+L) = 3,812 in⁴	201.88, 2.872, 0.702, 28.52, 41.79, 8.78, 10.31, 32.98, 3,759, 3,807
SP-SSB	same	Deflections (318-14 Ie, deviation)	Ie(D) = 4,337 in⁴, ΔLL = 0.634 in., Δcs + ΔLL = 1.465 in.	4,009, 0.601, 1.500
SP-CB	StructurePoint, Cantilever RC Beam (ACI 318-14), pp. 2–14, 35	16 × 24 in., 4 #9 top, 8.33 ft cantilever, tip loads	Mu = 280 kip-ft, As,req = 3.160 in², As,min = 1.143 in², φVn = 69.48 kips, Icr = 8,120 in⁴, Ie(D+L) = 8,409 in⁴	280.0, 3.146, 1.143, 69.48, 8,186, 8,377

Example	Source	What it checks	Published	ILX Calcs
SP-TOR	StructurePoint, Equilibrium Torsion (ACI 318-14; Wight Ex. 7-2), pp. 7–14, 34	14 × 24 in. cantilever, Tu = 28 kip-ft	φTth = 5.87 kip-ft, Aoh = 215.25 in², 325.55 ≤ 474.34 psi, At/s = 0.0204, Av/s = 0.0295, (Av+2At)/s = 0.0704 in²/in., Al = 1.265 in², Al,min = 0.506 in²	equal within 1 %
SP-WMSJ	StructurePoint, Wide-Module Joist System, exterior spandrel beam (ACI 318-14), pp. 64–65	24 × 21 in. spandrel with a 5 in. slab on one side, compatibility torsion	Acp²/pcp = 2,796 in³ with the flange, 2,822 in³ without (used), φTth = 12.48 kip-ft, Tu reduced to φTcr = 49.9 kip-ft	2,796, 2,822, 2,822, 12.48, 49.9
SP-CNB	StructurePoint, Continuous RC Beam (ACI 318-14), pp. 7–15, 35	T-beam bf = 130.5 in., As,req for Mu+ = 272.52 and Mu− = 471.13 kip-ft, φVn	2.78 in², 5.52 in², 71.11 kips	2.776, 5.469, 71.11

Hand calculations:

Example	What it checks	Hand calculation	ILX Calcs
B-1	14 × 24 in., 4 #8, 24 ft simple span, D = 1.0, L = 1.2 kip/ft	Mu = 224.6 kip-ft, φMn = 279.2 kip-ft, c = 4.686 in.	224.6, 279.2, 4.686
B-1	Shear at d from the face, #3 stirrups at 10 in.	Vu = 30.26 kips, Vc = 38.30 kips, Vs = 28.55 kips, φVn = 50.13 kips	30.26, 38.30, 28.55, 50.13
B-1	Deflection with Ie (Table 24.2.3.5)	Mcr = 53.13 kip-ft, Icr = 7,031 in.⁴, Ie = 7,235 in.⁴, ΔL = 0.375 in., long-term 0.884 in.	53.13, 7,031, 7,235, 0.375, 0.884
B-2	T-beam, bf = 24 in., hf = 3 in., 4 #10 (block below the flange)	Mn = 497.1 kip-ft, εt = 0.0099, φMn = 447.4 kip-ft	497.1, 0.0099, 447.4
B-3	Torsion, Tu = 15 kip-ft, Vu = 30 kips, #4 closed stirrups	Tth = 7.83 kip-ft; Eq. 22.7.7.1a 173.2 psi ≤ 474.3 psi; At/s = 0.131 in²/ft; Al = 0.678 in²	7.83, 173.2, 474.3, 0.131, 0.678
B-4	B-3 with θ = 35° from analysis	At/s = 0.0918 in²/ft, Al = (At/s) ph cot²θ = 0.968 in²	0.0918, 0.968
B-5	T-beam, slab 5 in. on both sides (overhangs 19 in.)	Acp²/pcp = 1,820 in³ (web alone 1,486), Tth = 9.59 kip-ft	1,820, 9.59
B-6	B-3 with four stirrup legs	outer leg At/s + (Av/s)/4 = 0.136 in²/ft	0.136

CHAPTER 83

Concrete column

A tied or spiral reinforced concrete column, rectangular or circular, designed to ACI 318-19 (Chapters 6, 10, 21, 22 and 25) for axial force and bending about one or both axes, with the slenderness of braced columns by the moment magnifier method and the ASCE 7-22 load combinations.

Inputs of the concrete column

- **Materials:** f'c, Grade 60 or 80 bars, normalweight or lightweight concrete, the modulus Ec (19.2.2.1) and the tie or spiral grade (spirals may use fyt up to 100 ksi for confinement, Table 20.2.2.4(a)).
- **Section:** rectangular (width b along x, depth h along y) or circular (diameter D), ties or a spiral (spirals in circular sections), clear cover to the ties (1.5 in. prefilled, Table 20.5.1.3.1) and the maximum aggregate size. Ties or a spiral set φ (Table 21.2.2) and Pn,max (Table 22.4.2.1).
- **Reinforcement:** bars around the perimeter (bars per face along b and along h, corners included, or a number of bars on a circle) or bars at entered coordinates from the centroid (any layout), the tie or spiral bar and the tie spacing or spiral pitch (center to center).
- **Loads:** per load type, the axial force P (compression positive) and the end moments about x and about y at the top and bottom, combined by ASCE 7-22 2.3.1; or factored load cases entered directly with βdns. Enter the end moments with the same sign at both ends for single curvature and opposite signs for double curvature. Mx bends the column about the x axis (the depth h resists it). For combinations from load types, βdns is the factored dead axial force over Pu (the live load is taken as not sustained).
- **Slenderness:** braced against sidesway or not, the unsupported length ℓu (0 = slenderness not considered), the effective length factors kx and ky (k = 1.0 is permitted for nonsway members, 6.6.4.4.3), whether a transverse load acts between the supports (Cm = 1.0) and the (EI)eff expression (Eq. 6.6.4.4.4a, b or c).
- **Biaxial bending:** the code leaves the method to the engineer. The Bresler reciprocal load method or the load contour method with the exponent α (1.0 is conservative; 1.15 to 1.55 is typical for square columns).

Auto-design (editor toolbar) varies the bar size and bars per face (or number of bars of a circular column), with the tie size and the largest tie spacing allowed; the section when unlocked. Objectives: least reinforcement; lowest cost index (concrete, formwork, reinforcement). Kept unless you select them: width b (along x), depth h (along y), diameter D. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Axial and flexural strength	P-M interaction by strain compatibility (22.2, εcu = 0.003, Whitney block, bars inside the block displace concrete) about each axis and direction, φ from εt (Table 21.2.2), capped at φPn,max; uniaxial: ratio along the ray of constant Mu/Pu
Biaxial bending	Bresler: 1/φPni = 1/φPnx + 1/φPny − 1/φPo for Pu ≥ 0.1f'c Ag (Mux/φMnx0 + Muy/φMny0 below); load contour: (Mux/φMnx0)^α + (Muy/φMny0)^α ≤ 1

Check	Basis
Maximum axial strength	Table 22.4.2.1: 0.80Po (ties), 0.85Po (spirals), Po by Eq. 22.4.2.2 (fy ≤ 80 ksi)
Slenderness	6.2.5.1 with r of 6.2.5.2 (0.3h, 0.25D); M1/M2 negative in single curvature
Moment magnification (braced)	6.6.4.5: Pc (Eq. 6.6.4.4.2), (EI)eff (Eq. 6.6.4.4.4a to c, Table 6.6.3.1.1(b)), Cm (Eq. 6.6.4.5.3a or b), M2,min (Eq. 6.6.4.5.4, Cm = 1.0 when it governs), δ (Eq. 6.6.4.5.2); δ ≤ 1.4 (6.2.5.3)
Reinforcement ratio	10.6.1.1: 0.01Ag ≤ Ast ≤ 0.08Ag
Number of bars	10.7.3.1: four with ties, six with spirals
Clear spacing of bars	25.2.3: max(1.5 in., 1.5db, 4/3 dagg)
Ties	Size 25.7.2.2, spacing 25.7.2.1 (16db, 48 tie diameters, least dimension; clear ≥ 4/3 dagg), lateral support 25.7.2.3 (advisory)
Spirals	ρs ≥ 0.45(Ag/Ach − 1)f'c/fyt (Eq. 25.7.3.3), clear pitch 1 in. (4/3 dagg) to 3 in. (25.7.3.1), bar ≥ 3/8 in. (25.7.3.2)

The drawings (on screen, in the report and on the DXF / PDF drawing sheet) show the column section with its bars, ties or spiral, clear cover and the x and y axes; the design interaction diagram φPn-φMn about x (and about y, dashed, when it differs) with the factored load points and the governing one called out (not to scale); and the column elevation over ℓu with the vertical bars and the ties at the designed spacing (or the spiral pitch). Crossties, splices, dowels, the first tie location and the floor members are not part of the calculation and are not drawn; the sheet notes say so.

Limitations

- Columns not braced against sidesway are covered only when k ℓu/r ≤ 22 (6.2.5.1(a)); above that the sway magnifier of 6.6.4.6 needs the story data and the input is reported as not feasible. Use a second-order analysis (6.7, 6.8) and enter its factored moments with ℓu = 0 instead.
- Biaxial bending uses Bresler's or the load contour approximation, not a full biaxial strain-compatibility surface; neither is code text (Bresler 1960; PCA load contour) (to confirm against the code text: against the source papers).
- Shear in the column, the reduced effective area of 10.3.1.2, splices, special moment frame requirements (Chapter
  1. and composite columns are not covered. The lateral support of the bars by crossties is an advisory: the crosstie layout is the engineer's detail.
- The live load is taken as not sustained in βdns for combinations from load types; enter factored cases with βdns to use another value.

Verification examples

Verification status: 52 values compared in 22 examples (18 published, 4 hand calculations from the cited equations): 46 within tolerance, 6 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The column is verified against the free StructurePoint ACI 318-19 design examples and against hand calculations derived step by step in the test file. For each control point of a published interaction diagram the published (φPn, φMn) is entered as a load and the program's capacity on the ray through that point is compared.

Example	Source	What it checks	Published	ILX Calcs
SP-IDT	StructurePoint, Interaction Diagram - Tied RC Column (ACI 318-19; Wight Ex. 11-1), p. 26	16 × 16 in., 4 + 4 #9, f'c = 5 ksi: φPo, φPn,max, fs = 0, fs = 0.5fy, balanced, εt = εty + 0.003, pure bending	997.1, 797.7 kips; (622.3, 169.86), (421.9, 220.05), (270.9, 250.77), (171.6, 286.75), (0, 213.96)	997.1, 797.7; φMn within 0.1 %
SP-IDS	StructurePoint, Interaction Diagram - Circular Spiral Column (ACI 318-19; Pincheira Ex. 10.18.1), p. 31	D = 20 in., 8 #10, #4 spiral	1,426.2, 1,212.3 kips; (984.5, 208.16), (642.2, 281.43), (389.2, 305.80), (26.6, 295.31), (0, 288.09)	1,426.2, 1,212.3; within 0.2 %
SP-SNS	StructurePoint, Slender Columns in Non-Sway Frame (ACI 318-19; Wang Ex. 13.17.3), pp. 1–3	Slenderness limit 34 − 12M1/M2 and k ℓu/r	34.0, 23.45	34.0, 22.56 (deviation ACI318-19-R: r = 0.3h instead of √(Ig/Ag), both permitted)

The sway-frame example (moment magnification by 6.6.4.6) is outside the module's scope.

Hand calculations:

Example	What it checks	Hand calculation	ILX Calcs
C-1	16 × 16 in., 8 #8, tied: Po and φPn,max	φPn,max = 0.65 × 0.80 Po = 638.6 kips	638.6
C-2	Balanced point by strain compatibility (εt = εty)	Pb = 365.9 kips, Mb = 271.6 kip-ft; a load at half φPb, φMb gives a ratio of 0.500	0.500
C-3	Slender braced column, ℓu = 20 ft, single curvature, βdns = 0.6	k ℓu/r = 50.0, Pc = 843.4 kips, δ = 1.902, Mc = 95.1 kip-ft	50.0, 843.4, 1.902, 95.1
C-4	Spiral, D = 18 in., #3 at 2 in.	ρs = 0.0147, ρs,min = 0.0132	0.0147, 0.0132

CHAPTER 84

One-way slab

A solid nonprestressed one-way slab designed on a 12 in. strip to ACI 318-19 Chapter 7, with ASCE 7-22 load combinations (strength design). The moments and shears come from the ACI coefficients of 6.5 or from an elastic analysis of any support layout with the live-load patterns of 6.4.2.

Inputs of the one-way slab

- **Materials:** f'c (at least 2,500 psi, 19.2.1.1), Grade 60 or 80 bars, normalweight or lightweight concrete (λ by Table 19.2.4.1(b), or from the equilibrium density wc by Table 19.2.4.1(a)), and the modulus Ec by 19.2.2.1(b) (57,000√f'c) or 19.2.2.1(a) (wc^1.5 × 33√f'c; always for lightweight concrete).
- **Slab and reinforcement:** thickness h, clear cover (3/4 in. prefilled, Table 20.5.1.3.1 for slabs not exposed to weather), bottom bars and spacing (positive moment), top bars and spacing (negative moment over the supports) and the shrinkage and temperature bars perpendicular to the span. The flexural bars are taken as the outer layer.
- **Spans and analysis:**
  - *ACI coefficients (6.5):* the clear spans ℓn, the support width and how the exterior support is built (integral with a spandrel beam, integral with a column, or unrestrained). The conditions of 6.5.1 are checked: at least two spans, the longer of two adjacent spans at most 20 percent longer than the shorter, L ≤ 3D. Uniform loads and a prismatic slab hold by construction. Where all spans are 10 ft or less, wu ℓn²/12 may be used at the supports (Table 6.5.2 (a)); a check box turns this off.
  - *Elastic analysis:* the slab length and any supports (pins or fixed), with the live load on all spans, on alternate spans and on each pair of adjacent spans (6.4.2) when the pattern box is ticked.
  - The support width sets the span ℓ = ℓn + width used for Table 7.3.1.1, the face of support for the elastic moments (7.4.2.1) and the shear critical section at d from the face (7.4.3.2).
- **Loads:** uniform area loads per load type (D, L, Lr, S, R) and the self-weight from the unit weight entered (150 pcf prefilled). See Loads and combinations.
- **Deflection:** calculated when h is less than Table 7.3.1.1 (7.3.2.1), or always. The limits are span / n for the immediate live load deflection and for the part of the deflection after attachment of nonstructural elements (Table 24.2.2: 360 for floors, 180 for flat roofs; 480 or 240); the sustained load duration (ξ of Table 24.2.4.1.3) and the sustained fraction of the live load are the engineer's.

Auto-design (editor toolbar) varies the thickness (1/2-in. steps) and the bottom, top and shrinkage bars (size and spacing). Objectives: lowest cost index (concrete, reinforcement); thinnest slab, then cost; least reinforcement. Every other input is kept; see Auto-design.

Checks of the one-way slab

Check	Basis
Minimum thickness	Table 7.3.1.1 per span (simply supported ℓ/20, one end continuous ℓ/24, both ends continuous ℓ/28, cantilever ℓ/10), × (0.4 + fy/100,000) for fy ≠ 60 ksi (7.3.1.1.1), × max(1.65 − 0.005wc, 1.09) for lightweight concrete of 90 to 115 pcf (7.3.1.1.2). A thinner slab is not an error: the deflections are calculated instead
Positive and negative moment strength	φMn by strain compatibility (22.2) with φ from εt (Table 21.2.2), at the governing section: Table 6.5.2 coefficients, or the envelope of the combinations and live-load patterns
Tension-controlled section	εt ≥ εty + 0.003 (7.3.3.1 as printed in ACI 318-19)
Minimum flexural steel	As ≥ 0.0018Ag (7.6.1.1)
Bar spacing	≤ min(3h, 18 in.) (7.7.2.3); crack control ≤ min(15(40,000/fs) − 2.5cc, 12(40,000/fs)) with fs = 2/3 fy (24.3.2, 24.3.2.1); clear spacing ≥ max(1 in., db) (25.2.1)
One-way shear	Vu ≤ φVc with Vc = 8λsλ(ρw)^(1/3)√f'c b d (Table 22.5.5.1(c), no shear reinforcement, 7.6.3.1), ρw of the top bars, Vu at d from the face (Table 6.5.4 shears less wu d, or the elastic shear at that section)
Shrinkage and temperature steel	≥ 0.0018Ag (24.4.3.2), spacing ≤ min(5h, 18 in.) (24.4.3.3, 7.7.6.2.1)
Development length	ld by Eq. 25.4.2.4a (Ktr = 0), advisory
Immediate live load deflection	Δ(D + L) − Δ(D) with Ie of Table 24.2.3.5 at each load level (24.2.3.7), against ℓ/n
Long-term deflection	λΔ(ΔD + sustained part of ΔL) + ΔL with λΔ = ξ/(1 + 50ρ') (24.2.4.1), against ℓ/n

The deflection model is the slab line itself: pinned at the support centrelines for the coefficient option, as entered for the elastic option. One Ie is used for the whole member: the value at the maximum positive service moment, or the smaller of that and the support value where the slab cantilevers.

The drawings (on screen, in the report and on the DXF / PDF drawing sheet) show the slab elevation of the strip with its supports and spans (centre to centre, and the clear spans between the support faces for the coefficient method), a typical section across the span with the bottom and top bars at their spacing, the shrinkage and temperature bars, the thickness and the cover, and for the elastic analysis the shear and moment envelopes per strip under the elevation. Bar lengths, cut-offs, laps and the extent of the top bars are not part of the calculation and are not drawn.

Limitations of the one-way slab

- Solid nonprestressed slabs of constant thickness with uniform area loads; no point or line loads, openings, joists or composite deck.
- No shear reinforcement: a slab that needs it (Vu > φVc) fails the shear check; thicken the slab.
- Moments by the coefficients are not redistributed (6.5.3). The elastic option does not redistribute either.
- The wu ℓn²/12 option is applied at the interior supports and at integral exterior supports (to confirm against the code text: whether Table 6.5.2 (a) also replaces the exterior-support values).
- Deflections: no compression steel is counted in ρ' (λΔ is conservative), no deduction for the deflection before the nonstructural elements are attached, and one Ie for the member (24.2.3.7).

- The top bars are assumed to be top bars for development (ψt) only where more than 12 in. of concrete lies below them.

Verification of the one-way slab

Verification status: 28 values compared in 4 examples (2 published, 2 hand calculations from the cited equations): 27 within tolerance, 1 recorded deviations. **Thin: only 2 published examples.** Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The one-way slab is verified against the free StructurePoint one-way slab example (ACI 318-14, Wight Ex. 5-7), the ACI Reinforced Concrete Design Handbook (MNL-17(21), ACI 318-19) One-way Slab Example 1 as tabulated in the free Tekla Tedds verification example, and hand calculations. ACI 318-19 changed Vc of slabs without shear reinforcement (Table 22.5.5.1(c), with λs and ρw); the 318-14 value is a deviation (ACI318-14-VC).

Example	Source	What it checks	Published	ILX Calcs
SP-OWS	StructurePoint, One-Way Slab (ACI 318-14), pp. 4–12, 39	4 spans, h = 7 in., #4 @ 12 in., ACI coefficients	hmin = 7.17 in., Mu+ = 3.14, Mu− = 4.65 kip-ft/ft, As,min = 0.151 in²/ft, s ≤ 18 in., crack spacing 12 in., S&T 0.151 in²/ft, ≤ 18 in.	7.13, 3.14, 4.65, 0.151, 18, 12, 0.151, 18
SP-OWS	same	φVc (318-14, deviation)	6.83 kips/ft	3.84 (Table 22.5.5.1(c))
ACI-HB-OW1	ACI MNL-17(21) One-way Slab Ex. 1 via Tekla Tedds	14 ft span + 6 ft cantilever, h = 7 in., f'c = 5 ksi	hmin 7.0 / 7.2 in., As,min = S&T = 0.15 in²/ft, φVc = 4.9 kips/ft	7.0, 7.2, 0.151, 0.151, 4.93

Hand calculations:

Example	What it checks	Hand calculation	ILX Calcs
OW-1	3 spans of 12 ft, h = 7 in., #4 at 12 in.: ℓ/24, wu ℓn²/14 and /10, φMn, 1.15wu ℓn/2 − wu d, φVc (Table 22.5.5.1(c)), crack-control spacing	6.50 in., 2.97 and 4.16 kip-ft, 5.27 kip-ft, 1.85 kips, 3.84 kips, 12 in.	equal within 0.1 %
OW-2	Simple span 12 ft, h = 5 in.: immediate live load and long-term deflection with Ie of Table 24.2.3.5	5wℓ⁴/(384 Ec Ie)	equal within 0.1 %

CHAPTER 85

Two-way slab

One panel of a flat plate, a flat slab with drop panels or a slab with beams on all sides, by the direct design method, with the minimum thickness, the reinforcement of each column and middle strip and the punching shear at a column (ACI 318-19 Chapter 8 and 22.6, ASCE 7-22 gravity combinations, strength design).

ACI 318-19 permits the direct design method for gravity loads (6.2.4.1) but no longer prints it: the commentary R6.2.4.1 refers to the method of the 2014 Code. The moment coefficients, strip fractions and unbalanced moments used here are therefore those of ACI 318-14 Section 8.10; everything else is ACI 318-19.

Inputs of the two-way slab

- **Materials:** as for the other concrete calculations (f'c, Grade 60 or 80, λ, Ec).
- **Slab system and panel:** the system, the centre-to-centre spans ℓx and ℓy, whether the panel is an interior or an end span in each direction, the adjacent spans and the number of spans (for the limits of 8.10.2 and the unbalanced moment of 8.10.7.2), the largest column offset, and the exterior edge (on columns, an unrestrained bearing wall or a fully restrained concrete wall: the column of Table 8.10.4.2), with or without edge beams.
- **Columns:** rectangular c1 × c2 (c1 along x), or circular, taken as a square of equal area (22.6.4.1.2).
- **Slab and reinforcement:** thickness h, clear cover, the bar size in each direction and which layer is outermost (it sets d in each direction; punching uses the average d, 22.6.2.1). For a flat slab: the drop panel projection below the slab and its plan size (8.2.4: at least h/4 and ℓ/6 each side of the column). For beams (or edge beams): the beam width and overall depth, used for αf (with the slab portion of 8.4.1.8) and βt.
- **Loads:** uniform gravity area loads per load type and the self-weight (the drop panel weight is averaged over the panel). Wind and earthquake are outside the direct design method.
- **Punching shear:** the column checked (interior, edge or corner; the slab edge is across the end-span direction) and the distance from the slab edge to the column face. Vu and the unbalanced moments come from the panel loads (tributary area; 8.10.7.2 at interior columns, 0.3Mo at edge and corner columns, 8.10.7.3) or are entered. The stresses of the two moment directions are added when the box is ticked (always at corner columns).
- **Analysis:** the direct design method (the default) or the **equivalent frame method** (ACI 318-14 8.11, permitted by ACI 318-19 6.2.4.1). For the equivalent frame: the floor-to-floor height of the columns above and below (far ends fixed; clear the column above at the roof), the columns' f'c (0 = the slab's), and whether the frame moments are reduced to Mo by 8.11.6.5 (permitted only within the 8.10.2 limits; off by default).

Equivalent frame method. One frame is analysed in each direction through the panel: the spans in that direction (up to nine; the panel is the end span or the middle span, its neighbours take the adjacent span entered), slab-beams with the moment of inertia at the column face divided by $(1 - c2/\ell2)^2$ from the column centre to the face (8.11.3; drop panels and beams included), columns with infinite stiffness within the joint (8.11.4), and torsional members of the slab (or slab and drop, or edge beam) as wide as the column, $Kt = \Sigma\ 9\ Ecs\ C\ /\ (\ell2\ (1 - c2/\ell2)^3)$ (8.11.5), joined in the equivalent column Kec. The stiffness, carry-over and fixed-end factors that PCA tabulates are computed by numerical integration of the non-prismatic members (the tables are not on file; the results agree with them within 0.5 %). Where the live load exceeds 3/4 of the dead load, 3/4 of the factored live load is placed on the span and alternate spans and on adjacent spans (6.4.3.3), besides the full load on every span. The negative

moments are taken at the column faces, not farther than 0.175 ℓ1 from the centre (8.11.6.1). The panel's moments then go to the column and middle strips (and beams) by the direct design method tables (8.11.6.6), and the unbalanced moment of the frame at the checked column is used for punching shear and moment transfer.

Auto-design (editor toolbar) varies the thickness (1/2-in. steps) and bar size in each direction (the strips' spacings follow). Objectives: thinnest slab (least concrete), then smaller bars. Every other input is kept; see Auto-design.

Checks of the two-way slab

Check	Basis
Limits of the direct design method	ACI 318-14 8.10.2: three spans each way, successive spans within 1/3 of the longer, ℓlong/ℓshort ≤ 2, offsets ≤ 10 %, gravity load, L ≤ 2D, 0.2 ≤ αf1ℓ2²/(αf2ℓ1²) ≤ 5.0 with beams. Outside them the calculation is not feasible
Minimum thickness	Table 8.3.1.1 (interpolated for fy; edge beams count with αf ≥ 0.8; at least 5 in., 4 in. with drop panels) or Table 8.3.1.2 with αfm and β (+10 % at a discontinuous edge without an αf ≥ 0.80 edge beam, 8.3.1.2.1). A thinner slab fails: calculated two-way slab deflections (8.3.2) are not covered
Strip moments	Mo = qu ℓ2 ℓn²/8 (ℓn face to face of columns, ≥ 0.65ℓ1); 0.65 / 0.35 in an interior span, Table 8.10.4.2 in an end span; column-strip fractions of Tables 8.10.5.1, 8.10.5.2 (βt) and 8.10.5.5 with αf1ℓ2/ℓ1; beams take 0.85 of the column strip when αf1ℓ2/ℓ1 ≥ 1 (8.10.5.7); the middle strips the rest
Reinforcement of each strip	The bar spacing is designed: As for Mu (rectangular stress block), at least 0.0018Ag (8.6.1.1), at most min(2h, 18 in.) (8.7.2.2); then φMn by strain compatibility (22.2). A strip that cannot be reinforced at a clear spacing of max(1 in., db) is not feasible
Tension-controlled strips	εt ≥ εty + 0.003 (8.3.3.1 as printed)
Punching shear	vu = Vu/(bo d) + γv Msc c/Jc (8.4.4.2, Jc of the straight-sided critical section at d/2, R8.4.4.2.3) against φvc, vc = least of 4λsλ√f'c, (2 + 4/β)λsλ√f'c, (2 + αs d/bo)λsλ√f'c (Table 22.6.5.2, √f'c ≤ 100 psi); for flat slabs also at d/2 outside the drop panel
Moment transfer by flexure	γf Msc (Eq. 8.4.2.2.2) resisted by top bars within bslab = column width + 1.5h each side (Table 8.4.2.2.3); the number of bars to concentrate there is given
As,min near the column	5vuv bslab bo/(φ αs fy) when vuv > φ2λsλ√f'c (8.6.1.2)

For gravity loads the moment-transfer stress adds at the face of the critical section towards the span; for entered moments the larger distance to a face is used (the moment may reverse).

The drawings (on screen, in the report and on the DXF / PDF drawing sheet) show the panel plan with the column lines, the columns, the exterior slab edges of an end span, the column and middle strips of both directions (tagged CS-X, MS-X, CS-Y, MS-Y), the drop panels or beams and the punching critical section at the checked column; a strip reinforcement schedule with the designed bar size and spacing of every strip at each moment location and the transfer bars of 8.4.2.2; and a typical section at a column through the column strip (slab, drop panel or beam, top and bottom layers with the outer layer as entered, cover). Bar lengths, cut-offs and the column, beam and drop panel reinforcement are not part of the calculation and are not drawn.

Limitations of the two-way slab

- Direct design method or equivalent frame method (no finite elements), gravity loads only, one panel and the column checked; the frame width is taken as the panel's transverse span ℓ2 (equal adjacent transverse spans).
- Equivalent frame: columns at the exterior edge (a wall edge is NOT FEASIBLE in this mode), one column size throughout, identical floors above and below; the drop panel weight is averaged over the panel (not applied as partial loads); slabs with beams use Kt × Isb/Is for beams parallel to the frame (to confirm against the code text: ACI 318-14 R8.11.5); the EFM text is that of ACI 318-14 8.11 (not on disk) as the StructurePoint examples apply it.
- The direct design method text is that of ACI 318-14 8.10 (to confirm against the code text: against the ACI 318-14 text, not on disk), including the uniform distribution of the exterior negative moment at wall supports (8.10.5.4), the 0.3Mo edge moment and the tributary-area column shear.
- The optional 10 % modification of 8.10.4.3 (318-14) and the increased γf of Table 8.4.2.2.4 are not used (conservative).
- Column strip negative bars use the slab d even over a drop panel (conservative for flexure).
- No shear reinforcement (stirrups or headed studs) for punching; no openings near the column.
- Edge beams of the flat-plate systems are counted for Table 8.3.1.1 and βt but are not designed; the beams of a slab with beams are reported (moment share) for design with the Concrete beam calculation. Their stiffness αf uses a slab width of ℓ2 (interior) and ℓ2/2 (edge) (to confirm against the code text: edge-beam slab width).
- Corner reinforcement 8.7.3 and integrity reinforcement 8.7.4.2 are the engineer's detailing.

Verification of the two-way slab

Verification status: 69 values compared in 7 examples (6 published, 1 hand calculations from the cited equations): 66 within tolerance, 3 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

The two-way slab is verified against the free StructurePoint examples (ACI 318-14, from PCA Notes on ACI 318-11 Examples 20.1 and 20.2), the ACI Design Handbook (MNL-17(21)) Two-way Slab Example 1 as tabulated in the free Tekla Tedds verification example, and hand calculations. The direct design method coefficients of the published flat plate match the module's ACI 318-14 8.10 tables exactly.

Example	Source	What it checks	Published	ILX Calcs
SP-TWFP	StructurePoint, Two-Way Flat Plate (ACI 318-14), pp. 5–13	18 × 14 ft end panel, h = 7 in., DDM	hmin = 6.67 in., Mo = 93.82, 0.26/0.52/0.70Mo = 24.39/48.79/65.67 kip-ft, column strip 24.39/29.27/49.25, middle strip 19.52/16.42 kip-ft (per 7 ft), As,min, interior punching Vu = 48.00 kips, bo = 87 in., vc = 252.98 psi, φvc = 189.74 psi	equal within 0.1 %
SP-TWDP	StructurePoint, Two-Way Flat Slab with Drop Panels (ACI 318-14), pp. 12–18	30 × 30 ft, h = 10 in., 10 × 10 ft drops	hmin = 10.30 in., d = 12.75 in., bo = 131 and 514 in.; vc = 282.8 psi at the column and at the drop	10.30, 12.75, 131, 514; 265.2 (λs, deviation ACI318-19-LS) and 188.2 psi (Table 22.6.5.2(c) governs, deviation SP-TWDP-VC)

Example	Source	What it checks	Published	ILX Calcs
SP-TWB	StructurePoint, Two-Way Slab with Beams (ACI 318-14), pp. 5–9	22 × 17.5 ft, beams 14 × 20 in., h = 6 in.	αfm = 3.57, β = 1.28, hmin = 5.69 in.	3.56, 1.28, 5.79 (ℓn face to face of beams, deviation SP-TWB-LN)
ACI-HB-TW1	ACI MNL-17(21) Two-way Slab Ex. 1 via Tekla Tedds	18 × 14 ft interior panel, h = 7 in., 24 in. columns	hmin = 5.8 in., Mo = 127, 0.65Mo = 83 kip-ft, Vu = 70 kips, φVc = 137.4 kips	5.82, 126.8, 82.4, 69.6, 137.8
SP-TWFP-EFM	StructurePoint, Two-Way Flat Plate (ACI 318-14), pp. 18–28	Equivalent frame, three 18 ft spans, 9 ft stories	kNF 4.127, COF 0.507, mNF 0.08429, kc 4.74, C 1,325 in⁴, Kt 367,484 and Kec 554,074 kip-in., DF 0.388 / 0.280; centreline moments 46.65 / 83.91 / 76.21, positive 44.94, face moments 32.42 / 66.99 kip-ft	by numerical integration: 4.130, 0.509, 0.0843, 4.740; the moments within 0.5 %
SP-TWDP-EFM	StructurePoint, Two-Way Flat Slab with Drop Panels (ACI 318-14), pp. 25–28	Equivalent frame stiffnesses with drop panels	kNF 5.587, COF 0.578, mNF 0.0915, kc 5.318 / 4.879, C 10,632 in⁴, Kt 1,352,595, Kec 1,628,679 kip-in., DF 0.551 / 0.355	5.577, 0.578, 0.0916, 5.319 / 4.883

Hand calculations:

Example	What it checks	Hand calculation	ILX Calcs
TW-1	Flat plate 20 × 20 ft end panel, h = 8 in., 16 in. columns: ℓn/30, Mo, column and middle strip moments, interior column punching (vuv + γvMsc c/Jc against 4λsλ√f'c), γf Msc	7.47 in., Mo = 195.1 kip-ft, 10.24 / 5.07 / 4.06 kip-ft per ft, vu = 168.4 psi, φvc = 189.7 psi, 11.7 kip-ft	equal within 0.1 %

The behaviour tests also check the Table 8.10.5.1, 8.10.5.2 and 8.10.5.5 fractions, the commentary Jc of an interior section, and an edge-column section derived by hand.

CHAPTER 86

Concrete shear wall

A rectangular cast-in-place reinforced concrete wall segment to ACI 318-19 Chapter 11 (and Chapter 18 for special structural walls), by strength design with the ASCE 7-22 2.3.1 load combinations. The wall is checked in its plane (shear, combined flexure and axial force) and, optionally, out of its plane per foot of wall (simplified method of 11.5.3 or the alternative slender wall method of 11.8).

Inputs of the concrete shear wall

- **Wall:** length ℓw, thickness h, the height of the entire wall hw (it sets hw/ℓw and αc, 11.5.4.3) and the unsupported height ℓc between out-of-plane supports (11.5.3, 11.8). The self-weight of the wall (unit weight prefilled at 150 pcf) is added to D: the full height hw at the base section in plane, and the half of ℓc above midheight for the out-of-plane strip.
- **Reinforcement:** one curtain (at the centre) or two (one at each face) of vertical and horizontal bars with their spacing, the clear cover to the vertical bars (it sets d for out-of-plane bending) and, optionally, concentrated end bars at each end with the distance of their centroid from the wall end.
- **Materials:** f'c, Grade 60 or 80 reinforcement, normalweight or lightweight concrete (λ by Table 19.2.4.1) and the modulus Ec (19.2.2.1).
- **In-plane loads:** per load type (D, L, Lr, S, R, W, E) the axial force P (compression positive), the in-plane shear V and the in-plane moment M at the section checked (usually the base). W and E act in both directions. See Loads and combinations.
- **Out-of-plane:** not checked, the simplified method (11.5.3: effective length factor k of Table 11.5.3.2 is an input), or the slender wall method (11.8). The loads are per foot of wall: the axial load at the top with its eccentricity from the centreline, and the lateral pressure. The service combination for the 11.8.4 deflection is D + (factor) × one of L, Lr, S, R + (factor) × W or E; the factors are inputs prefilled from commentary R11.8.4.1 (0.5 on live loads, 0.6 on W, which is conservative against the serviceability wind Wa it suggests, and 0.7 on E).
- **Special structural wall:** turns on Chapter 18.10. Inputs: the height above the critical section hwcs, the number of stories ns above it (ωv, Eq. 18.10.3.1.3), the overstrength factor Ωv (Table 18.10.3.1.2: the greater of Mpr/Mu and 1.5, prefilled 1.5; it is taken as 1.0 automatically when hwcs/ℓw ≤ 1.5), the boundary element method (displacement 18.10.6.2 or stress 18.10.6.3) and, for the displacement method, the top design displacement δu.
- **Code options:** the single-story basement / cantilever retaining wall exception to the two-layer rule (11.7.2.3).

Auto-design (editor toolbar) varies the thickness (1-in. steps) and the vertical and horizontal bars (size and spacing). Objectives: lowest cost index (concrete, formwork, reinforcement); thinnest wall, then cost; least reinforcement. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
In-plane shear	Vn = (αc λ √f'c + ρt fyt) Acv ≤ 8√f'c Acv (Eqs. 11.5.4.3, 11.5.4.2), αc for net axial tension by Eq. 11.5.4.4, φ = 0.75; special walls: Ve = Ωv ωv Vu ≤ 3Vu for combinations with E (18.10.3.1), Eq. 18.10.4.1, φ = 0.60 when Vn is less than the shear that develops Mn (21.2.4.1), not for walls designed by 18.10.6.2 (18.10.4.6)
In-plane flexure and axial force	Strain compatibility of the whole segment (22.2, 22.4, 11.5.2.1) with φ from εt (Table 21.2.2) and φPn ≤ 0.65 × 0.80Po; the ratio is measured along the line from the origin through (Mu, Pu)
Minimum reinforcement	Table 11.6.1 when Vu ≤ 0.5φαcλ√f'c Acv, else 11.6.2 (ρt ≥ 0.0025, ρℓ by Eq. 11.6.2 and at least 0.0025, but not more than the ρt required for strength, itself at least 0.0025); shown as areas per foot of wall
Bar spacing	3h and 18 in.; ℓw/3 (vertical) and ℓw/5 (horizontal) where shear reinforcement is required for strength (11.7.2.1, 11.7.3.1)
Two layers	Walls thicker than 10 in. (11.7.2.3)
Ties for vertical bars	Advisory when Ast > 0.01Ag (11.7.4.1)
Out-of-plane, simplified	φPn = 0.65 × 0.55 f'c Ag [1 − (kℓc/32h)²] when the resultant lies within the middle third (Eq. 11.5.3.1; otherwise not feasible), minimum thickness of Table 11.3.1.1
Out-of-plane, slender wall	Mu = Mua / (1 − 5Puℓc²/((0.75)48EcIcr)) with Icr by Eq. 11.8.3.1c (Eq. 11.8.3.1d) against φMn at Pu; conditions of 11.8.1.1 (tension-controlled, φMn ≥ Mcr, Pu ≤ 0.06f'c Ag, Δs ≤ ℓc/150); service deflection by Table 11.8.4.1 with the Ps Δs iteration of Eq. 11.8.4.2
Out-of-plane shear	22.5 (11.5.5.1), Table 22.5.5.1(c) with λs, axial compression neglected
Special wall reinforcement	ρℓ, ρt ≥ 0.0025 (ρt may follow 11.6 when Vu ≤ λ√f'c Acv), spacing ≤ 18 in. (18.10.2.1), two curtains (18.10.2.2), ρℓ ≥ ρt for hw/ℓw ≤ 2.0 (18.10.4.3), end-zone steel ≥ 6√f'c/fy over 0.15ℓw for hw/ℓw ≥ 2.0 (18.10.2.4(a))
Special boundary elements	Need by 18.10.6.2 (c ≥ ℓw/(600(1.5δu/hwcs)), δu/hwcs ≥ 0.005, then b ≥ √(0.025cℓw) or the drift capacity of Eq. 18.10.6.2b) or 18.10.6.3 (σ > 0.2f'c, gross section, combinations with E)

The drawings (on screen, in the report and on the DXF / PDF drawing sheet) show the wall elevation with the vertical and horizontal bars at their designed spacing and the end bars; plan section A at the wall end with the thickness, the curtains, the vertical bars at their designed positions, the horizontal bars, the end bars at their designed centroid and the cover; and the in-plane design interaction diagram φPn-φMn with every factored load point (not to scale). The arrangement and ties of the end bars, boundary elements, splices, openings and the foundation are not part of the calculation and are not drawn.

Limitations

- Solid rectangular cast-in-place segments with deformed bars; no flanges, openings, precast or prestressed walls (the precast rows of Table 11.6.1 are not used), no welded wire reinforcement.
- One section (the base) in plane; in-plane slenderness, coupling beams, wall piers (18.10.8) and the strut-and-tie alternative for squat walls (11.5.4.1) are not covered.
- The horizontal shear reinforcement stress fyt is limited to 60 ksi for ordinary walls (Table 20.2.2.4(a), conservative reading) and to the bar grade for special walls.
- Out-of-plane design is per foot of a uniform strip: concentrated loads spread per 11.8.2.2 are the engineer's input as a load per foot. Only the curtain on the tension face is counted in φMn and Icr (conservative). For the service deflection Mn and Icr are taken at the service axial force.

- Special boundary elements are found where needed but not detailed (18.10.6.4 extent, width and transverse reinforcement; 18.10.6.5 where they are not required): these are advisories for the engineer.
- Ev, Ω0 and the 0.5L option of ASCE 7-22 are not modeled.
- Items to confirm against the code text: the radical in 18.10.6.2(b)(ii) b ≥ √(0.025cℓw) and the order of terms of Eq. 18.10.6.2b (the extracted text lost the layout); the 11.6.2(a) cap on ρℓ ("need not exceed ρt required for strength") is not taken below 0.0025 (R11.6.2); 18.10.2.1 as printed lets only ρt drop to the 11.6 values at low shear.

Verification examples

Verification status: 63 values compared in 17 examples (7 published, 10 hand calculations from the cited equations): 56 within tolerance, 7 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The wall is verified against the free StructurePoint ACI 318-19 wall examples and hand calculations derived step by step in `tests/test_verify_concrete_wall.py`.

Example	Source	What it checks	Published	ILX Calcs
SP-BW	StructurePoint, Bearing Wall (ACI 318-19; PCA Notes Ex. 21.3), pp. 4–11	8 in. precast wall, ℓc = 20 ft, slender wall method	Pu = 4.05 kips/ft, Mua = 32.61, Mu = 45.22, φMn = 67.76, Mcr = 60.72 in.-kips/ft, Icr = 32.3 in⁴, 0.06f'cAg = 23.04 kips, Δs = 0.072 in., Ma = 22.15 in.-kips	equal within 1 %
SP-TU	StructurePoint, Tilt-Up Wall Panel (ACI 318-19; ACI 551.2R Ex. B.1), pp. 5–9, 36	6.25 in. panel, ℓc = 29.5 ft	Pum = 43.49 kips, Mua = 24.77, Mu = 61.00 kip-ft, Icr = 353.56 in⁴ (15 ft panel); Mcr = 46.32 kip-ft, Δcr = 0.550 in., Δs = 0.247 in.	equal within 0.5 % (per foot)
SP-SW	StructurePoint, Shear Wall (ACI 318-19), pp. 4–7, 34	18 ft × 10 in., hw = 54 ft	αc = 2, φVn = 447.92 kips, φMn = 4,808 kip-ft at Nu = 207 kips (approximate method), ρℓ,min = ρt,min = 0.0025	2, 447.92, 4,853 (strain compatibility, +0.9 %), 0.0025

Hand calculations:

Example	What it checks	Hand calculation	ILX Calcs
W-1	20 ft × 10 in. wall, #5 @ 12 in. each face, αc = 3	φVn = 899.5 kips	899.5
W-2	hw/ℓw = 1.75, αc = 2.5	φVn = 842.6 kips	842.6
W-3	Net tension 200 kips, Eq. 11.5.4.4	φVn = 747.7 kips	747.7
W-4	Pure axial strength	Po = 8,862 kips, φPn,max = 4,608 kips	8,862, 4,608
W-5	Simplified method, k = 0.8, ℓc = 12 ft	φPn = 149.4 kips/ft	149.4
W-6	Slender 8 in. wall, ℓc = 20 ft, 25 psf	Mu = 1.412, φMn = 5.453 kip-ft/ft, Icr = 31.40 in.⁴, Δs = 0.032 in., φVc = 3.39 kips/ft	1.412, 5.453, 31.40, 0.032, 3.39
W-7	Special wall, stress method	σ = 0.407 ksi	0.407
W-8	Special wall, Ve = Ωv ωv Vu	315.9 kips	315.9
W-9	Table 11.6.1 minimum vertical steel	0.144 in.²/ft	0.144

Example	What it checks	Hand calculation	ILX Calcs
W-10	Eq. 11.6.2 minimum vertical steel (not below 0.0025)	0.300 in.²/ft	0.300

CHAPTER 87

Punching shear

Two-way shear at a column, concentrated load or reaction area to ACI 318-19 22.6 and 8.4.4: the critical section at d/2, moment transfer by eccentric shear, the concrete strength with its size effect, stirrups or headed shear studs, and the section outside the shear reinforcement. ACI 318 has one method for this check, strength design, so the type has no LRFD / ASD switch. Enter factored values; combine the loads by ASCE 7-22 2.3 first. Use it to check a slab analysed elsewhere, or a footing or mat when you want the shear reinforcement options. The Two-way slab, Spread footing and Pile cap types keep their own punching checks.

Inputs of the punching check

- **Column or loaded area:** rectangular (c1 along x, c2 along y) or circular. For a circle choose the section: the exact circle at d/2 (22.6.4.1, the smallest perimeter, and the default), the square of equal area that 22.6.4.1.2 permits, or the square of equal perimeter of ACI 352.1R. **Location:** interior, edge (free edge on the −x side) or corner (free edges on the −x and −y sides), with the distance from the column face to each edge. The program takes the section that gives the smallest bo (22.6.4.1): it closes at d/2 where it fits, and runs to the edge otherwise. The sides that stay in the slab set αs (R22.6.5.3), so a column far from an edge counts as interior.
- **Slab:** thickness h, effective depth d (the average of the two directions, 22.6.2.1), f'c, λ, fy of the slab bars, and the **size effect factor** λs (Eq. 22.5.5.1.3). λs is on by default, because Table 22.6.5.2 requires it. Turning it off raises a warning: 13.2.6.2 allows that only for some shallow foundations. **Drop panel or shear cap:** its plan size and its projection below the slab. d at the column then includes the projection, and a second section is checked at the edge of the panel (22.6.4.1(b)).
- **Factored loads:** up to 20 cases of reaction and moments in x and y. A positive Mx raises the stress on the +x side, the side away from an edge, and a positive My the stress on the +y side. Mind the signs at edges and corners, where the two sides of the section differ. **Moments entered at:** the centroid of the critical section (R8.4.4.2.3, usual for program output) or the column centre, in which case the eccentricity of the reaction about the centroid is added. **Factored load inside the section wu:** subtracted from the reaction over the plan area inside each section; 0 takes the reaction as Vu, which is conservative.
- **Shear reinforcement:** none, stirrups or headed studs; the stud diameter or stirrup bar; rails or legs on each x face and each y face (running square to the face, evenly spaced between the insets from the corners, none across a free edge); s0, s, the number of peripheral lines, fyt (taken as at most 60,000 psi, 22.6.3.2; studs are commonly 51,000 psi, R20.4); and whether the studs are smooth with a shaft of 10 in. or less, one condition of λs = 1.0 (22.6.6.2(b)).
- **Moment transfer and top bars:** γf by Eq. 8.4.2.2.2, or modified by you with the net tensile strain εt; the program checks Table 8.4.2.2.4 for what you enter. Top bars within bslab, optional: when entered they are checked against Eq. 8.6.1.2.
- **Openings:** centre and size of up to 8. Those within 4h of the column reduce bo (22.6.4.3).

Auto-design varies the slab thickness (d follows, keeping the entered h − d) and, with shear reinforcement, the stud diameter or stirrup bar, the line spacing and the number of lines. The column, loads, f'c, openings and the rails on each face stay as entered.

Checks performed

Check	Basis
Critical section at the column	22.6.4.1, straight sides for a rectangle (22.6.4.1.1), the choices for a circle above, bo a minimum
Shear stress vu	8.4.4.2: Vu/(bo d) + γv Msc c / Jc in each direction, at the worst point of the section; Jc of R8.4.4.2.3; γv = 1 − γf
Concrete alone	vc is the least of (a), (b), (c) of Table 22.6.5.2 with λs, β, αs and √f'c ≤ 100 psi (22.6.3.1); φ = 0.75
With shear reinforcement	vn = vc + vs (22.6.1.3); vc of Table 22.6.6.1; vs = Av fyt/(bo s) (Eqs. 22.6.7.2, 22.6.8.2); λs = 1.0 under 22.6.6.2
Maximum stress	Table 22.6.6.3: φ6√f'c stirrups, φ8√f'c headed studs
Section outside the reinforcement	22.6.4.2: d/2 beyond the outermost line, vc = 2λsλ√f'c (Table 22.6.6.1)
Layout	Tables 8.7.6.3 and 8.7.7.1.2: s0 ≤ d/2; s ≤ d/2 for stirrups; s ≤ 3d/4 or d/2 for studs by vu; 2d along the first line
Stirrup depth	22.6.7.1: d ≥ 6 in. and d ≥ 16 db
Drop panel edge	22.6.4.1(b), with the slab's d
Openings	22.6.4.3: the part between the straight lines from the column centroid tangent to the opening is ineffective
Modified γf	8.4.2.2.4, Table 8.4.2.2.4: the largest γf, vuv and εt limits by location
Minimum top bars	8.6.1.2: over bslab (Table 8.4.2.2.3) where vuv exceeds φ2λsλ√f'c

The drawings show the plan with the column, free edges, every section (dashed), openings with their ineffective part, the shear reinforcement as entered, and a section along a rail. The stud assembly and stirrup shape are not drawn.

Limitations

- Prestressed slabs (22.6.5.5), shearheads, L-shaped and re-entrant columns and beams framing into the column are not covered.
- The section outside the reinforcement is the shortest line at d/2 from the outermost legs and the column, with rounded corners, which gives the smallest perimeter. It is conservative compared with a straight-sided polygon. ACI 421.1R-20 gives the polygon in detail; it is not on disk (see Verification).
- Moments act about the section's own x and y axes, as the published examples do, not about principal axes at a corner.
- At the outer section γv is the value of the section at the column (8.4.2.2.2 defines it there).
- Rails run square to the faces, not radially. For a circular column they start from the column's bounding square. Enter the layout your supplier gives by its rails on each face, and treat the outer section as an estimate if the real layout differs.
- Flexural design, integrity reinforcement (8.7.4.2) and the assembly height (8.7.7.1.1) are yours to check. Seismic drift limits on slab-column connections (18.14.5) are not checked.

Verification examples

Verification status: 72 values compared in 16 examples (16 published, 0 hand calculations from the cited equations): 67 within tolerance, 5 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published examples, written to ACI 318-14 except the last; below d = 10 in. the 318-19 size effect factor is 1.0, so the values carry over.

Example	Source	Values checked
Interior, edge, and edge with γf raised to 1.0	StructurePoint DE-GF	bo 87.0 and 59.5 in., Jc 40,131 and 14,110 in.⁴, Mu 54.65 kip-ft, γv 0.383, vu 166.7, 211.9, 105.3 psi, φvc 189.7 psi
Edge, interior, corner	StructurePoint DE-TWFP	vu 137.20, 110.12, 108.37 psi, bo 37.75 in.
Circular column, three section choices	StructurePoint DE-PSC	bo 139.18, 148.01, 161.7 in., Jc/c 16,240 in.³, load inside 10.55, 9.38, 11.19 kips, vu 234.6, 239.6, 215.4, 193.0 psi, φvc 212.1 psi
With and without an opening	StructurePoint DE-SSO	bo 95.0 and 85.03 in., vu 144.86 and 161.84 psi, φvc 158.0 psi
Headed studs	Dlubal verification example 009972, to ACI 318-19	bo 86.5 in., Jc 28,000 in.⁴, vu 294 psi, vc 190 psi, Av/s 0.40, outer section at 29.8 in., vu/φ 125 psi, limit 126 psi

Three things to read with these. DE-PSC has no λs; the program's φvc at d = 10.3 in. is 0.7 % lower. DE-PSC prints 193.0 psi for a sum that is 192.0, and the program gives 191.9. The Dlubal page was read through a web page summary and does not say how its 10 studs per line sit on the faces; the outer section result rests on an assumed layout of 3 rails on each long face and 2 on each short one.

Stirrups and the drop panel section have no published example here: they are checked by behaviour tests against the code expressions. ACI 421.1R-20, whose stud examples would close that gap and confirm the outer polygon, is not on disk; only its preview is.

CHAPTER 88

Anchors in concrete

Cast-in headed anchors (headed bolts or studs) and hooked bolts, single or in a rectangular group of up to 4 × 4, in tension and shear, to ACI 318-19 Chapter 17 with ASCE 7-22 strength combinations. Chapter 17 is strength design, so the calculation always uses the LRFD combinations.

Inputs of the anchor group

- **Anchors:** headed or hooked, diameter da, ASTM steel (F1554 Gr. 36, 55, 105, A449, A36; futa ≤ 1.9 fya and 125 ksi), whether the steel element is ductile (ACI 2.3; F1554 and A449 rods are), the effective embedment hef, the bearing area Abrg (0 = heavy hex nut) or the hook length eh.
- **Pattern and edges:** anchors along x and y with their spacings, the distance from the outermost anchors to the edge on each of the four sides (0 = no edge on that side) and the member thickness ha.
- **Concrete:** f'c (2,500 to 10,000 psi), λa, cracked or uncracked, supplementary reinforcement for tension and for shear (Table 17.5.3(b): φ = 0.75 with, 0.70 without; the former Conditions A and B), and the edge reinforcement of 17.7.2.5.1 for ψc,V.
- **Loads:** service forces per load type at the group centroid: tension N, moments Mx (tension on the +y anchors) and My (tension on the +x anchors), shears Vx and Vy, and where the shear acts (ex, ey from the centroid).
- **Seismic:** Seismic Design Category C to F; the tension option of 17.10.5.3 (d) Eh × Ω0 or (a) a ductile steel element (with its stretch length), and Ω0 (ASCE 7-22 Table 12.2-1, the engineer's input).
- **Code options:** built-up grout pad (0.80 Vsa), torqued anchors (Table 17.9.2(a) spacing and edge 6 da), the specified cover (20.5.1.3) for the minimum edge distance, and anchors welded to the attachment (shear breakout Case 2 of Fig. R17.7.2.1b).

Auto-design (editor toolbar) varies the anchor diameter and effective embedment hef (the layout is kept); anchor steel when unlocked. Objectives: least anchor steel; lowest cost index (diameter × length). Kept unless you select them: anchor steel. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Anchor forces	Rigid attachment: tension by the elastic distribution about the centroid with compressed anchors at zero (the compression bears at them); shear in equal shares plus the polar share of the torsion of an eccentric shear
Steel in tension	17.6.1: Nsa = Ase,N futa, most highly stressed anchor, φ = 0.75 (ductile) or 0.65
Concrete breakout in tension	17.6.2: ANc/ANco, ψec,N, ψed,N, ψc,N (1.25 uncracked); hef' near three or more edges (17.6.2.1.2); Nb by Eq. 17.6.2.2.1, or 16 λa √f'c hef^5/3 for 11 ≤ hef ≤ 25 in.
Pullout	17.6.3: 8 Abrg f'c (headed) or 0.9 f'c eh da (hooked, 3 da ≤ eh ≤ 4.5 da), ψc,P = 1.4 uncracked, φ = 0.70
Side-face blowout	17.6.4: headed anchors with hef > 2.5 ca1, the anchors along each edge (Nsbg), corner factor

Check	Basis
Steel in shear	17.7.1: 0.6 Ase,V futa, × 0.80 through a grout pad; the row that takes the whole shear in Cases 2 and 3
Concrete breakout in shear	17.7.2 toward and parallel to every edge: Cases 1 and 2 (row spacing s ≥ ca1,1), Case 3 (s < ca1,1) or Case 2 only (welded) of Fig. R17.7.2.1b; ca1' in narrow, thin members (17.7.2.1.2); ψec,V, ψed,V, ψc,V, ψh,V; parallel shear twice the perpendicular value with ψed,V = 1.0
Pryout	17.7.3: kcp Ncpg, kcp = 1.0 (hef < 2.5 in.) or 2.0, φ = 0.70
Interaction	17.8: either ratio ≤ 0.2 ignores it; otherwise the sum ≤ 1.2
Seismic (SDC C to F)	17.10.5 and 17.10.6 when the earthquake share of the tension or the shear exceeds 20 %: 0.75 on the concrete tension strengths (17.10.5.4); option (d) Eh × Ω0 or option (a) steel governs (1.2 Nsa), stretch length ≥ 8 da and futa/fya ≥ 1.3; shear by Eh × Ω0 (17.10.6.3(c))
Detailing	Table 17.9.2(a): spacing 4 da (6 da torqued); edge distance cover + da/2 (6 da torqued); hook length

Drawings

The drawing sheet (screen, PDF and DXF) has the **anchor plan** (the member with the edges you entered, the sides without an edge cut off with break lines, the anchors, their spacing and edge distances, the section cut) and **section A** through an anchor row (the member hatched, the anchors with their headed or hooked end at hef, the embedment and the member thickness). The attachment, anchor length and projection, nuts and washers at the attachment and any anchor reinforcement are not part of the calculation and are not drawn; the sheet notes say so.

Limitations

- Cast-in anchors only: post-installed (expansion, screw, undercut) and adhesive anchors are not covered.
- Anchor reinforcement (17.5.2.1) is not designed; with supplementary reinforcement only φ changes.
- Shear with components perpendicular and parallel to the same edge is combined linearly (the code gives no rule; conservative). Intermediate rows of a 3- or 4-row group are not checked separately for shear breakout.
- The anchor tension ignores a bearing zone beyond the anchors (the lever arm is at most the anchor spacing), which is conservative for an attachment larger than the pattern.
- Seismic options 17.10.5.3(b) and (c) and 17.10.6.3(a) and (b) (attachment-limited forces) are not modelled: enter the attachment-limited force as the earthquake load if you use them. The seismic provisions were written from the code text; no worked seismic example was available to verify them.

Verification examples

Verification status: 110 values compared in 12 examples (12 published, 0 hand calculations from the cited equations): 106 within tolerance, 4 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Guide 1, 3rd Ed. (2024) checks anchor rods to ACI 318-19 Chapter 17:

Example	What it checks	Published	ILX Calcs
DG1 4.7-3	Four ⅞ in. rods, hef = 15 in., uplift 70 kips	φNsa = 20.1, φNpn = 26.7, Nb = 92.3, φNcbg = 76.3 kips	20.1, 26.6, 92.3, 76.6

Example	What it checks	Published	ILX Calcs
DG1 4.7-3	Same rods with a 3½ in. hook	φNpn = 5.80 kips	5.79
DG1 4.7-4	Shear toward an edge, ca1 = 12 in.	AVc = 720 in.², Vb = 23.7, φVcbg = 18.4 kips	720, 23.7, 18.4
DG1 4.7-9	Two 1 in. rods	ANc = 1,510 in.², φNcbg = 51.7, φNpn = 33.6 kips	1,512, 52.0, 33.6
DG1 4.7-15	Four rods in a 20 in. column	hef' = 5.33 in., ANc = 400 in.², Ncbg = 29.2, φNcbg = 20.4 kips	5.33, 400, 29.2, 20.4
DG1 4.7-7	Eight 1½ in. rods, hef = 24 in., grout pad, brace tension and shear	φNsa = 61.4, ANc = 7,850 in.², φNcbg = 239, φNpn = 87.5, φVsa = 25.5, φVcpg = 479 kips, interaction 1.15	61.1, 7,850, 239.5, 87.3, 25.4, 478.9, 1.146

For DG1 4.7-7 the rods sit on a 28 × 6.5 in. outline with unequal x spacings; ILX takes a uniform 4 × 2 grid with the same outline, which gives the same ANc and forces per rod. The engine tests (`tests/test_aci_anchors.py`) add the narrow, thin member of ACI 318-19 Fig. R17.7.2.1.2 (ca1' = 5.33 in., AVc = 168 in.², AVco = 128 in.²). For ψed,V that figure prints 0.98 (0.3 × 5/5.33); ILX applies Eq. 17.7.2.4.1b, 0.7 + 0.3 × 5/(1.5 × 5.33) = 0.89. The load distribution and the seismic rules are covered by hand-calculation tests.

CHAPTER 89

Post-installed anchors

Expansion (torque-controlled and displacement-controlled), screw, undercut and adhesive anchors, single or in a rectangular group of up to 4 × 4, in tension and shear, to ACI 318-19 Chapter 17 with ASCE 7-22 strength combinations. You enter the values of the anchor's evaluation report (an ICC-ES ESR or equal, to ACI 355.2 for mechanical anchors and ACI 355.4 for adhesive anchors). ILX Calcs ships no product data. The prefilled values are samples of the kind in ACI SP-17(14) Tables E.1 and E.2, which ACI marks as fictional: replace them. The calculation says so until you name the report. Chapter 17 is strength design, so the calculation always uses the LRFD combinations. For cast-in anchors use Anchors in concrete.

Inputs of the post-installed anchors

Each hint in the editor says which row of a report the value comes from. You are the engineer of record: a report's value belongs to one anchor, size, embedment and set of conditions, and you choose which conditions apply.

- **Anchor:** the type, the report's name, diameter da, effective embedment hef (for a screw anchor, less than the nominal embedment), ductile or brittle steel element, the steel strengths Nsa and Vsa, and the anchor category (1, 2 or 3).
- **Breakout and pullout values:** kc for cracked concrete (17; product tests may raise it to 24, 17.6.2.2.2) and for uncracked concrete with its ψc,N: 1.0 with the report's kc,uncr, or 1.4 with kc = 17 (17.6.2.5.1). Pullout Np for cracked or uncracked concrete from the report, with the f'c it belongs to and the exponent of the report's adjustment; 0 means the report gives no value and pullout is not evaluated (17.6.3.2.1 does not allow it to be calculated). The load-bearing length le (0 = hef; never more than 8 da, 17.7.2.2.1).
- **Bond values (adhesive):** τcr and τuncr of the report for your temperature range, drilling method and concrete condition, with a factor for the report's modifications (concrete strength, installation); or the minimum values of Table 17.6.5.2.5, indoor or outdoor, under its conditions (17.6.5.2.5). The factors apply to the bond stress in Nba, not to cNa, which uses the characteristic τuncr even in cracked concrete. The anchor category for bond is separate, because reports give it by installation condition (dry, water-saturated, water-filled hole).
- **Pattern and edges:** anchors along x and y, spacings, the distance from the outermost anchors to the edge on each side (0 = no edge on that side), the member thickness ha.
- **Spacing, edge and thickness limits (17.9):** smin, cmin and hmin of the report, and cac; 0 uses the code tables. Where a report gives a cmin and smin curve, enter the pair you use. The specified cover and maximum aggregate size also set the minimum edge distance. With supplementary reinforcement to control splitting the limits of 17.9 do not apply and ψcp,N and ψcp,Na are 1.0.
- **Concrete:** f'c (taken as at most 8,000 psi; above that the anchor needs tests, 17.3.1), λ (λa follows Table 17.2.4.1 for lightweight concrete: 1.0 λ undercut, 0.8 λ expansion, screw and adhesive concrete failure, 0.6 λ bond) or an alternate λa from the report, cracked or uncracked, whether the anchor is qualified for cracked concrete, supplementary reinforcement for tension and for shear (Table 17.5.3(b)), and edge reinforcement for ψc,V.
- **Loads:** service forces per load type at the group centroid (N, Mx, My, Vx, Vy), where the shear acts, and for each row the part that is sustained, 0 to 1. You decide what is sustained; a dead load row marked as not sustained gets an advisory.

- **Seismic:** Seismic Design Category C to F; whether the anchor is qualified for earthquake forces (17.10.3); the report's seismic Vsa,eq, Np,eq and factor on τ, used in combinations with E; the tension option of 17.10.5.3 and Ω0.
- **Installation and code options:** built-up grout pad (0.80 Vsa), concrete at least 21 days old (adhesive, 17.2.2), installation direction, and the temperature range and installation conditions your values assume, which print on the sheet.

Auto-design (editor toolbar) varies the anchor spacing and the embedment of an adhesive anchor within 4 da to 20 da, for the least drilled depth, then the smallest pattern. The diameter and the report's values are kept. The embedment of a mechanical anchor is kept too, because Np and the installation data of a report belong to one embedment: try another line of the report yourself. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Anchor forces	Rigid attachment: tension by the elastic distribution about the centroid with compressed anchors at zero; shear in equal shares plus the polar share of the torsion of an eccentric shear
Steel in tension	17.6.1: Nsa of the report, most highly stressed anchor, φ = 0.75 (ductile) or 0.65 (Table 17.5.3(a))
Concrete breakout in tension	17.6.2: ANc/ANco, ψec,N, ψed,N, ψc,N, ψcp,N (17.6.2.6), kc of the report, hef' near three or more edges; φ = 0.65, 0.55, 0.45 for Categories 1, 2, 3, or 0.75, 0.65, 0.55 with supplementary reinforcement (Table 17.5.3(b))
Pullout	17.6.3: Np of the report at f'c, ψc,P, most highly stressed anchor, φ = 0.65, 0.55, 0.45 (Table 17.5.3(c)); expansion, screw and undercut only
Bond	17.6.5: Nba = λa τ π da hef, cNa, ANa/ANao, ψec,Na, ψed,Na, ψcp,Na; φ by the category for bond (adhesive only)
Sustained tension	17.5.2.2: 0.55 φ Nba against the factored sustained tension on the most loaded anchor (17.5.2.2.1); adhesive only
Steel in shear	17.7.1: Vsa of the report, × 0.80 through a grout pad, φ = 0.65 or 0.60; the row that takes the whole shear in Cases 2 and 3
Concrete breakout in shear	17.7.2 toward and parallel to every edge, Cases of Fig. R17.7.2.1b, le of the report, ψec,V, ψed,V, ψc,V, ψh,V; φ = 0.70, or 0.75 with supplementary reinforcement, for every anchor type
Pryout	17.7.3: kcp Ncb, adhesive the lesser of bond and breakout (17.7.3.1.1, 17.7.3.1.2); φ = 0.70
Interaction	17.8
Embedment limits	17.3.3 adhesive 4 da ≤ hef ≤ 20 da; 17.3.4 screw 5 da ≤ hef ≤ 10 da, hef ≥ 1.5 in.
Spacing, edge distance, thickness	17.9: smin (report or Table 17.9.2(a)); edge distance the greatest of cover, twice the aggregate size, and cmin (report or Table 17.9.2(b)); hmin of the report, or hef limited by 17.9.4 for mechanical anchors
Seismic	17.10.3 qualification and seismic report values; the 20 % rule (17.10.5.1, 17.10.6.1); 0.75 on breakout, pullout and bond (17.10.5.4); 17.10.5.3(d) Eh × Ω0 or (a) ductile steel element with its stretch length and futa/fya; 17.10.6.3(c) for shear
Qualification and installation	Stops the calculation: not qualified for cracked concrete (17.6.2.5.2, 17.6.5.2.3), not qualified for earthquake forces (17.10.3), concrete under 21 days for adhesive (17.2.2). Notes: Table 17.6.5.2.5 conditions, horizontal or upward adhesive anchors under sustained tension (17.2.3, 26.7.2(e)), installation and inspection (17.2.5, 26.7, 26.13)

Limitations of the post-installed anchors

- The values are yours to read correctly: nothing checks them against the report, and no product list ships with the calculation.
- Table 17.9.2(a) has no column for adhesive anchors. The minimum spacing without a report value is 6 da, the value it gives the other post-installed anchors, and the value SP-17(14) uses. Enter smin of the report.
- cNa takes the characteristic τuncr, without the sustained and earthquake notes of Table 17.6.5.2.5 and without your factor on τ. The code does not say this in so many words; it follows R17.6.5.1 and SP-17(14), and is on the safe side near edges and in groups.
- When Seismic is on, the report's seismic values apply in combinations that include E, including where the earthquake share is 20 % or less. 17.10.3 does not tie them to the 20 % rule.
- Not covered: 17.9.3 (a substituted diameter), interpolation of a report's cmin and smin curve, anchor reinforcement (17.5.2.1), shear lugs, fatigue or impact (17.1.4), plastic hinge zones (17.10.2), specialty inserts, through-bolts and grouted anchors (17.1.5), group patterns other than a rectangle, concrete bearing under the attachment (the lever arm is never taken larger than the anchor spacing).
- The sheet draws each anchor as a plain rod to hef; the product's own shape is not part of the calculation.
- For sustained loads longer than the 50 years of ACI 355.4, or at higher temperatures, lower factors should be considered (R17.5.2.2). The 0.55 factor is not changed by the calculation.

Verification examples

Verification status: 289 values compared in 24 examples (24 published, 0 hand calculations from the cited equations): 270 within tolerance, 19 recorded deviations. Code basis: the provisions it uses were read in the target edition.

ACI SP-17(14), Anchorage Examples, to ACI 318-14 (same equations as ACI 318-19, renumbered); the product data there are ACI's fictional samples. Hilti PROFIS Engineering reports, to ACI 318-19. Values agree within 1 % unless noted.

Example	What it checks	Published	ILX
SP-17 Example 3 (pp. 226-230), expansion, tension	φNcb, φNpn	11,582, 9,265 lb	11,581, 9,265 lb
SP-17 Example 4 Part A (pp. 231-237), adhesive, code table stresses	φNa, 0.55 φNba	1,612, 890 lb	1,612, 886 lb
SP-17 Example 4 Part B (pp. 238-239), adhesive, test stress	φNa, 0.55 φNba	2,548, 1,400 lb	2,548, 1,402 lb
SP-17 Example 6 (pp. 246-251), expansion, shear at an edge	φVcb, φVcp	1,495, 9,261 lb	1,495, 9,276 lb
SP-17 Example 10 (pp. 279-287), expansion at a beam end	hef', φNcb, φVcb, φVcp	5.33 in., 4,180, 2,570, 9,012 lb	5.33 in., 4,158, 2,573, 8,956 lb
SP-17 Example 11 (pp. 288-298), adhesive at a beam end	φNa, φVcp	3,898, 8,413 lb	3,893, 8,385 lb
SP-17 Example 13 (pp. 305-310), adhesive group near an edge	φNcbg, φNag	17,454, 12,599 lb	17,420, 12,580 lb
SP-17 Example 17 (pp. 344-352), adhesive group, double eccentricity	ψec,N, ψec,Na, φNcbg, φNag	0.849, 0.824, 19,138, 32,480 lb	0.855, 0.831, 19,272, 32,764 lb

Example	What it checks	Published	ILX
PROFIS Kwik Bolt 1 (pp. 2-6)	φNsa, φVsa, φVcpg, Ncbg with PROFIS's anchor forces	8,148, 3,394, 55,778, 4,986 lb	8,148, 3,394, 55,778, 4,984 lb
PROFIS HIT-HY 200 (pp. 2-5)	φNag, φNcbg	45.29, 25.00 kips	45.22, 25.00 kips

107 published values in all. One documented difference: Examples 10 and 11 add the interaction ratios with capacities rounded to the nearest 100 lb (printed sums 1.05, 1.08; ILX 1.03, 1.06). In Example 17 the eccentricity differs slightly because ILX takes the resultant of the anchor tensions with the compressed anchor at zero, while the example takes the load's position. Example 17 uses τuncr in a member it treats as cracked: that value is entered as τ to follow its arithmetic.

Not published, hand calculations from the cited equations: ψcp,N and ψcp,Na with cac, λa of Table 17.2.4.1, the f'c cap, the Np adjustment, le = 2 da, the sustained part of a group, the notes of Table 17.6.5.2.5, and every seismic path (no worked seismic post-installed example was on disk). PROFIS distributes forces with concrete bearing under a rigid plate and this calculation does not, so PROFIS's demands under moment are not compared. SP-17 Examples 15 and 19 (shear and moment, column anchors) were not run for the same reason.

CHAPTER 90

Rebar development and splices

Development and lap splice lengths of one deformed bar size to ACI 318-19 Chapter 25: straight bars, standard hooks and headed bars in tension, bars in compression, and tension and compression lap splices. Every length is listed in the results with every modification factor shown as a step; the anchorage you pick is checked against the length you provide. Development lengths take no strength reduction factor (25.4.1.3), so the method switch is hidden.

Inputs of the bar and anchorage

- **Bar and concrete:** bar size #3 to #18; Grade 40, 60, 80 or 100 (ψg = 1.0, 1.0, 1.15, 1.3, Table 25.4.2.5); f'c (√f'c is limited to 100 psi, 25.4.1.4); normalweight or lightweight concrete (λ = 0.75 for any lightweight concrete in Chapter 25); uncoated / galvanized or epoxy-coated bars; top bar when more than 12 in. of fresh concrete is placed below it (ψt = 1.3).
- **Anchorage checked and length provided:** straight bar in tension (ℓd), standard hook (ℓdh, measured to the outside of the hook), headed bar (ℓdt, to the bearing face of the head), straight bar in compression (ℓdc), tension lap (ℓst) or compression lap (ℓsc). The length provided is measured from the critical section, which is yours to locate.
- **Cover and spacing:** clear cover (top or bottom; in the plane of a hook), clear side cover (normal to the plane of a hook) and the center-to-center spacing of the bars developed or spliced. cb is the least of the two covers plus db/2 and half the spacing (R25.4.2.4).
- **Straight bars and laps:** the method is your choice (25.4.2.1): the general Eq. 25.4.2.4a or the simplified Table 25.4.2.3, with the other shown for comparison. The table's first row needs clear cover ≥ db and either clear spacing ≥ 2db or clear spacing ≥ db with stirrups or ties throughout ℓd at least the Code minimum (a box you tick). Ktr is 0 (permitted as a simplification, 25.4.2.4) unless you include the transverse reinforcement Atr at spacing s crossing the splitting plane of n bars (Eq. 25.4.2.4b); the same bars show 25.4.2.2 / 25.5.1.5 (Ktr ≥ 0.5db for fy ≥ 80 ksi bars closer than 6 in.). The excess reinforcement reduction of 25.4.10.1 (As,required / As,provided) applies to ℓd and ℓdc only when you state that none of the exclusions of 25.4.10.2 applies; it never applies to hooks, heads or laps. For a tension lap, As,provided / As,required at the splice and the percent of the bars spliced within the lap set Class A or B (Table 25.5.2.1).
- **Hooks and heads:** 90- or 180-degree hook; the number of bars developed at the section (Ahs = n Ab); the ties confining the hooks Ath (25.4.3.3); whether the hook or head ends inside a column core; whether the hook is at a discontinuous end (25.4.3.4); the net bearing area of the head Abrg and whether it meets ASTM A970 Class HA; whether headed bars are in a beam-column joint, with the parallel ties Att (25.4.4.4; elsewhere Att is not counted, 25.4.4.5); compression bars enclosed in a spiral or ties / hoops at 4 in. or less (ψr = 0.75).

Auto-design (editor toolbar) varies the length provided (whole inches) and keeps the bar, the anchorage, cover, spacing and confinement. Objective: shortest length provided. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Straight bar in tension ℓd	25.4.2.1, Table 25.4.2.3 or Eq. 25.4.2.4a with Table 25.4.2.5 (λ, ψg, ψe 1.5 below 3db cover or 6db clear spacing else 1.2, ψs, ψt, ψtψe ≤ 1.7), (cb + Ktr)/db ≤ 2.5, at least 12 in.
Standard hook ℓdh	25.4.3.1 with Table 25.4.3.2 (λ, ψe, ψr 1.0 for No. 11 and smaller with Ath ≥ 0.4Ahs or s ≥ 6db else 1.6, ψo 1.0 inside a column core with 2.5 in. side cover or with 6db side cover else 1.25, ψc), at least 8db and 6 in.; hook geometry Table 25.3.1
Hook at a discontinuous end	25.4.3.4 (advisory: ties at 3db or less when side and top cover are both under 2.5 in.)
Headed bar ℓdt	25.4.4.2 with Table 25.4.4.3 (ψe, ψp, ψo, ψc), at least 8db and 6 in.; conditions 25.4.4.1(a) to (f) each as a check; 25.4.4.6 note in joints
Compression ℓdc	25.4.9.1, 25.4.9.2 (a) and (b) with Table 25.4.9.3 ψr, at least 8 in.
Tension lap ℓst	Table 25.5.2.1 (Class A 1.0ℓd, Class B 1.3ℓd, at least 12 in.), ℓd per 25.4.2.1(a) without the excess reduction (25.5.1.4); 25.5.1.3 noncontact spacing note
Compression lap ℓsc	25.5.5.1 (a), (b) or (c) by fy, × 4/3 below 3,000 psi
Lap of bars larger than No. 11	25.5.1.1, 25.5.5.2: not feasible
Ktr for high-strength bars	25.4.2.2, 25.5.1.5
Length provided	The anchorage checked: provided ≥ required

Drawings

The sheet shows the anchorage checked in elevation from the critical section (straight bar, standard hook drawn to its Table 25.3.1 bend diameter and extension, headed bar, or the two lapped bars), the length provided and the length required, and a section through the bars with the cover and spacing used. The member, the location of the critical section, the head dimensions (drawn schematically from Abrg) and the transverse reinforcement are not part of the calculation.

Limitations

- One bar size per calculation; laps of bars of different sizes (25.5.2.2, 25.5.5.4) and bundled bars (25.6) are not covered.
- Welded wire reinforcement (25.4.6, 25.4.7, 25.5.3, 25.5.4), mechanical anchorage (25.4.5), mechanical and welded splices (25.5.7) and end-bearing splices (25.5.6) are not covered.
- The column lap splice provisions of 10.7.5 and the seismic provisions of Chapter 18 (for example 18.8.5 in special moment frame joints) are not applied.
- Grade 100 bars: where they may be used is limited by Table 20.2.2.4(a); that is the engineer's check.
- The hooked-bar confinement Ath (25.4.3.3) and the headed-bar ties Att (25.4.4.4) are taken as entered.

Verification examples

Verification status: 151 values compared in 88 examples (78 published, 10 hand calculations from the cited equations): 147 within tolerance, 4 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Example	What it checks	Published	ILX Calcs
CRSI ACI 318-19 Tables A1 to A3 (30 bar, cover and position cases, 58 values)	Tension ℓd (Class A) and Class B laps, #3 to #18, f'c = 3,000 / 4,000 / 5,000 psi, covers 0.75 to 3 in., bottom and top bars	e.g. #8, 4,000 psi, 1.5 in.: 36 / 47 in. bottom, 47 / 61 in. top	same (rounded up to the inch as CRSI tabulates)
CRSI ACI 318-19 Table C1 (32 values)	Standard hooks #5, #8, #11, #18 at f'c = 3,000 to 10,000 psi	e.g. #8 at 4,000 psi: 30 in.	same
Simpson Strong-Tie SE Blog example 1	#5 dowel lap, f'c = 3,000 psi, (cb + Ktr)/db capped at 2.5	ℓd = 16.4 in., Class B 21.3 in.	16.4, 21.4
VCmaster template (ACI 318-11 simplified method)	#7 lightweight, epoxy-coated top bar, other cases, ψtψe = 1.7	ℓd = 142 in., Class B 184 in.	141.1, 183.5 (the template takes db = 0.88 in.)

Hand calculations in closed form cover what no free published example shows: headed bars (outside and inside a beam-column joint), compression development with confining ties, compression laps (Grade 60, Grade 80 below 3,000 psi), Ktr from transverse reinforcement, Grade 80 and 100 bars, the excess reinforcement reduction and a confined hook. Published examples for these (CRSI Reinforcing Bars: Anchorages and Splices, ACI MNL-17(21)) are listed in research/DOCUMENTS_NEEDED.md.

CHAPTER 91

Shear friction

Checks shear transfer across a plane by ACI 318-19 22.9: a crack, a construction joint, new concrete against old, or concrete against steel. A second mode checks the contact surface of a composite concrete flexural member by 16.4, a topping on a beam for instance. Strength design only. The result is the steel across the plane, or the ties, against the shear, with the limits and the development.

Inputs of the shear plane

- **Calculation:** a shear plane (22.9) or a composite flexural member (16.4).
- **Contact surface:** the four rows of Table 22.9.4.2. This is your statement of what will be built: the construction documents must say the same (22.9.1.4).
- **Strength reduction factor:** 0.75 for shear, Table 21.2.1. Enter less where 21.2.4 applies to a seismic element; that is your decision.
- **Materials:** f'c, the bar grade and the concrete type. **f'c of the other concrete:** the lesser sets the limits of Table 22.9.4.4 and each side's development uses its own. Grade 80 bars still count as 60 ksi (Table 20.2.2.4(a)); their development uses their true fy.
- **Loads:** by load type (the ASCE 7-22 2.3.1 combinations) or factored values from your analysis. **Amplification** multiplies the shear when a provision says so, for example the factor on a precast diaphragm joint.
- **Plane:** width b and length L along the shear, so Ac = b L. **Angle α:** 90 degrees is perpendicular. Under 90 degrees, Eq. 22.9.4.3 applies, and only if the shear puts the bars in tension: if the shear can reverse, use perpendicular bars.
- **Bars:** size, bars in a row across b, rows along L. Rows are spaced evenly between the covers.
- **Eccentricity e:** the moment Vu e is checked against the bars past L/2.
- **Permanent compression:** a credit by 22.9.4.5, only if it is permanent, and only as you enter it: use the lowest factored value. It offsets net tension first.
- **φ for tension steel:** net tension across the plane adds steel (22.9.4.6). 0.90 is prefilled as tension-controlled (21.2.2). The corbel rule of 16.5 uses 0.75 if you prefer it here.
- **Development on both sides:** straight, standard hook, or "headed, mechanical or welded", which is your own check and is reported as such. Against structural steel, side 1 is the weld.
- **Composite member:** Vu at the section with bv d (16.4.3), or the change of the chord force Vuh over a segment with the contact area (16.4.5). Enter Vuh as the factored shear. Ties: bar, legs, spacing, and the least dimension of the supported element for the spacing limit.

Auto-design varies the bar size, the bars in a row and the rows; for a composite member, the tie bar, legs and spacing. The plane, the surface, the loads and the embedment stay yours.

Checks performed

Check	Basis
Steel across the plane	Eq. 22.9.4.2 Vn = μ Avf fy, or Eq. 22.9.4.3 Vn = Avf fy (μ sin α + cos α); μ by Table 22.9.4.2, λ at most 0.85; φVn ≥ Vu at the combination needing the most steel
Net tension	22.9.4.6: An = Nu / (φ fy), added to Avf
Permanent compression	22.9.4.5: added to the clamping force, as entered
Upper limit	Table 22.9.4.4 on the lesser f'c
Moment across the plane	R22.9.4.6: the flexural steel for Vu e by the stress block of 22.2, plus the share of An, against the bars in the tension half. No steel is added unless it exceeds them
Spacing	25.2.1 for the bars in a row and the rows
Development	22.9.5.1: ℓd by Eq. 25.4.2.4a, or ℓdh by 25.4.3, on each side
Horizontal shear	16.4.3 or 16.4.5: above φ(500 bv d), 22.9 over the contact length with Table 22.9.4.4; else Table 16.4.4.2 (a) to (d)
Ties	16.4.6.1 Av,min; 16.4.7.2 spacing; 16.4.7.3 development is a note

Limitations

- Only the transfer across the plane. Whether the plane is in the right place is your judgment (R22.9.4.2).
- Rows are evenly spaced; with a moment, R22.9.5.1 says to place the steel mainly in the tension zone. If you do, add rows or split the plane into its own calculation.
- The alternative design methods of 22.9.1.2 are not offered.
- Headed studs: their anchorage in the concrete and any shear checks of Chapter 17 are not part of this. Use Anchors in concrete or Embed plate.
- 18.12.9 diaphragm joints use this equation, with their own φ and amplification; enter those.
- The composite mode does not design the member, and ties are not developed by calculation (a note instead). 16.4.5.2 distribution of the ties is a note.

Verification examples

Verification status: 31 values compared in 20 examples (9 published, 11 hand calculations from the cited equations): 31 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published examples for this as a subject of its own are thin. None on disk is written to ACI 318-19. The equations used below read the same in the older editions.

Example	Source	Published
Plank diaphragm joints 1, 4, 5	FEMA P-1051 (2015), Ch. 11, pp. 526, 530, 531 (ACI 318-14)	Avf 2.92, 0.64, 1.41 in²
Topped plank joints 4, 5 and the collector bars	FEMA P-1051, pp. 539, 540	Avf 2.15, 4.03 in²; φVn 120 kips

Example	Source	Published
Column base on a footing	VCmaster US catalog, pp. 65 to 66 (ACI 318-11)	Avf 3.11 in²; φ × limit 86.4 kips
Plate on lightweight concrete	VCmaster, pp. 73 to 74	Avf 0.151 in²
Composite slab on a precast beam	VCmaster, pp. 90 to 91	φ500 bv d 71.3 kips; 1.09 in²/ft

Hand calculations from the equations cover inclined bars, net tension, permanent compression, the grade 80 cap, the lesser f'c, λ from the density, the moment check, Table 16.4.4.2 and 16.4.5. The published set does not exercise the moment check, inclined bars or Table 16.4.4.2 (a): those are hand checked only. ACI MNL-17(21) may hold a 318-19 example; it comes after wave 7.

CHAPTER 92

Embed plate

Checks a steel plate with welded headed studs, flush in a concrete face, taking an attached member's moment, shear and tension. ACI 318-19 Chapter 17 for the studs, strength design. The plate is checked for bending and the concrete for bearing under the compression. The studs act as a welded group.

Inputs of the embed plate

- **Plate:** width B along x, height H along y (the direction of the moment), thickness, Fy.
- **Attached member depth and width:** the depth places the compression force and the tension flange. The width sets the other way's plate bending.
- **Studs:** diameter, effective embedment, head, fya, futa and grid. For a flush plate, hef = stud length + plate thickness − head thickness − the weld burn-off of 3/16 in., as SP-17(14) does; check the product. **No stud at the center** leaves an odd grid's middle clear.
- **Concrete:** f'c, λa, member thickness, cracked, the distance from the outer stud to each edge (0 = no edge), and the cover. **Supplementary reinforcement** changes φ by Table 17.5.3(b); you detail it. **Edge reinforcement** sets ψc,V.
- **Loads:** by load type or factored. M puts the +y side in tension; My the +x side; V acts toward the −y side; N pulls off the concrete. **Shear distance from the concrete face:** V times this adds to the moment; leave it at 0 if your moment already has it.
- **Stud forces:** your choice of model. By default elastic over the group while every stud stays in tension; otherwise the compression sits under the compression flange and tensions grow with the distance from it. That second model is SP-17(14) Example 14's assumption, not a code rule. You can force either.

Auto-design varies the stud diameter, embedment and count, and the plate thickness. The plate's outline, spacing, concrete, loads and member stay yours.

Checks performed

Check	Basis
Stud forces	17.2.1, elastic analysis with the plate stiff; the choice above
Steel in tension, in shear	17.6.1 and 17.7.1 (Eq. 17.7.1.2a, without the 0.6 of a bolt); stud shank area; futa at most 1.9 fya and 125 ksi
Concrete breakout, pullout, side-face blowout	17.6.2, 17.6.3, 17.6.4, on the studs in tension
Breakout in shear, pryout	17.7.2 Case 2 for a welded group (the farthest row), 17.7.3
Interaction	17.8
Plate bending	The moment of the studs past the tension flange, and past the member's side, against φ Fy S (elastic, so the rigid plate assumption holds)

Check	Basis
Stud size	AISC 360-22 I8.1: at most 2.5 t
Bearing	22.8: the length under the flange at φ 0.85 f'c
Spacing, edges	Table 17.9.2(a), 20.5.1.3

Limitations

- Headed studs only. **Deformed bar anchors are not offered:** Chapter 17 has no path for a bar without a head, and treating it as a developed bar is a different design that the PCI Handbook covers.
- The weld of the member to the plate is not designed. Use Weld group.
- Anchor reinforcement (17.5.2.1) and seismic requirements (17.10) are not part of this. Use Anchors in concrete for 17.10 on a bolt group. In SDC C to F, check 17.10 by hand.
- Plate bending takes the full width of the plate and ignores two-way action, and the studs' own local plate effects. Keep the plate stiff.
- The compression position is an assumption. If the plate and member are not as Example 14's, choose by the depth entered or by your own analysis.

Verification examples

Verification status: 73 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 60 within tolerance, 13 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Example	Source	Values
Eight studs, moment and shear	ACI SP-17(14), Anchorage Example 14, pp. 806 to 817 (318-14)	Stud forces 6.79, sum 26.2 kips; plate moment 61 kip-in.; φNsa 14.9; Ncbg 75.5 kips; and with deviations φNpn 27.6, Vcbg 49.3 kips, interaction
Six studs, eccentric tension	SP-17(14), Example 16, pp. 830 to 836	T 10.9; φNsa 21.5; Ncbg 41.6 kips

Three things differ on purpose in Example 14. Its pryout takes the breakout area of the top row alone; this type uses the group's. Its pullout uses 0.75, the factor of 318-14; 318-19 has 0.70. Its Vb takes the load-bearing length as hef, past the limit of 8 da in 17.7.2.2.1, so this type's shear breakout is about 6 % lower and the interaction slightly higher. Example 16 prints one stud at −205 lb; this type sets a stud in compression to zero, so the sum differs slightly. No published example is on disk to ACI 318-19: ACI MNL-17(21) comes after wave 7 and would confirm these. The plate's side bending, the bearing check, biaxial moment in the flange model and I8.1 are hand checked or read from the code.

CHAPTER 93

Concrete diaphragm

Checks one direction of a rectangular concrete floor or roof diaphragm by ACI 318-19 Chapter 12, as a beam spanning between two lines of vertical elements: in-plane shear, the chord, a collector, the transfer into the vertical element, and one opening. Strength design. Enter the factored in-plane force yourself; Diaphragm forces gives Fpx.

Inputs of the diaphragm

- **Span, depth, force:** L between the lines of vertical elements, B along the force, and the factored force on the span. Run one calculation per direction.
- **Reaction factor:** the reaction over w L / 2 where torsion or stiffness gives a line more. Yours; 1.0 for a flexible, symmetric plan.
- **φ:** 0.75 by 12.5.3.2. Enter less where 21.2.4 requires it, for example 0.60 for many shear-controlled diaphragms in SDC D to F.
- **Construction:** cast-in-place; a composite topping on precast (Acv on the combined thickness, the lesser f'c); a noncomposite topping (Acv on the topping). A topping also checks shear friction at the joints.
- **Field reinforcement:** area per foot crossing the shear, bars or welded wire. You can leave it out of Vn, as SP-17(14) does.
- **Chord:** bars at each edge, the lever arm as B less your offset, and the distance from the tension edge to the bars.
- **Collector:** the depth it drags, a force factor (1.0 or your overstrength from ASCE 7-22 12.10.2.1), and its bars. 0 means no collector.
- **Transfer:** the length of the vertical element, the dowels and the surface there.
- **Opening:** its center, size, the depth of the strip on one side, and the share of shear in that strip. By default the share is by depth; SP-17(14) Example 2 uses mass.

Auto-design varies the slab thickness, field reinforcement, chord bars, collector bars and dowels. The plan, forces and surfaces stay yours.

Checks performed

Check	Basis
In-plane shear	Eq. 12.5.3.3: Vn = Acv (2λ√f'c + ρt fy); √f'c at most 100 psi
Section limit	Eq. 12.5.3.4: Vu ≤ φ 8√f'c Acv
Precast joints	12.5.3.5(b), 22.9: μ Avf fy with the topping's steel, fy at most 60 ksi
Chord	R12.5.2.1: T = M / lever arm, As = T / (0.90 fy); 12.5.2.3, within h/4
Collector	R12.5.4.2, 12.5.4.2: T = v × depth dragged × factor, As = T / (0.90 fy); 12.5.4.3, the bars' ℓd along the element
Transfer	12.5.3.7, 22.9: dowels into the vertical element, with Table 22.9.4.4

Check	Basis
Opening	12.2.2: net section shear; chord force plus the strip's local moment
Notes	12.6.3 what may be shared; with the SDC switch, what 18.12 adds

Limitations

- A rectangle, a simple span and a uniform force. A trapezoidal force, an overhang, a stepped plan: use your own analysis and check sections by hand.
- Precast without a topping, mechanical connectors (12.5.3.6), stress fields and distributed chords are not covered.
- **18.12 is not checked.** The SDC switch only says what it adds. Enter φ yourself, and check the thickness and details by hand.
- One opening. The strip's local moment assumes each strip acts fixed at both ends of the opening: a simple assumption, and yours to replace. Trim bars around the opening are not detailed.
- Concrete cover, slab flexure, and combining these bars with those for gravity are yours (12.6.3).
- The collector length here is ℓd as a floor. The transfer length by shear friction (12.5.4.3(b)) is what the transfer check is about.

Verification examples

Verification status: 37 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 37 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Published
East-west	ACI SP-17(14), Rigid Diaphragm Example 1, pp. 298, 305, 306 (318-14)	w 1.94 kip/ft; V 70, Vn 2,590, φVn 1,940 kips; M 1,260 kip-ft; As 0.1 in²
North-south	Same, pp. 298 to 304	V 52.2, φVn 513 kips (φ = 0.60); M 2,595 kip-ft; T 37.6 kips; As 0.70 in²; collector 15.8 kips
Noncomposite topping	FEMA P-1051 (2015), 11.1.2, p. 539 (318-14)	φVn at the joint 264 kips (3.67 kip/ft)

The example's moment takes a trapezoidal force and differs from the uniform one here by under 0.3 %. The rest is hand checked from the equations: opening, section limit, transfer. Example 2 of SP-17(14), with its opening shares by mass, was not run, and no published example is to ACI 318-19. ACI MNL-17(21) comes after wave 7 and would confirm these.

CHAPTER 94

Deep beam (strut-and-tie)

Checks a deep beam by the strut-and-tie method of ACI 318-19 Chapter 23 with 9.9. Strength design, φ = 0.75. Two ready models: one point load, which is a transfer girder under a column and, seen upside down, a two-pile cap; and two symmetric point loads. You choose the model's node depths and the checks tell you if they work.

Inputs of the deep beam

- **Beam:** b, h, the span between support centers, the load's distance, and the length past each support.
- **Bearings:** the length at the supports and under the loads, along the span.
- **Top node depth ws and tie node depth wt:** the model's geometry. The lever arm is h − ws/2 − wt/2. Try wt as the depth the bars take, and ws as what the top strut needs: it is yours to iterate.
- **βc:** Table 23.4.3(b): at a bearing up to the lesser of √(A2/A1) and 2. 1.0 is on the safe side.
- **Depth d:** for Eq. 9.9.2.1 and the grid's spacing. 0 takes h − wt/2.
- **Loads:** unfactored, at each point. **Own weight** is added as dead load at the points if you choose: a lumping that is your decision.
- **Tie:** bars, anchorage, cover. A headed or mechanical anchorage is your own check, reported as such.
- **Distributed steel:** bars on each face, both ways. If it meets Table 23.5.1, interior struts take βs = 0.75; if it falls short, βs = 0.4, unless you say the strut is laterally restrained (23.5.3) or you choose Eq. 23.4.4. 9.9.3.1 requires the grid anyway, so a shortfall still fails.

Auto-design varies the tie bars and the distributed grid. The beam, the bearings, the node depths and the loads stay yours.

Checks performed

Check	Basis
Truss forces	Solved by the library's stiffness method; statics in hand checks
Angle	23.2.7: at least 25 degrees
Section limit	Eq. 9.9.2.1: Vu ≤ φ 10√f'c b d. Eq. 23.4.4 if you use it for βs
Diagonal struts	23.4: fce = 0.85 βc βs f'c, at the narrower end; width lb sin θ + wt cos θ at a support, bearing sin θ + ws cos θ at a load
Top strut (two loads)	23.4: βs = 1.0
Nodes	23.9: βn = 0.80 at the supports (one tie), 1.0 under the load; the bearing, tie and strut faces
Tie	Eq. 23.7.2: Ats ≥ T / (φ fy); 9.9.3.2 minimum
Tie anchorage	23.8.3: straight or hooked, past the extended nodal zone
Distributed steel	9.9.3.1: 0.0025 each way; 9.9.4.3: spacing at most d/5 and 12 in.

Limitations

- Two ready models and no truss of your own. **Not offered:** three-dimensional pile caps, corbels by strut-and-tie, openings, uniform loads as multiple panels, prestress, seismic 23.11.
- Strut widths use the bearing and node depth as drawn; a node with unequal stress faces is not reshaped.
- Unequal two-point loads have no direct truss here. Model them with Truss analysis and check the pieces by hand.
- The tie's centroid is taken at wt/2: fit the bars inside the depth yourself (23.8.1).
- λ enters only in Eq. 23.4.4, as in the code. Chapter 23 has no λ in fce.

Verification examples

Verification status: 46 values compared in 7 examples (4 published, 3 hand calculations from the cited equations): 42 within tolerance, 4 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Two point loads	ACI SP-17(14), Strut-and-Tie Example 1, pp. 660 to 667 (318-14)	Pu 456, S1 and T1 533 kips; Ast 11.8 in²; φ fce 2,550, 1,913, 2,040 psi; φ Fns 673 and 918; φ Fnn 734 kips
One point load	StructurePoint DE-STM, pp. 10 to 16 (318-14)	Pu 638.5, R 319, strut 603, tie 512 kips; angle 32 degrees; φ 10√f'c b d 512 kips; Ast 11.4 in²

Three published values differ on purpose. SP-17(14) prints the limit on Vu with 0.85, an older factor; the code has 0.75. A strut without distributed steel is 0.6 in 318-14 and 0.4 in 318-19. StructurePoint's extended zone follows its own node depth from a stress; here the depth is what you enter. Hand checks: a load off center, βc, Eq. 23.4.4. Both examples are to ACI 318-14. ACI MNL-17(21) Chapter 14 comes after wave 7 and would confirm these to 318-19; SP-17(14) Examples 2 to 5 and the FHWA and TxDOT worked examples are not run.

CHAPTER 95

Corbel

A corbel (bracket) projecting from the face of a column, designed by ACI 318-19 16.5 with shear friction (22.9) and bearing (22.8), under the ASCE 7-22 2.3.1 load combinations (strength design). The section at the face of the column resists the factored shear Vu, the restraint force Nuc and the moment Mu = Vu av + Nuc(h − d) together; the primary tension bars Asc and the closed ties Ah follow from them. Corbels with av/d greater than 1 or Nuc greater than Vu are outside 16.5 (strut-and-tie design, Chapter 23) and are reported as not feasible.

Inputs of the corbel

- **Materials:** f'c, Grade 60 or 80 bars, normalweight or lightweight concrete (λ by Table 19.2.4.1). For lightweight concrete the shear-friction μ uses λ ≤ 0.85 (Table 22.9.4.2) and the Vu/φ limits of 16.5.2.5 apply.
- **Corbel:** width b (usually the column width), depth h at the column face, depth at the outer edge (equal to h for a rectangular corbel), projection from the face, the column depth along the projection (for the development of the bars into the column), clear cover to the primary bars and the maximum aggregate size (clear spacing, 25.2.1). The effective depth d is taken at the face (16.5.2.1): h − cover − db/2.
- **Bearing:** the shear span av from the column face to the center of the bearing, and the bearing plate length (along the projection) and width (across the corbel).
- **Reinforcement:** the primary tension bars (size, number), how they are anchored at the front face (16.5.6.3: a welded transverse anchor bar of at least the same size, or a horizontal loop: the engineer's choice), the closed ties (size, number; two legs each) spread uniformly within 2/3 d below the primary bars (16.5.6.6), how the bars are developed into the column (standard hook with the confinement ψr and location ψo factors of Table 25.4.3.2, or straight with ψt for top bars), the bar coating, and the shear plane at the column face: monolithic (μ = 1.4λ, prefilled), against hardened concrete intentionally roughened (1.0λ), or not roughened (0.6λ).
- **Loads:** per load type, the unfactored vertical reaction V and any horizontal tension N at the bearing. The restraint force Nuc of 16.2.2.3 is the engineer's choice: on bearing pads, 0.2 × the sustained unfactored reaction × 1.6 (16.2.2.3(b); the sustained reaction is D plus the sustained part of L entered); a restraint force treated as a live load (16.2.2.3(a): the entered force times the live-load factor of each combination); or none (the N column only). A friction coefficient from tests caps Nuc at 1.6 μ × the sustained reaction (16.2.2.4). Vu and Nuc of the same combination act together (16.5.3.1, 16.5.3.2).

Auto-design (editor toolbar) varies the depth at the column face (1 in. steps, the outer depth kept in proportion), the primary bars (size, number) and the closed ties (size, number). Objectives: lowest cost index (concrete, reinforcement); shallowest corbel, then cost. The width, projection, bearing and loads stay the engineer's; see Auto-design.

Checks of the corbel

Check	Basis
Applicability	av/d ≤ 1 and Nuc ≤ Vu in every combination (16.5.1.1); otherwise not feasible

Check	Basis
Depth at the outside edge of the bearing	≥ 0.5d, with the depth taken linearly between h and the outer depth (16.5.2.2)
Bearing area position	the outside edge of the bearing (av + plate length / 2) within the inside face of the welded anchor bar, or within the straight part of a looped bar (the loop bend of Table 25.3.2) (16.5.2.3)
Maximum shear	Vu ≤ φ × least of 0.2f'c b d, (480 + 0.08f'c) b d, 1,600 b d for normalweight concrete (16.5.2.4); lesser of (0.2 − 0.07 av/d) f'c b d and (800 − 280 av/d) b d for lightweight (16.5.2.5); Table 22.9.4.4 (d), (e) for concrete against hardened concrete not roughened; φ = 0.75 (Table 21.2.1(f))
Primary tension reinforcement	Avf = Vu / (φ μ fy) (Eq. 22.9.4.2), An = Nuc / (φ fy) (Eq. 16.5.4.3), Af from φ Af fy (d − a/2) = Mu by the stress block of 22.2 (16.5.4.5); Asc ≥ greatest of Af + An, (2/3)Avf + An, 0.04(f'c/fy) b d (16.5.5.1); the combination with the largest Asc governs
Closed ties	Ah ≥ 0.5(Asc − An) with Asc the primary reinforcement provided (Eq. 16.5.5.2); their clear spacing within 2/3 d at least the 25.2.1 minimum (16.5.6.6)
Bar spacing	clear spacing of the primary bars across b ≥ max(1 in., db, 4/3 dagg) (25.2.1, 16.5.6.2)
Bearing	Vu ≤ φ 0.85 f'c A1 with φ = 0.65 (Table 22.8.3.2(c): the corbel is not wider than the plate on all sides)
Development into the column	ℓdh (25.4.3.1) or ℓd (Eq. 25.4.2.4a, Ktr = 0) ≤ column depth − cover (16.5.6.4)
Front anchorage	note: the welded anchor bar or the horizontal loop chosen (16.5.6.3), development for a stress near fy up to the load (16.5.6.5)

Limitations

- One bearing at a time, at mid-width; Vu and Nuc act at the bearing. Corbels with av/d > 1, Nuc > Vu or compression across the shear plane are not covered (strut-and-tie design by Chapter 23 is not offered; 22.9.4.5 is not taken).
- Nuc follows 16.2.2.3 as the engineer chooses; the 0.2Vu minimum of ACI 318-14 and earlier is not part of ACI 318-19 and is not applied (enter it as the N of a load row if wanted).
- Ah uses the primary reinforcement provided (16.5.5.2 as written); some worksheets use the required area (deviation CORBEL-AH-ASC).
- The weld of the anchor bar, the bearing plate thickness and its anchorage, the column design and the supported member are not part of the calculation; the bearing check takes no √(A2/A1) increase.
- Verified against two free vendor worksheets and hand calculations: no free corbel example to ACI 318-19 from ACI, PCA or PCI is on disk (research/DOCUMENTS_NEEDED.md).

Verification examples

Verification status: 28 values compared in 5 examples (2 published, 3 hand calculations from the cited equations): 24 within tolerance, 4 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Published (free worksheets; research/sources/design_examples). VCmaster is written to ACI 318-11, whose corbel provisions (11.8) are those of ACI 318-19 16.5; the Maple Flow worksheet is labelled ACI 318-19 but keeps the older 800 psi shear limit.

Example	Source	What it checks	Published	ILX Calcs
VC-CORBEL	VCmaster US catalog, Corbel design (ACI 318-11), pp. 4–6	14 × 12 in., av = 3 in., Vu = 88.8, Nuc = 32 kips, f'c = 5,000 psi	φVn,max = 101.6 kips, Avf = 1.41, An = 0.71 in², Mu = 298.4 kip-in., Asc,min = 0.51, Asc = 1.65, Ah = 0.65 in²	101.6, 1.41, 0.711, 298.4, 0.513, 1.651, 0.644
VC-CORBEL	same	Af with a 0.9d lever arm (deviation CORBEL-AF-LEVER)	0.67 in²	0.62 (stress block)
MP-CORBEL	Maplesoft, Maple Flow Corbel Design, pp. 1–4	15 × 22 in., av = 4 in., Vu = 80, Nuc = 40 kips	An = 0.889, Avf = 1.270, Mu = 33.33 kip-ft, Asc,min = 1.000, Asc = 1.735 in²	0.889, 1.270, 33.33, 1.000, 1.735
MP-CORBEL	same	deviations CORBEL-VN-800, CORBEL-AF-PHI, CORBEL-AH-ASC	φVn,max = 180 kips (800 psi), Af = 0.373 (φ = 0.90), Ah = 0.423 in²	198 (318-19), 0.449 (φ = 0.75), 0.456 (Asc provided)

Hand calculations (closed form in `backend/tests/test_verify_corbel.py`):

Example	What it checks	Hand calculation	ILX Calcs
CB-1	Default corbel on bearing pads, D = 30, L = 20 kips	Nuc = 9.60 kips, Avf = 1.079, Af = 0.599, Asc = 0.933, Ah = 0.553 in², φBn = 106.1 kips, ℓdh = 19.42 in.	equal
CB-2	Sand-lightweight concrete (16.5.2.5, μ = 1.4 × 0.85)	φVn,max = 134.6 kips, Avf = 1.270 in²	equal
CB-3	Against hardened concrete not roughened (μ = 0.6, Table 22.9.4.4 (d), (e))	φVn,max = 154.2 kips, Avf = 2.519 in²	equal

CHAPTER 96

Coupling beam

A reinforced concrete coupling beam between two special structural walls, ACI 318-19 18.10.7: which reinforcement the code allows for its aspect ratio and shear, and the design of either two intersecting groups of diagonal bars (18.10.7.4) or longitudinal bars and hoops as a special moment frame beam (18.6.3 to 18.6.5, the wall boundary taken as a column). The demands come from the analysis of the coupled walls with the earthquake effects; this type designs the beam, not the walls.

Inputs of the coupling beam

- **Materials:** f'c, reinforcement grade, concrete type (λ).
- **Beam:** width bw (usually the wall thickness), depth h, clear span ℓn between the wall faces, clear cover to the hoops.
- **Demands (factored, from the analysis with E):** shear Vu; for longitudinal bars also the moment Mu at the face, the factored gravity shear at the face (added to the capacity-design shear Ve) and the axial compression (Vc is taken as zero only when it is below Ag f'c/20, 18.6.5.2(b)).
- **Reinforcement:** diagonal or conventional. 18.10.7.1 requires conventional reinforcement when ℓn/h ≥ 4; 18.10.7.2 requires diagonals when ℓn/h < 2 and Vu ≥ 4λ√f'c Acw, unless the engineer shows that the loss of the beam does not impair vertical load capacity, egress or nonstructural components (a check box: the engineer's determination); between, either is allowed. For a slender beam the engineer may waive 18.6.2.1(b), (c) by showing lateral stability by analysis.
- **Diagonal bars:** bar size, bars per group, layers per group, the distance from the beam face to the centroid of each group at the wall face (it sets α: tan α = (h − 2 × distance)/ℓn), and the confinement: each group in its own cage (18.10.7.4(c), cage width and depth out-to-out) or the full section (18.10.7.4(d)).
- **Longitudinal bars:** top and bottom bars (size, number).
- **Hoops:** bar, spacing (along the diagonals for group confinement), legs across bw (vertical legs: also the shear legs of a conventional beam) and legs over the depth; the embedment of the bars into each wall.

Auto-design chooses the lightest diagonal bars (size and number per group) or top and bottom bars, then the hoops (size, spacing, legs); the beam size, span, demands and the reinforcement type stay the engineer's. See Auto-design.

Checks performed

Check	Basis
Reinforcement type allowed	18.10.7.1 to 18.10.7.3
Diagonal: φVn = 0.85 × 2 Avd fy sin α ≤ 0.85 × 10√f'c Acw	Eq. 18.10.7.4, 21.2.4.4
Diagonal: at least four bars per group in two or more layers	18.10.7.4(b)

Check	Basis
Group confinement: cage ≥ bw/2 by bw/5; Ash ≥ max(0.09 s bc f'c/fyt, 0.3 s bc (Ag/Ach − 1) f'c/fyt) each way with the cover all round; spacing ≤ 6db and so = 4 + (14 − hx)/3 (4 to 6 in.); legs ≤ 14 in.; perimeter bars 0.002 bw s at 12 in. (note)	18.10.7.4(c), 18.7.5.3(d)
Full-section confinement: the same Ash on the beam core; s ≤ 6 in. and 6db; legs ≤ 8 in.	18.10.7.4(d)
Conventional: As,min = max(3√f'c, 200) bw d / fy and ρ ≤ 0.025 (0.02 for Grade 80) at each face; two continuous bars; Mn+ ≥ Mn−/2	9.6.1.2, 18.6.3.1, 18.6.3.2
Conventional: φMn ≥ Mu (strain compatibility, φ from εt)	22.2, 21.2.2
Conventional: Ve = (Mpr+ + Mpr−)/ℓn + Vg with Mpr at 1.25 fy; Vc = 0 when the earthquake shear is at least Ve/2 and Pu < Ag f'c/20; Vs = Av fyt d/s ≤ 8√f'c bw d; φ = 0.75, or 0.60 when Vn is below the shear at the nominal moments	18.6.5, 22.5, 21.2.4.1
Conventional: hoop spacing ≤ d/4, 6 in., 6db (5db Grade 80); for ℓn/h ≥ 4 also ℓn ≥ 4d and bw ≥ min(0.3h, 10 in.)	18.6.4.4, 18.6.2.1
Development into each wall for 1.25 fy: embedment ≥ 1.25 ℓd (Eq. 25.4.2.4a, Ktr = 0)	18.10.2.5 (to confirm against the code text), 25.4.2.4

The drawings show the beam elevation between the wall piers (the crossing diagonal groups, or the bars and hoops) and the section at the wall face.

Limitations

- Rectangular beams; slabs or flanges acting with the beam, steel and composite coupling beams are not covered.
- The walls, their boundary elements and the 18.10.9 ductile coupled wall checks are not part of this type.
- The shear and moment come from the engineer's analysis; the beam's stiffness (effective section properties) is not computed.
- Development uses Eq. 25.4.2.4a with Ktr = 0 (conservative inside a confined boundary element); the 1.25 fy requirement is our reading of 18.10.2.5 (to confirm against the code text).

Verification examples

Verification status: 13 values compared in 3 examples (1 published, 2 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_coupling_beam.py):

Source	What it checks	Published	ILX Calcs
FEMA P-2192-V1 (2020) 4.5.3: ℓn = 76 in., h = 28 in., bw = 26 in., f'c = 8,000 psi, 6 #8 top and bottom, six #4 legs at 6 in.	As,min, As,max	2.97, 16.58 in.²	hand-checked equal
same	φMn (spColumn), Mpr	526, 724 kip-ft	strain compatibility, within 1 % by hand
same	Ve, Vs, 8√f'c bw d, hoop spacing limit	229, 305, 474 kips, 6 in.	hand-checked equal

Hand calculations cover the diagonal option (α from the geometry, 2 Avd fy sin α, the 10√f'c Acw cap) and the Ash of the full-section confinement. No published diagonally reinforced coupling beam example to ACI 318-19 is on disk.

CHAPTER 97

Dapped-end beam

The reduced-depth end (nib) of a precast beam or double-tee stem that bears on a ledge or corbel, designed for the five failure modes of the PCI Design Handbook (8th Ed., 5.6) with the ACI 318-19 provisions they rest on (shear friction 22.9, shear 22.5, bearing 22.8, development 25.4) and the ASCE 7-22 2.3.1 combinations. **The PCI Design Handbook is not on disk:** its equations are built from knowledge and marked "to confirm against the code text" on screen and in the report until the handbook is read.

Inputs of the dapped end

- **Materials:** f'c, reinforcement grade, concrete type (λ).
- **Beam and nib:** web width b (one stem of a double tee), full depth H, nib depth hn, nib projection ℓp from the dap face, clear cover.
- **Bearing:** distance av from the dap face to the bearing center, pad length and width.
- **Reinforcement:** the nib top bars As, the horizontal closed ties Ah (within 2/3 d below As), the hanger stirrups Ash next to the dap face and the distance from the dap face to their centroid (it sets the shear span a = av + that distance), the nib stirrups Av, the beam stirrups beyond the dap and the full-depth d.
- **Shear-friction coefficient:** the code's μ = 1.4λ for monolithic concrete (default) or the handbook's effective μe = 1000 λ b hn μ / Vu ≤ 3.4 (the engineer's choice).
- **Loads:** per load type, unfactored, the reaction V and the horizontal tension N at the bearing; Nu is taken as at least 0.2 Vu unless the engineer clears that box (provision made for the restraint force otherwise).

Auto-design chooses the lightest As (size, number), then the numbers of hanger stirrups, ties and nib stirrups; the section, nib depth, bearing and loads stay the engineer's. See Auto-design.

Checks performed

Check	Basis
(1) Flexure and axial tension of the nib: As ≥ [Vu (a/d) + Nu (hn/d)]/(φ fy)	PCI 5.6 (to confirm), φ = 0.75
(2) Direct shear at the dap face: As ≥ 2/3 Avf + An, Avf = Vu/(φ fy μ), An = Nu/(φ fy); Vu ≤ φ Vn,max of Table 22.9.4.4	PCI 5.6, ACI 318-19 22.9
Horizontal ties Ah ≥ 0.5 (As − An)	PCI 5.6 (to confirm)
(3) Hanger reinforcement at the re-entrant corner: Ash ≥ Vu/(φ fy)	PCI 5.6 (to confirm)
(4) Diagonal tension in the nib: Vu ≤ φ(Av fy + Ah fy + 2λ√f'c b d), Av ≥ (Vu/φ − 2λ√f'c b d)/(2 fy)	PCI 5.6 (to confirm)
(5) Shear of the full-depth section: Vu ≤ φ(2λ√f'c b d + Av fy d/s), stirrup spacing ≤ d/2 and 24 in.	ACI 318-19 22.5, 9.7.6.2.2
Bearing on the nib: Vu ≤ 0.65 × 0.85 f'c A1	ACI 318-19 22.8

Check	Basis
Clear spacing of As	ACI 318-19 25.2.1
Nib depth about H/2 or more; a/d ≤ 1 (advisories)	PCI 5.6 (to confirm)
Development lengths of As, Ah, Ash (reported)	ACI 318-19 25.4.2.4, Ktr = 0

The drawings show the end elevation (the nib, bearing pad and reaction, the bar groups) and the section through the nib.

Limitations

- **The strut-and-tie alternative (ACI 318-19 Chapter 23) is not built in this release**; the checks follow the handbook's failure modes only.
- The PCI equations, the 0.2 Vu minimum, the μe limit of 3.4 and the H/2 and a/d ≤ 1 limits are from knowledge of the handbook (to confirm against the code text).
- Rectangular webs; flanges, prestress within the nib, inclined hanger bars and the bearing pad design are not covered. The anchorage details (welded cross bar or plate at the end of As, hooks of the hangers) are the engineer's; the program reports the development lengths.

Verification examples

Verification status: 11 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Every value is a closed-form hand calculation of the equations above (backend/tests/test_verify_dapped_end.py): the five failure modes of the default beam, bearing, and the PCI effective shear-friction coefficient. No published dapped-end example is on disk (the PCI Design Handbook and the PCI connections manual are listed in research/DOCUMENTS_NEEDED.md).

CHAPTER 98

Bearing on concrete

The bearing strength of concrete under a plate or a bearing member by ACI 318-19 22.8: the loaded area A1, the area A2 of the 1:2 frustum inside the support, the √(A2/A1) ≤ 2 increase of Table 22.8.3.2, φ = 0.65, with the ASCE 7-22 combinations of the bearing loads.

Inputs of the bearing

- **Support:** f'c of the supporting concrete (grout stronger than it is ignored), its plan length and width and its depth below the bearing surface. **Area A2:** from the geometry (the frustum with sides of 1 vertical to 2 horizontal, the same spread on every side: the least edge distance or twice the depth), entered by you for a sloped or stepped support (Fig. R22.8.3.2), or no increase (Table 22.8.3.2(c)).
- **Loaded area:** the plate (or the bearing member's section) B × N and its offset from the support centre. A1 is not greater than the plate (22.8.3.2).
- **Bearing force:** loads per type (the ASCE 7-22 2.3.1 combinations) or a factored force; enter the component normal to the bearing surface.
- **Conditions:** a post-tensioned anchorage zone (22.8 does not apply: 25.9), tension in the plane of bearing (R22.8.3.2: an advisory), a force not normal to the surface (a note).

Auto-design varies the plate length and width (1/2 in. steps) for the smallest loaded area; the support, its depth, f'c, the plate position and the loads stay yours.

Checks performed

Check	Basis
Applicability	22.8.1.2 (not post-tensioned anchorage zones); the plate on the support
A2	22.8.3.2: the frustum wholly inside the support, spread = min(edge distances, 2 × depth)
Bearing strength	Bn = 0.85 f'c A1 √(A2/A1) ≤ 2(0.85 f'c A1), or 0.85 f'c A1 (Table 22.8.3.2); φBn ≥ Bu (Eq. 22.8.3.1), φ = 0.65 (Table 21.2.1)
Tension in the plane of bearing	Advisory (R22.8.3.2)

The drawings show the plan with A1 and the frustum base A2 (dashed), and the section with the frustum sides.

Limitations

- A rectangular loaded area on a rectangular support; the frustum is taken similar and concentric with A1 (the usual reading of "the largest frustum"); wedge-shaped supports and other plan shapes: enter A2.

- The support (punching shear 22.6, the pedestal, wall or footing), dowels, anchor rods, the grout and the plate are not designed; confinement reinforcement is not credited.

Verification examples

Verification status: 11 values compared in 7 examples (3 published, 4 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	What it checks	Published	ILX Calcs
AISC DE J.6	22 × 22 in. plate on a 24 × 24 in. pedestal, f'c 3 ksi (AISC 360 J8 is the same expression)	Pu 690 kips, A2 576 in², φPp 875 kips	same
StructurePoint spread footing 5.1, 5.2	18 in. column on the footing (factor 2) and at the column base (f'c 5 ksi, no increase), ACI 318-14	1,070 kips, 895 kips	same

Hand calculations: an eccentric plate (the 2 in. edge distance governs), a thin slab (twice the depth governs), the factor 2 cap, and A2 entered.

CHAPTER 99

Joist and waffle slab

A ribbed (pan-joist) slab by ACI 318-19 9.8, or a two-way joist (waffle) slab by 8.8, designed per rib as a T-beam: the dimensional limits of joist construction, the slab thickness over the pans, the rib's flexure at midspan and at the support, shear with the 10 % increase of 9.8.1.5, minimum steel and deflection.

Inputs of the joist slab

- **Forming:** the pan (or dome) width at the top of the rib, the pan depth, the rib width at the bottom and the side taper of the forms, and the top slab thickness. The section is the tapered rib (in slices) under a flange one rib spacing wide. Standard sizes (20 and 30 in. pans, 8 to 20 in. deep; 19 and 30 in. waffle domes) are from the CRSI handbook, which is not on file: the forming dimensions are yours, from the form supplier.
- **Cover** (3/4 in. for joists not exposed, Table 20.5.1.3.1), aggregate size, materials.
- **Loads:** superimposed dead and live load per ft² (times the rib spacing) and the computed self-weight.
- **Moments and shears:** the coefficients of 6.5 (end or interior span, the discontinuous end, two or more spans; 6.5.1(c) is checked, (d) and (e) are yours to confirm), a simple span, or entered per rib. Two-way joists take entered moments: they come from the two-way slab analysis.
- **Reinforcement per rib:** bottom bars, top bars over the support (placed in the slab), optional stirrups.
- **Deflection:** the minimum depth of Table 9.3.1.1, or computed with Ie (24.2.3) against your limits.

Auto-design varies the pan depth (standard depths and your value) and the bottom and top bars per rib; the pan width, rib width, taper, slab, loads and spans stay yours.

Checks performed

Check	Basis
Joist limits	Rib width ≥ 4 in., depth ≤ 3.5 × minimum rib width, clear spacing ≤ 30 in. (9.8.1.2 to 9.8.1.4; 8.8.1.2 to 8.8.1.4); outside them: designed as slabs and beams (9.8.1.8), NOT FEASIBLE
Slab thickness	≥ max(clear spacing / 12, 2 in.) over removable forms (9.8.3.1)
Demands	Tables 6.5.2 and 6.5.4 with 6.5.1, simple span, or entered
Flexure	Strain compatibility (22.2), φ by εt, tension-controlled (9.3.3.1), As,min (9.6.1.2) at midspan and at the support
Shear	1.1 Vc of Table 22.5.5.1 at the minimum rib width (9.8.1.5); Av,min not required where Vu ≤ φVc (Table 9.6.3.1)
Deflection	Table 9.3.1.1 (with 9.3.1.1.1, 9.3.1.1.2), or Ie (Table 24.2.3.5) and λΔ (24.2.4)
Detailing	Integrity bar (9.8.1.6) and slab steel across the ribs (9.8.1.7) stated

The drawing is a section across two ribs with the bars and the forming dimensions.

Limitations

- One typical rib. The coefficients take equal adjacent spans for the negative moment (Table 6.5.2 note [1] uses their average). Concentrated loads, distribution ribs, solid heads at columns of a waffle slab and the slab reinforcement across the ribs are not designed.
- The computed deflection uses the elastic formula of the idealized end condition (simple 5/384, one end continuous 1/185, both ends 1/384) with Ie at midspan: an approximation; the code permits any method in reasonable agreement with tests (24.2.3).
- Wide-module joists (pans over 30 in.) are beams and slabs (9.8.1.8): use the continuous beam calculation.

Verification examples

Verification status: 16 values compared in 4 examples (0 published, 4 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published ACI 318-19 joist example is on file. The values are hand calculations (closed form): the rib geometry and self-weight, the coefficients of Tables 6.5.2 and 6.5.4 for end and interior spans, the 1.1 Vc of 9.8.1.5 with Table 22.5.5.1(c), the minimum depths of Table 9.3.1.1 and the slab thickness of 9.8.3.1.

CHAPTER 100

Continuous concrete beam

A nonprestressed rectangular or T-beam over two or more spans by ACI 318-19: moment and shear envelopes by analysis with the live-load patterns of 6.4.2, or by the coefficients of 6.5 where they apply; flexure at every span and support, shear, an optional torsion check, the cutoff points of the top bars (9.7.3) and the deflection of each span.

Inputs of the continuous beam

- **Materials:** f'c, the bar grade, the concrete type and density, Ec, the stirrup grade, the bar coating.
- **Section:** rectangular or T-beam (flange width by Table 6.3.2.1 from the least clear span, or entered), bw, h, cover to the stirrups, aggregate size.
- **Spans:** centre-to-centre lengths with the uniform superimposed dead and live loads (service) and the bottom bars of each span; **top bars at the supports** (one row per support from the left; a missing row repeats the last). The support width sets the faces. The web self-weight is added to D (the slab of a T-beam belongs in wD).
- **Analysis:** with the 6.4.2 patterns (the live load on the span and alternate spans, on adjacent spans, and on all spans), end supports pinned or fixed, interior supports as knife edges; or the coefficients of 6.5 with the exterior support condition and the stiff-column case (wu $\ell n^2/12$).
- **Shear and torsion:** stirrups (bar, legs, spacing), shear at d from the face, Tu with the torsion type, Al and θ.
- **Deflection:** your limits (Table 24.2.2), the sustained part of the live load and its duration.

Auto-design varies the overall depth (2 in. steps) and the stirrup spacing with the bars as you laid them out.

Checks performed

Check	Basis
Demands	Analysis with the 6.4.2 patterns and the ASCE 7-22 combinations, or Tables 6.5.2 / 6.5.4 with 6.5.1 (checked: two spans or more, adjacent clear spans within 20 %, L ≤ 3D)
Flexure	Each span (bottom bars, T-section) and each support face (top bars), strain compatibility (22.2), φ by εt, tension-controlled (9.3.3.1), As,min (9.6.1.2)
Shear	Vu at d from the face (9.4.3.2), Vc of Table 22.5.5.1, Vs, the section limit 22.5.1.2, Av,min (9.6.3.1), spacing (Table 9.7.6.2.2)
Torsion	22.7 (threshold, compatibility reduction, section limit, At/s, Al, minimums, spacing)
Cutoff points	Top bars: the inflection point of the negative envelope, one-third of the bars extended max(d, 12db, ℓn/16) beyond it (9.7.3.8.4), the others to where the continuing bars suffice plus max(d, 12db) (9.7.3.3), all at least ℓd from the face (9.7.3.4); bottom bars per 9.7.3.8.1 / 9.7.3.8.2 stated
Deflection	Table 9.3.1.1 reported; Ie (Table 24.2.3.5) at midspan applied to the analysis deflection with the live load on the span and alternate spans, λΔ (24.2.4)

The drawings show the beam elevation with its loads and the factored moment and shear envelopes.

Limitations

- Uniform dead and live loads only; columns are not modelled (knife-edge supports), so column moments and the restraint of the exterior columns come from the end condition you choose. No moment redistribution (6.6.5).
- One stirrup layout and one torsion moment for the whole beam; the governing span's shear is checked.
- The deflection scales the elastic deflection by Ig/Ie of the span (Ie at midspan, 24.2.3.7): an approximation the code permits in principle (24.2.3) but not a prescribed method.
- Cutoffs are computed lengths for the detailing; 9.7.3.5 for bars cut in a tension zone is yours.

Verification examples

Verification status: 12 values compared in 3 examples (1 published, 2 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Example	What it checks	Published	ILX Calcs
StructurePoint continuous T-beam (ACI 318-14)	Flange width, Ib of the T-section, minimum depth of the end span	130.5 in., 40,634 in⁴, 25.94 in.	same

The StructurePoint moments come from a frame with columns, which this calculation does not model, so they are not compared. Hand calculations (closed form): two equal spans with the patterns ($M_{u+} = 1.15^2\,\ell^2\,w / 5.6$, $M_{u-} = w_u\,\ell^2/8$, V_u at d), the coefficients of Tables 6.5.2 and 6.5.4 for three spans, and the embedment of the top bars beyond the point of inflection (9.7.3.8.4).

CHAPTER 101

Prestressed concrete beam

A simply supported pretensioned or post-tensioned beam with bonded or unbonded tendons by ACI 318-19: tendon stress limits, losses (lump sums or the Zia et al. expressions), stresses at transfer and at service with the class of Table 24.5.2.1, flexural strength with fps of 20.3.2, the 1.2 Mcr minimum, shear by Vci and Vcw, camber and deflection.

Inputs of the prestressed beam

- **Materials:** f'c and f'ci at transfer, the concrete type, Ec; the bar grade for As and the stirrups.
- **Section and span:** rectangular or T-section, the simple span.
- **Prestressing:** pretensioned (bonded) or post-tensioned (bonded or unbonded), Aps, the strand diameter (transfer length 50 db), fpu and the steel type (fpy/fpu and γp), the jacking stress, the tendon profile (straight, parabolic, harped at a fraction of the span) by its eccentricities below the centroid at the ends and at midspan, bonded mild steel As and its depth.
- **Losses:** lump sums at transfer and in total (the supplier's or your values), or the Zia et al. (1979) expressions for elastic shortening, creep, shrinkage and relaxation with the relative humidity, V/S, Ksh and C, plus friction and seating for post-tensioning. These expressions are not in ACI 318 (20.3.2.6 lists what losses include); they are written from knowledge of the paper as the PCI Design Handbook gives them, marked "to confirm".
- **Loads:** the self-weight, superimposed dead and live load (uniform), the sustained part of the live load; whether bonded reinforcement resists the tension at transfer (24.5.3.2.1).
- **Shear and deflection:** stirrups; the live load deflection limit; the long-term multipliers (PCI values prefilled, from knowledge).

Auto-design varies the depth (2 in. steps) and the number of strands; the profile, losses and loads stay yours.

Checks performed

Check	Basis
Jacking stress	≤ 0.94 fpy and 0.80 fpu; ≤ 0.70 fpu at post-tensioning anchorages after transfer (Table 20.3.2.5.1)
Losses	Lump sums or Zia et al. (1979) [to confirm]; fse ≥ 0.5 fpu for the approximate fps (20.3.2.3.1, 20.3.2.4.1)
Transfer stresses	0.70 f'ci / 6√f'ci at the end (transfer length or anchorage), 0.60 f'ci / 3√f'ci at midspan (Tables 24.5.3.1, 24.5.3.2)
Class	U (≤ 7.5√f'c) or T (≤ 12√f'c) at service (Table 24.5.2.1); Class C NOT FEASIBLE here (24.5.2.3)
Service compression	0.45 f'c sustained, 0.60 f'c total (Table 24.5.4.1)
Flexure	fps by Eq. 20.3.2.3.1 (bonded) or Table 20.3.2.4.1 (unbonded); rectangular or T block; φ by εt with εty = 0.002

Check	Basis
Minimum reinforcement	φMn ≥ 1.2 Mcr with bonded tendons (9.6.2.1, unless 9.6.2.2); As ≥ 0.004 Act with unbonded tendons (9.6.2.3)
Shear	Vci and Vcw (22.5.6.3) at h/2 and every tenth of the span to midspan, prestress reduced in the transfer length; Vs; Av,min (Table 9.6.3.4); spacing
Deflection	Camber and self-weight at transfer (Eci), live load with Ig (Class U) or Ie (Class T, approximation), final by the multipliers

The drawings show the elevation with the tendon centroid and the midspan section.

Limitations

- One simple span with uniform loads; continuous members and their secondary moments, composite toppings, point loads, debonding and the anchorage zones (25.9) are not covered. Continuous post-tensioned slabs: the PT slab strip.
- fps by the approximate expressions only (no strain compatibility); for a T-section ρp uses the flange width.
- The Class T deflection uses Ie with the tendons and bars transformed and the axial prestress ignored in Icr; the long-term multipliers are planning values.
- The loss expressions and the multipliers are from knowledge (Zia et al. 1979; PCI Design Handbook), not on file.

Verification examples

Verification status: 19 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 19 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No published prestressed beam example is on file (the PCI Design Handbook and the PTI manual are not). The values are hand calculations (closed form): the transfer and service fiber stresses, fps of Eq. 20.3.2.3.1, the stress block and Mn, Mu, Mcr, Vcw at midspan, the live load deflection and the camber of a harped tendon for the default 12 × 28 in. beam; the Zia et al. losses as the module writes them; fps of an unbonded tendon and As,min = 0.004 Act.

CHAPTER 102

Tilt-up wall panel

A site-cast concrete wall panel that spans vertically from its base to the roof, designed out of plane by the alternative slender wall method of ACI 318-19 11.8 with the ASCE 7-22 2.3.1 load combinations (strength design). A solid panel is one design strip over its full width. A panel with a door or window opening is designed as two design strips (legs), one on each side of the opening, by the approach of ACI 551.2R-15 7.2 as the StructurePoint tilt-up examples apply it: each leg carries the loads of its own width plus one half of the opening.

Inputs of the tilt-up panel

- **Panel:** width, the height ℓc between the base and the roof support (the simply supported span of 11.8.2.1), the parapet height above the roof support, the structural thickness h (deduct reveals and form liners yourself) and the self-weight (150 pcf prefilled). The parapet weight is included above midheight; wind on the parapet, which would reduce the midheight moment, is conservatively not counted.
- **Opening:** none (solid panel), a door (from the base) or a window (with a sill height), its width and height and its distance from the left panel edge. Each leg must be at least 1 ft wide and the opening head must stay at least 6 in. below the roof support. The **design strip width limit** (12h prefilled) is the guideline of ACI 551.2R-15 7.2 ("about 12 times the panel thickness", not an ACI 318 limit): a wider leg counts only this width for strength and stiffness, while it still carries the loads of the whole leg plus half the opening.
- **Reinforcement:** one layer at the center or two layers (one at each face, with the clear cover), the vertical and horizontal bars and spacings (per layer), the bars added at each jamb (left and right separately; with two layers, the total of both faces, half of which are taken on the tension face), and the header bars above the opening. The vertical steel of a strip is As = Ab b / s plus the jamb bars (the area per width, not a bar count).
- **Materials:** f'c, Grade 60 or 80, normalweight or lightweight concrete and Ec, as in the other concrete types.
- **Loads:** roof reactions at points along the top (joists or girders: load type, value, position from the left edge and eccentricity from the panel centerline), line loads along the top (ledger, deck) with their eccentricity, the strength-level wind pressure W and an optional strength-level out-of-plane seismic pressure E (for example ASCE 7-22 12.11.1). With an opening, the roof loads are shared between the legs as a simple beam across the panel top supported at the leg centerlines (statics; loads outside the centerlines are cantilevered); the lateral pressure acts on the tributary width (leg + half the opening) over the full height, which is conservative for a window.
- **Code options:** the row of Table 11.6.1 (a tilt-up panel is cast elsewhere than its final position and so is precast by the ACI 318 definition; the cast-in-place row is the conservative choice of the StructurePoint and ACI 551 examples and is prefilled), the precast wall exposure (spacing limit 18 in. exterior, 30 in. interior, 11.7.2.2) and the service factors on L-type loads (0.5), W (0.6) and E (0.7) for the deflection check (R11.8.4.1: D + 0.5L + Wa, D + 0.5L + 0.7E; Wa from serviceability wind speeds may be less than 0.6W).

Auto-design (editor toolbar) varies the thickness (1/4-in. steps), the vertical and horizontal bars (size and spacing) and the added jamb bars at each side of the opening. Objectives: lowest cost index (concrete, reinforcement); thinnest panel, then cost; least reinforcement. Every other input is kept; see Auto-design.

Checks of the tilt-up panel

Every strip is checked for every ASCE 7-22 2.3.1 combination of the load types present; the step list shows the governing one.

Check	Basis
Out-of-plane flexure with P-Δ	Pum = Pua + factored self-weight above midheight; Mua = wu ℓc²/8 + Pua e/2; Icr = n Ase (d − c)² + b c³/3 with n = Es/Ec ≥ 6 and Ase = As + Pu h/(2 fy d) (Eq. 11.8.3.1c, R11.8.3.1); Mu = Mua / (1 − Pum/(0.75 Kb)), Kb = 48 Ec Icr / (5 ℓc²) (Eq. 11.8.3.1d, the closed form of 11.8.3.1(b)); Δu = Mu / (0.75 Kb) (Eq. 11.8.3.1b); against φMn at Pum by strain compatibility (22.2), φ = 0.90
Tension-controlled section	εt ≥ εty + 0.003 for every combination (11.8.1.1(b), Table 21.2.2)
φMn at least Mcr	Mcr = fr Ig / (h/2), fr = 7.5λ√f'c (11.8.1.1(c), 19.2.3.1)
Axial stress	Pum / Ag ≤ 0.06 f'c at midheight (11.8.1.1(d))
Out-of-plane shear	Vu = wu ℓc / 2 against φVc of Table 22.5.5.1(c) with λs (11.5.5.1), axial compression neglected
Service deflection	Δs by Table 11.8.4.1 with Ma = Msa + Ps Δs iterated (Eq. 11.8.4.2), Δcr and Δn by Eqs. 11.8.4.3a and b (Mn and Icr at the service axial force), Δs ≤ ℓc/150 (11.8.1.1(e)); service combinations D + (factor) W or E with and without each L-type load
Minimum reinforcement	Table 11.6.1, cast-in-place or precast row (the engineer's choice)
Bar spacing	3h and 18 in. (11.7.2.1, 11.7.3.1) or, for precast, 5h and 18 / 30 in. (11.7.2.2, 11.7.3.2)
Two layers	Walls thicker than 10 in. (11.7.2.3)
Bars around the opening	At least one No. 5 bar (one layer) or two (two layers) at each jamb and at the header, in addition to the minimum reinforcement (11.7.5.1)
Strip method	Advisory when the opening is wider than ℓc/2 (ACI 551.2R-15 7.2 guideline)

The drawings (on screen, in the report and on the DXF / PDF drawing sheet) show the panel elevation (outline, parapet, roof support line, opening, widths and heights), the typical wall section through the (left) design strip (supports, roof reaction with its eccentricity, the lateral pressure over ℓc, the vertical bars) and a plan section at the jamb (thickness, vertical bars at their spacing from the opening edge, jamb bars, horizontal bars, cover). Bar lengths, laps, header bar anchorage, embeds, lifting inserts, braces and the footing are not part of the calculation.

Limitations

- One opening at most; a panel with two or more openings, a multi-story (continuous) panel or a panel with a thickened pilaster is not covered.
- In-plane shear and overturning of the panel (a shear wall), the header above the opening as an in-plane deep member, and the connections (ledger, embeds, roof and base anchorage) are not designed; 11.8.2.2 (spreading of concentrated loads) is replaced by the strip tributary widths.
- **Lifting, stripping and bracing are not checked.** The methods for handling stresses (insert patterns, impact factors, the stress limit against the modulus of rupture) are in ACI 551.2R-15 Chapter 8 and the Tilt-Up Concrete Association manuals, which are not freely available; no free worked example could be obtained (a free MIT thesis on lifting stresses could not be downloaded). See research/DOCUMENTS_NEEDED.md.

- Only the bars at the tension face count in φMn and Icr (the compression layer of a two-layer panel is neglected, conservative); the compression and tension directions of W and E are taken as the same magnitude.
- The lateral load acts over ℓc only; wind on the parapet and the base fixity are neglected. Ev and the Ω0 load combinations of ASCE 7-22 are not modeled.

Verification examples

Verification status: 66 values compared in 5 examples (3 published, 2 hand calculations from the cited equations): 66 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The panel is verified against the two free StructurePoint ACI 318-19 tilt-up examples (ACI 551.2R-15 Examples B.1 and B.2) and hand calculations derived step by step in `tests/test_verify_tilt_up_panel.py` (66 values).

Example	Source	What it checks	Published	ILX Calcs
SP-TU	StructurePoint, Tilt-Up Wall Panel (ACI 318-19 – ACI 551), pp. 8–12, 36	Solid 15 ft × 6.25 in. panel, 16 #6, ℓc = 29.5 ft, 1.2D + 1.6Lr + 0.5W	Self-weight above midheight 19.04 kips, Pum = 43.49 kips, Mua = 24.77, Mu = 61.00, Mn = 106.55, Mcr = 46.32 kip-ft, Icr = 353.56 in⁴, Kb = 97.64 kips; service Ps = 26.24 kips, Msa = 20.32, Ma = 20.84 kip-ft, Δcr = 0.550, Δs = 0.247 in.	equal within 0.1 %
SP-TUO-L	StructurePoint, Tilt-Up Wall Panel with Opening (ACI 318-19 – ACI 551), pp. 8–13, 27	Left leg, 4 ft + 5 ft tributary, 7 #6	Joist loads 4.48 / 4.67 kips, Pum = 31.87 kips, Mua = 14.92, Mu = 31.68, φMn = 60.13, Mcr = 24.21 kip-ft, Icr = 290.85 in⁴, Kb = 80.32 kips, Δu = 6.311 in., Pum/Ag = 75.89 psi, Ma = 12.55 kip-ft, Δs = 0.203 in.	equal within 0.5 %; φMn 60.31 (+0.3 %)
SP-TUO-R	Same, p. 14	Right leg, 6 ft + 5 ft tributary, 7 #6	Joist loads 5.12 / 5.33 kips, Pum = 37.97 kips, Mua = 18.11, Mu = 37.38, φMn = 65.35, Mcr = 36.32 kip-ft, Icr = 355.58, Ig = 4,019.53 in⁴, Kb = 98.20 kips, Δu = 6.091 in., Δs = 0.164 in.	equal within 0.5 %; φMn 65.50 (+0.2 %)

The examples compute φMn with Ase from Pum and the stress block from Pum/φ; ILX Calcs uses strain compatibility at Pum, which for a single centered layer is Mn = (As fy + Pu)(d − a/2) and matches the solid-panel example exactly.

Hand calculations:

Example	What it checks
TU-1	Solid 12 ft × 7.25 in. panel, #5 @ 12 in., ℓc = 24 ft, W = 30 psf, 0.9D + 1.0W: Pu, Mua, Icr, Mu, φMn, Δu, Mcr in closed form
TU-2	Design strips: a roof load shared between the legs by statics, the self-weight above midheight of a leg with the panel above a door head, the tributary wind

No free worked example of lifting stresses or of a panel with a window (sill) was found; the window case uses the same strip equations with the self-weight area checked in `tests/test_tilt_up_panel.py`.

CHAPTER 103

Concrete stair

A straight waist-slab stair, with or without landings, designed per 12 in. strip as a one-way slab to ACI 318-19 Chapter 7 with the ASCE 7-22 2.3.1 load combinations (strength design). The stair is analyzed on its horizontal projection: under vertical loads the bending moment of the kinked slab equals that of the projection, the shear normal to the flight is V cos θ, and the vertical deflection of the inclined waist is that of a member of stiffness Ec Ie cos θ on the projection.

Inputs of the concrete stair

- **Materials:** f'c, Grade 60 or 80 bars, normalweight or lightweight concrete, Ec by 19.2.2.1 (as the other concrete types).
- **Stair:** riser R and tread T, the number of risers (the flight has one tread fewer: the top landing is the last step), the waist thickness t measured perpendicular to the soffit from the inside corners of the steps, and the supports: at the outer ends of the landings (bottom and top landing lengths, 0 = none, and the landing thickness), or at the ends of the flight (landings carried by beams or walls of their own). Each end is pinned (simple bearing) or fixed / continuous (built into a wall or beam that restrains rotation: top bars are then designed). The support width sets the critical section for shear at d from its face (7.4.3.2). The stair width is used for the drawings; the design is per 12 in. strip. Clear cover: 3/4 in. prefilled (Table 20.5.1.3.1).
- **Reinforcement:** bottom (main) bars and spacing, top bars and spacing at fixed ends, distribution bars across the stair (24.4.3).
- **Loads:** area loads on plan per load type (finishes, live load; ASCE 7-22 Table 4.3-1: 100 psf for stairs, 40 psf in one- and two-family dwellings) and the self-weight: $\gamma(t\sqrt{(R^2 + T^2)}/T + R/2)$ on the flight (the waist and the triangular steps) and γ tl on the landings.
- **Deflection:** span / n limits for the immediate live load deflection and the deflection after attachment (Table 24.2.2), the load duration (ξ) and the sustained part of the live load: the engineer's.

Auto-design (editor toolbar) varies the waist thickness (1/2-in. steps; the landings follow when they had the same thickness) and the bottom, top and distribution bars (size and spacing). Objectives: lowest cost index (concrete, reinforcement); thinnest waist, then cost; least reinforcement. The steps, landing lengths, supports and loads stay the engineer's; see Auto-design.

Checks of the concrete stair

Check	Basis
Flexure, bottom bars	the largest positive Mu in the flight and in each landing against φMn of that part's thickness by strain compatibility (22.2, φ from Table 21.2.2), per 12 in.
Flexure, top bars	the negative Mu at each fixed / continuous end (7.5.2.1, 22.2)
Tension-controlled section	$\varepsilon t \geq \varepsilon ty + 0.003$ (7.3.3.1, as printed)

Check	Basis
Minimum reinforcement	As ≥ 0.0018 b h per part (7.6.1.1)
Bar spacing	≤ min(3h, 18 in.) (7.7.2.3) and the crack-control spacing of Table 24.3.2 with fs = 2/3 fy (24.3.2.1)
Distribution bars	≥ 0.0018 Ag of the thicker part, spacing ≤ min(5h, 18 in.) (24.4.3.2, 24.4.3.3)
One-way shear	Vu (normal to the flight in the flight) ≤ φVc with Vc = 8λsλ(ρw)^(1/3)√f'c b d (Table 22.5.5.1(c)), at d from the support face (7.4.3.2), no stirrups (7.6.3.1)
Minimum thickness	Table 7.3.1.1 on the horizontal span (ℓ/20, ℓ/24, ℓ/28 by the end conditions, × (0.4 + fy/100,000)): a reference only; the deflections are always calculated
Deflections	Ie of Table 24.2.3.5 for the waist at the largest positive service moment (24.2.3.7), stiffness Ec Ie cos θ over the whole span; immediate ΔL and λΔ(ΔD + sustained ΔL) + ΔL (24.2.4.1) against the entered limits (Table 24.2.2)
Notes	re-entrant corners (bars cross at the top of the flight and are developed past the kink), the 300 lb tread load of ASCE 7-22 Table 4.3-1, ℓd of the bottom bars (25.4.2.4)

Limitations

- Straight flights spanning in their own direction only; stairs spanning across the width (between stringers or walls), cantilevered treads, free-standing (dog-leg without support, helical) stairs and stairs with intermediate supports are not covered.
- The moments come from an elastic analysis of the projection with a uniform stiffness; for fixed ends the actual distribution varies somewhat with the different thicknesses and the slope of the flight.
- The deflection uses the flight's stiffness (Ec Ie cos θ) over the whole span, conservative where the landings are as thick or thicker; Ie is taken at the largest positive service moment.
- The axial force in the flight is neglected (it is small and compressive near the bottom support).
- The bar lengths, the extension past each kink, the top-bar extent, hooks and laps, and the supporting walls or beams are not designed.
- No free published worked example of an ACI stair is on disk; the module is verified by hand calculations (research/DOCUMENTS_NEEDED.md).

Verification examples

Verification status: 11 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Hand calculations in closed form (`backend/tests/test_verify_concrete_stair.py`): R = 7 in., T = 11 in., 10 risers (flight run 8.25 ft), waist 8 in., f'c = 4,000 psi, Grade 60, #5 at 6 in. bottom, D = 15 psf finishes plus the self-weight, L = 100 psf. The flexure, minimum steel and effective moment of inertia used are those of the one-way slab, which is verified against the StructurePoint one-way slab example (DE-OWS).

Example	What it checks	Hand calculation	ILX Calcs
ST-1	Flight only, pinned ends	flight self-weight 162.3 psf on plan, Mu = 3.171 kip-ft, φMn = 18.08 kip-ft, Vu = 1.011 kips (V cos θ at d from the face), φVc = 6.169 kips, ΔL = 0.0067 in. (uncracked, Ec Ig cos θ)	equal
ST-2	Flight with two 3 ft landings, pinned at the landing ends	Mu at midspan = 9.125 kip-ft	equal
ST-3	Flight only, fixed at both ends	Mu− = wu ℓ²/12 = 2.114 kip-ft, φMn of #4 at 12 in. = 6.168 kip-ft, Mu+ = wu ℓ²/24 = 1.057 kip-ft	equal

CHAPTER 104

Special moment frame members

The beam and the column of a reinforced concrete special moment frame by ACI 318-19 18.6 and 18.7, in one frame direction: the dimensional limits, longitudinal bar limits and strengths, the hoops of the beam hinge zones and the column confinement over ℓo, the strong-column rule at the joint above the column, and the capacity-design shears from the probable moments Mpr. The joint itself is the *Beam-column joint* type.

Inputs of the special moment frame members

- **Materials:** f'c, Grade 60 or 80; Mpr and Mn by strain compatibility with all the bars (default: counting the compression bars gives a slightly larger Mpr and so the safer shear) or as a singly reinforced rectangle.
- **Beam:** width, depth, clear span, bar centroid from the faces, top and bottom bars (continuous over the span), the slab bars inside the effective width (they count in Mnb− for 18.7.3.2 when developed), hoops and spacings.
- **Beam loads:** dead and live load per foot for the 18.6.5.1 gravity shear with Ev = 0.2SDS D, the live load factor (1.0, or 0.5 where 2.3.6 permits it), and the factored face moments of your analysis.
- **Column below the joint:** c1 (in the frame direction), c2, clear height, bars per face, cover, hoops with the number of legs (hoop legs and crossties) in each direction, the spacings within and beyond ℓo.
- **Column forces:** the range of factored axial force with E, the factored moment and shear of the analysis, and the axial force range of the column above (same section) for the strong-column check; interior or exterior joint.

Auto-design varies the column bar size and the hoop spacings of the beam and the column; sections, beam bars and forces are kept.

Checks performed

Check	Basis
Beam span, width, projection	18.6.2.1 (a) to (c)
Beam bars	Two continuous bars each face, As,min (9.6.1.2), ρ ≤ 0.025 / 0.02 (18.6.3.1); Mn+ ≥ Mn−/2 (18.6.3.2)
Beam flexure at the face	φMn ≥ Mu, φ by Table 21.2.2
Beam hoops	s ≤ d/4, 6 in., 6db / 5db within 2h (18.6.4.4); d/2 beyond (18.6.4.6)
Beam shear	Ve = (Mpr− + Mpr+)/ℓn + wu ℓn/2 (18.6.5.1); Vc = 0 where the earthquake part ≥ Ve/2 (18.6.5.2)
Column dimensions and steel	18.7.2.1; 0.01 to 0.06 Ag (18.7.4.1)
Column axial and flexure	φPn,max; φMn at the axial force of the range that governs (strain compatibility)
Strong column	ΣMnc ≥ 6/5 ΣMnb (Eq. 18.7.3.2), Mnc at the least-strength axial force, slab bars in Mnb−

Check	Basis
Confinement	ℓo (18.7.5.1); hx ≤ 14 in. or 8 in. (18.7.5.2); s ≤ b/4, 6db / 5db, so (18.7.5.3); Ash of Table 18.7.5.4 (a), (b), (c) with kf, kn, both directions; spacing beyond ℓo (18.7.5.5)
Column shear	Ve = 2Mpr/ℓu over the Pu range, not more than ΣMpr,b/ℓu, not less than Vu (18.7.6.1); Vc = 0 per 18.7.6.2.1

The drawings are the beam section at the joint face and the column section within ℓo with the bars and hoops checked.

Limitations

- One frame direction, rectangular sections, the beam's two ends equal; circular columns, walls and prestressed beams are not covered.
- The joint, lap splice locations, bar development, hooks and crosstie details and the frame analysis are yours.
- For 18.6.5.2 (b) the beam axial force is taken as below Ag f'c/20 (enter beams with larger axial force elsewhere); for 18.7.6.2.1 (b) the least column axial force of the range is used.

Verification examples

Verification status: 13 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published special moment frame example with these numbers is on disk. Hand calculations (closed form) check the beam Mn and Mpr of a singly reinforced section, wu and Ve, ℓo, hx and so, the Table 18.7.5.4 ratios and kf, kn.

CHAPTER 105

Ledge and inverted-tee beam

The ledge of a spandrel (L-beam) or an inverted-tee beam carrying repeated bearings (double-tee stems, joists): the punching shear of the ledge, its flexure and direct tension, the hanger reinforcement that lifts the load into the web, the bearing, and the shear and torsion of the beam. The ledge rules follow the PCI Design Handbook ledge method, which is not on disk: those steps are written from knowledge and marked "to confirm against the code text". ACI 318-19 (on disk) gives the bearing, the shear and the torsion (22.5, 22.7, 22.8).

Inputs of the ledge

- **Beam and ledge:** spandrel or inverted tee (two ledges loaded alike), web width and depth, ledge projection and depth, cover, span, other dead load on the beam.
- **Bearings:** spacing along the beam, bearing width (along the beam) and length (across the ledge), the distance av from the web face to the load, the distance de from the beam end to the first load, the loads per bearing by load type, and the horizontal force as a fraction of Vu (0.2 usual).
- **Reinforcement:** ledge bars and spacing, hanger bars and spacing (the legs on the ledge side), closed web stirrups and the longitudinal torsion bars provided.

Auto-design varies the ledge depth and the ledge bar, hanger and stirrup spacings.

Checks performed

Check	Basis
Ledge punching, interior and near the end	Vn = 3λ√f'c hl (2 le + w), w = least of bt + hl and s; at the end half of it plus the end distance (PCI, to confirm)
Ledge flexure and direct tension	As = (Vu av + Nu (hl − d))/(φ fy d) + Nu/(φ fy) within bt + 5av, not more than s (PCI, to confirm; ACI 318-19 16.5 analogy)
Hangers	Ash = Vu/(φ fy) within bt + 3av, not more than s (PCI, to confirm)
Bearing	ACI 318-19 22.8.3.2, φ = 0.65
Beam shear	ACI 318-19 22.5 at the support with the bearing loads as a line load
Spandrel torsion	Tu = (Vu/s) e L/2 at the support, ACI 318-19 22.7 (section size, stirrups, Al, spacing)

The drawing is a section at a bearing with the ledge bars and hangers.

Limitations

- The PCI ledge equations, effective widths and end rules are from knowledge (to confirm against the code text).
- Loads of one size at one spacing; the strut-and-tie alternative, the beam's flexure, dapped ends and connections are not covered. The torsion is uniformized over the span.

Verification examples

Verification status: 8 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

No published ledge example is on disk. Hand calculations (closed form) check Vu, the punching strength, the ledge and hanger steel, the bearing and the spandrel torsion, and the spacing limit on the punching width.

CHAPTER 106

Hollow-core plank

One simply supported precast, pretensioned hollow-core unit, untopped or with a composite topping, by ACI 318-19: stresses at transfer and at service, flexural strength with fps of Eq. 20.3.2.3.1, the 1.2 Mcr minimum, strand development, shear by Vci and Vcw without shear reinforcement, horizontal shear at the topping and the camber at release. The section comes from the producer (or a Dynaspan section of the PCI Design Handbook 8th Ed. Table 3.7).

Inputs of the hollow-core plank

- **Section:** depth, area, centroid, moment of inertia and weight of the untopped unit, the unit width, the total web width (for shear) and the top flange above the cores (for the stress block), f'c and f'ci.
- **Strands:** count, size, height of their centroid, jacking stress and the losses at transfer and in total (lump sums: the producer's or a loss calculation).
- **Topping:** thickness and f'c; it is transformed by √(f'c,top / f'c), as the PCI table does.
- **Span and loads:** design span, bearing length, superimposed dead and live load and the sustained part.

Auto-design varies the strand size and count; the section, concretes, topping, span, loads and losses are kept.

Checks performed

Check	Basis
Transfer stresses at the end of ℓtr = 50db and at midspan	0.70 / 0.60 f'ci, 6√f'ci / 3√f'ci (24.5.3, 22.5.9.1)
Service stresses	Class U / T by the bottom tension (Table 24.5.2.1); top compression 0.45 / 0.60 f'c (Table 24.5.4.1)
Flexural strength	fps (Eq. 20.3.2.3.1, γp = 0.28), stress block in the topping, flange and webs; φ by εt (21.2.2)
Minimum strength	φMn ≥ 1.2 Mcr (7.6.2.1)
Strand development	ℓd (Eq. 25.4.8.1) within half the span
Shear at h/2	Vc = least of Vci and Vcw with the prestress over ℓtr (22.5.6.3); no stirrups: Vu ≤ φVc
Horizontal shear (topped)	80 bv d, roughened surface without ties (Table 16.4.4.2 (d))
Camber (note)	Release camber from Pi and the self-weight; PCI long-time multipliers (to confirm against the code text)

The drawings show the plank elevation with its loads and the section with the topping, the strands and schematic cores.

Limitations

- Simple spans with uniform loads only; continuity, cantilevers, openings, point and line loads, diaphragm action and fire ratings are not covered.
- Losses are entered; the camber multipliers of the PCI Design Handbook are from knowledge (to confirm against the code text). The hollow-core rows of Table 9.6.3.1 are not readable in the text copy: the deep-unit advisory is to confirm.
- The stress block uses strength-equivalent widths where it passes from the topping into the precast.

Verification examples

Verification status: 12 values compared in 5 examples (4 published, 1 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	What it checks	Published	ILX Calcs
PCI Table 3.7, Dynaspan 4, 6, 8, 10 in. with 2 in. topping	Composite ycb and Ic from the untopped values	e.g. 8 in.: 5.15 in., 3,205 $in.^4$	same

The 12 in. row of the table is not compared: it matches the topping at its full width, unlike the other rows. Hand calculations check fps, fse, ℓd and the depth of the stress block.

CHAPTER 107

Concrete deflection

Immediate and long-term deflections of a continuous line of nonprestressed concrete beams or one-way slabs by ACI 318-19 24.2: cracked sections at midspan and at every support, Ie by the 2019 expression of Table 24.2.3.5 at three service load levels in sequence (D, D + sustained L, D + L), the Ie of each span by the method you choose, the time-dependent factor λΔ and the limits of Table 24.2.2. *Concrete beam* checks the deflection of the member it designs with Ie at midspan; this type is the detailed, multi-span version for a beam whose bars are known.

Inputs of the concrete deflection

- **Materials:** f'c, the concrete type and density, Ec by 19.2.2.1.
- **Section:** rectangular or T (the effective flange of 6.3.2 is yours); the negative regions as the web rectangle (the usual simplification) or as the T with its flange in tension; bar centroids from the top and bottom faces.
- **Spans and reinforcement:** one to eight spans, left to right, each with its bottom bars and the top bars at its right support; the top bars at the left end; compression bars at midspan for ρ'; pinned or fixed ends.
- **Service loads:** superimposed dead load, self-weight, live load (for flat roofs the largest of Lr, S and R), the sustained part of the live load (yours), live load on every span or the checkerboard pattern of each span.
- **Method and limits:** Ie at midspan (24.2.3.7), the average of the positive and negative sections (24.2.3.6), the ACI 435 weighted average (0.70 Iem + 0.15 (Ie1 + Ie2); end spans 0.85 Iem + 0.15 Ie,cont), or Ie varying along the beam with the moment; ξ of Table 24.2.4.1.3; the part of the time-dependent deflection that occurs before attachment (footnote 2 of Table 24.2.2, 0 by default); the limits as span / n.

Auto-design varies the depth h (8 to 60 in.); the spans, loads, bars, widths, the method and the limits are kept.

Checks performed

Check	Basis
Section properties	Ig, Mcr = fr Ig / yt (Eq. 24.2.3.5), Icr of the cracked transformed section
Minimum depth (note)	Table 9.3.1.1 (beams) or 7.3.1.1 (slabs), with the fy and lightweight modifications
Immediate live load deflection, each span	Δ(D + L) − Δ(D) against ℓ/n (Table 24.2.2)
Deflection after attachment, each span	(1 − before) λΔ Δ(D + sustained L) + ΔL against ℓ/n (24.2.4.1, Table 24.2.2)

The moments of each level come from an elastic analysis with Ig; the deflections from a second analysis with the stiffness EcIe of each span (or of each element for the varying method). The drawings show the beam elevation with its loads and the immediate deflection under D + L.

Limitations

- Prestressed members, two-way slabs, cantilevers and haunched members are not covered.

- The moments are those of a continuous beam on knife-edge supports: support rotations through the columns (a frame) are not modelled.
- The bars are taken over the whole region they are entered for; cut-offs are not modelled.

Verification examples

Verification status: 14 values compared in 4 examples (2 published, 2 hand calculations from the cited equations): 14 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Example	What it checks	Published	ILX Calcs
StructurePoint DE-CNB-ACI14, Table 3	Negative region 16 × 24 in., 7 #8 at 22 in.	Ig 18,432 in.⁴, Mcr 60.72 kip-ft, kd 8.42 in., Icr 10,897 in.⁴	same
Same, Table 4	T section, 120 × 8 in. flange	Ig 39,684 in.⁴, Mcr+ 89.77, Mcr− 240.36 kip-ft	same

The example's Ie and deflections use ACI 318-14 (Branson) and a frame with columns, so they are not compared. Hand calculations check a simple span, uncracked ($5wL^4/384EcIg$) and cracked (Ie of Table 24.2.3.5, $\lambda\Delta = 2.0$).

CHAPTER 108

Liquid-containing tank wall

A concrete wall of a rectangular or circular tank under the hydrostatic pressure of a full tank: the vertical moments and shear, the hoop tension of a circular wall, flexure and hoop steel with the environmental durability factor Sd, shear without stirrups, and the liquid-tightness detailing rules (minimum steel by the distance between movement joints, thickness, cover, spacing). ACI 350-20 is not readable on disk and the PCA tank tables are not on disk: the ACI 350 rules are written from knowledge of ACI 350-06 and marked "to confirm against the code text"; the walls are analysed in closed form.

Inputs of the tank wall

- **Materials:** f'c and the bar grade.
- **Tank:** rectangular (a vertical strip per foot: cantilever from a fixed base, fixed base and pinned top, or pinned at both ends: your choice) or circular (fixed or free sliding base, inside radius, Poisson's ratio); liquid depth, wall height, liquid unit weight.
- **Wall:** thickness, cover, the vertical bars at the face in tension at the base and their spacing, the horizontal bars on each face, the exposure (normal or severe, yours) and the distance between movement joints.

Auto-design varies the thickness and the vertical and horizontal bar spacings; the bar sizes and the rest are kept.

Checks performed

Check	Basis
Service effects	Strip: $\gamma H^3/6$, $\gamma H^3/15$, $\gamma H^3/(9\sqrt{3})$ moments; cylinder: free base $T = \gamma Ha$, fixed base Timoshenko §117 (to confirm)
Vertical flexure	$Sd\ Mu \le \varphi Mn$, $Mu = 1.4\ Ms$ (ASCE 7-22 2.3.1), $Sd = \varphi\ fy/(\gamma\ fs) \ge 1$, $fs = 320\ (260) / (\beta \sqrt{(s^2 + 4(2 + db/2)^2)})$ (ACI 350, to confirm)
Hoop tension (circular)	$As \ge Sd\ Tu/(\varphi\ fy)$, fs = 20 ksi (17 ksi severe) (to confirm)
Shear	$Vu \le \varphi\ 2\lambda\sqrt{f'c}\ b\ d$ (ACI 318-19 22.5.5.1)
Minimum horizontal steel	ρ by the movement joint spacing (0.3 %, 0.4 %, 0.5 %) (to confirm)
Thickness, cover, spacing	12 in. for walls 10 ft or taller, 2 in. cover, 12 in. spacing (to confirm)
Long shell (advisory)	$\beta H \ge 3$ for the fixed-base cylinder solution

The drawing is a wall section with the liquid level, the pressure and the bars.

Limitations

- Full tank, hydrostatic only: the empty tank with backfill or groundwater, sloshing (ASCE 7-22 15.7.6), temperature, two-way action of rectangular walls and short circular walls (βH < 3) are not covered.
- Every ACI 350 value and the cylinder solution are from knowledge (to confirm against the code text).

Verification examples

Verification status: 7 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

No published tank example is on disk. Hand calculations (closed form) check the three strip moments, fs and Sd, the hoop tension of a free-base cylinder and the base moment of a fixed-base long cylinder.

CHAPTER 109

Concrete fire resistance

The prescriptive fire-resistance requirements of a concrete wall, floor or roof slab, beam or column: the minimum thickness, size and cover for a required rating of 1 to 4 hours, by the aggregate of the concrete. The method is ACI 216.1 / TMS 216.1 as the IBC adopts it in Section 722.2; the masonry counterpart is the masonry fire resistance type. US and metric units.

Edition. IBC 2021 Section 722.2 (pp. 7-65 to 7-72) is on hand and is the basis: it is the code's own copy of the ACI 216.1 tables. ACI 216.1 / TMS 216.1 itself is not on hand; differences in its current edition are to be confirmed against the code text.

Inputs of the concrete fire resistance

- **Element:** wall, floor or roof slab, beam or column.
- **Aggregate:** siliceous, carbonate, sand-lightweight or lightweight concrete. Siliceous aggregate needs the most thickness and cover.
- **Required rating:** 1, 1-1/2, 2, 3 or 4 hours, from IBC Tables 601 and 602 and Chapter 7 for the element.
- **Restrained (slabs and beams):** off by default, the conservative case. Whether an assembly is restrained against thermal expansion is your judgment by ASTM E119 Appendix X3.
- **Prestressed (slabs and beams):** selects Table 722.2.3(2) or 722.2.3(4).
- **Equivalent thickness (walls and slabs):** the actual thickness of a solid member with flat faces. Hollow-core, ribbed, undulating or tapered members: enter the equivalent thickness by 722.2.1.1.1 to 722.2.1.1.4 or 722.2.2.1.1 to 722.2.2.1.3.
- **Interpolate between ratings:** off by default, so the rating is the highest tabulated one met. IBC 722.2 does not state interpolation for concrete; turn it on if ACI 216.1 permits it for your case.
- **Cover:** to the positive moment reinforcement (slabs, beams) or the longitudinal bars (columns). For several bars in a beam, enter the average cover with the corner bars at half (Table 722.2.3(3) note c, 722.2.3.3.1).
- **Width and depth:** the beam width (the depth is drawn only) or the column's two dimensions; 36 in. or more on two parallel sides permits notes a and b of Table 722.2.4.
- **Restrained beams spaced 4 ft or less:** note a of Tables 722.2.3(3) and (4), 3/4 in. cover for 4 hours or less.
- **f'c (columns):** above 12,000 psi the least dimension is 24 in.
- **Wall finishes:** gypsum wallboard or plaster on either side, the fire side (both, or side 1 only for an exterior wall more than 5 ft from the lot line), and whether the wall is load-bearing.

Auto-design varies the wall or slab thickness, the cover and the beam width or column size. Objective: the thinnest member, then the least cover. The aggregate, restraint, prestressing, finishes and the required rating are kept.

Checks performed

Check	Basis
Wall rating	Table 722.2.1.1; finishes by 722.2.1.4.1 (Table 722.2.1.4(1) factor × thickness away from the fire) and 722.2.1.4.2 (Table 722.2.1.4(2) time on the fire side), both sides (722.2.1.4.3)
Concrete alone at least half the rating; finish away from the fire at most half the concrete's contribution (load-bearing walls)	722.2.1.4.4
Slab rating	Table 722.2.2.1
Slab cover	Table 722.2.3(1) (reinforced) or 722.2.3(2) (prestressed)
Beam cover	Table 722.2.3(3) (reinforced, interpolated by width, note b) or 722.2.3(4) (prestressed, 8 to 12 in. interpolated); note a for restrained beams spaced 4 ft or less
Column least dimension	Table 722.2.4 with notes a and b; 24 in. above f′c = 12,000 psi (722.2.4.1.2)
Column cover	min(1 in. per hour, 2 in.) (722.2.4.2)

A table entry printed as a dash (an unrestrained 5 in. beam at 3 or 4 hours, an 8 in. unrestrained prestressed beam at 4 hours) is reported as not available for that rating. The drawing is a section through the element with its thickness or size and its cover; the bars are schematic.

Limitations

- Multiwythe and foam-plastic sandwich walls (Figure 722.2.1.2, Eq. 7-4), joints between precast panels (722.2.1.3), multicourse floors and roofs (Figures 722.2.2.2, 722.2.2.3), Table 722.2.3(5) (prestressed beams by area), columns built into walls (722.2.4.4) and the ties of high-strength columns (722.2.4.3) are not covered.
- The analytical methods of ACI 216.1 (heat transfer, moment redistribution in continuous members) are not covered: the prescriptive tables only.
- A tested and listed assembly (IBC 703.2) may be used instead of this calculation.

Verification examples

Verification status: 15 values compared in 11 examples (8 published, 3 hand calculations from the cited equations): 15 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values are rows of IBC 2021 Tables 722.2.1.1, 722.2.2.1, 722.2.3(1) to (4) and 722.2.4: a member at a tabulated thickness or size gives the tabulated rating or requirement. The interpolation by width (notes b), the wall finishes and the 722.2.1.4.4 limits are **hand calculations** from the same tables; no published worked example is on hand.

CHAPTER 110

Spread footing

A square or rectangular isolated footing under one column, designed to ACI 318-19 (Chapters 13, 22 and 25) with ASCE 7-22 load combinations. The soil checks use the service (ASD) combinations of ASCE 7-22 2.4.1 against the allowable pressure you enter; the concrete is designed for the factored combinations of 2.3.1.

Inputs of the spread footing

- **Footing:** length L (x), width B (y), thickness h, the depth of soil over the footing and the soil and concrete unit weights. The footing and soil weights are dead load.
- **Column:** a concrete column or pedestal (c1 along L, c2 along B, its f'c for bearing on the column base) or a steel column on a base plate (section depth and flange width, plate N × B). The column sits at the footing centre. A round column is entered as the square of equal area (13.2.7.3).
- **Loads at the top of the footing,** per load type, unfactored: P (compression positive), Mx (moves the resultant toward +x, along L), My (toward +y), and shears Vx, Vy at the top of the footing (they add V × h at the base).
- **Soil:** allowable bearing pressure qa, its basis (gross pressure, or net pressure above the original overburden γs Df), the base friction coefficient μ, the allowable passive pressure on the footing face (equivalent fluid, 0 to ignore it), and the factors of safety for sliding, overturning and uplift (1.5 prefilled; with the 0.6D combinations some engineers use 1.0, which is your choice).
- **Concrete and reinforcement:** f'c, fy, λ, clear cover (3 in cast against earth, Table 20.5.1.3.1), bar size and number in each direction, which layer is at the bottom, straight or hooked bar ends, epoxy coating.
- **Code options:** the size effect factor λs, which 13.2.6.2 lets you neglect for isolated footings (prefilled), or apply.

Auto-design (editor toolbar) varies the plan L × B (1/2-ft steps, square when the input is square), thickness (2-in. steps) and the bars each way (size and count). Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost; smallest plan, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Soil bearing pressure	Rigid footing, linear pressure: full contact inside the kern, partial contact (no tension) outside it with the contact area solved for equilibrium; worst 2.4.1 combination against qa (gross or net)
Resultant within the footing, overturning, sliding, uplift	13.2.6.1 with your factors of safety; sliding resistance μP plus passive; an advisory flags partial contact under gravity loads
Effective depth ≥ 6 in	13.3.1.2
Flexure in both directions	Critical section at the column face, or halfway to the base plate edge for steel columns (Table 13.2.7.1); moment of the factored net pressure (soil pressure minus the factored weights) beyond the section; Whitney block, φ by εt (21.2.2)
Minimum steel, tension control, spacing	8.6.1.1 (0.0018 Ag), 8.3.3.1, 8.7.2.2 (2h, 18 in)

Check	Basis
Band steel in rectangular footings	13.3.3.3: γs = 2/(β + 1) of the short-direction bars in the centre band; spacing inside and outside the band
One-way shear in both directions	At d from the critical section (13.2.7.2), Table 22.5.5.1(c) (no shear reinforcement)
Punching	Section at d/2 (22.6.4.1), Table 22.6.5.2, moment transfer by eccentric shear (8.4.4.2)
Bearing	Column on the footing with A2 of the 1:2 frustum and on the column base (22.8.3.2); dowel minimum 0.005 Ag (16.3.4.1) as a note
Development	ℓd (25.4.2.4, Ktr = 0) or ℓdh (25.4.3.1) from the critical section to the bar end

Limitations

- The column is at the footing centre; footings with offset columns, sloped or stepped footings and pedestals are not covered (enter a pedestal's loads at the top of the footing yourself).
- Plain concrete footings (Chapter 14) and seismic foundation provisions (18.13) are not included.
- The punching shear moment is the column moment (the soil pressure gradient inside the section is not credited).
- No reduction of ℓd for excess reinforcement (25.4.10) is taken.
- Not checked against published worked examples yet (the CRSI Design Handbook, PCA Notes and ACI SP-17 are not on file); the verification below is by closed-form hand calculation.

Verification examples

Verification status: 37 values compared in 6 examples (3 published, 3 hand calculations from the cited equations): 32 within tolerance, 5 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published worked examples (backend/tests/test_verify_spread_footing.py): the free StructurePoint design examples that reproduce Wight, *Reinforced Concrete Mechanics and Design*, 7th Ed., Examples 15-2 (square footing) and 15-1 (wall footing, checked as a 10 ft length with the wall as the "column"), both to ACI 318-14. Values that ACI 318-19 changed, or that the example computes by another permitted method, are deviations, explained on the Verification page.

Example	What it checks	Published	ILX Calcs
SP-SF (Wight 15-2)	Mu at the column face	954 kip-ft	954.0
SP-SF	Punching vu / φvc (d = 28 in)	156 / 164 psi	156.2 / 164.3
SP-SF	As,min = 0.0018 b h; bar length available	7.72 in²; 55 in	7.72; 55.0
SP-SF	φBn on the footing / on the column	1070 / 895 kips	1074 / 895
SP-SF	One-way Vu at d (deviation: average d vs the layer's d)	204 kips	200.8
SP-SF	φVc one-way (deviation: ACI 318-14 2√f'c bw d vs 318-19 Table 22.5.5.1(c))	308 kips	165.1
SP-SF	ℓd of #8 (deviation: simplified table vs general Eq. 25.4.2.4(a))	54.8 in	32.9

Example	What it checks	Published	ILX Calcs
SP-STF (Wight 15-1)	Mu at the wall face / one-way Vu at d (10 ft)	134 kip-ft / 78.7 kips	134.4 / 78.7
SP-STF	As,min; steel along the wall; its spacing	2.81 in²; 1.45 in²; 13.84 in	2.81; 1.45; 13.84
SP-STF	Bar length available beyond the wall face	22 in	22.0
SP-STF	φVc one-way (deviation: ACI 318-14 vs 318-19)	96.1 kips	54.9

Under ACI 318-19 both example footings need a few inches more thickness for one-way shear: the 318-19 strength of members without stirrups depends on the flexural reinforcement ratio and is much lower for lightly reinforced footings.

Hand calculations in closed form:

Example	What it checks	Hand	ILX Calcs
SF-1	9 ft × 9 ft × 24 in, D 150 + L 100 kips: service q	3.503 ksf	3.503
SF-1	Mu at the column face / φMn (11 #6)	265.6 / 440.6 kip-ft	265.6 / 440.6
SF-1	One-way Vu / φVc	76.7 / 109.5 kips	76.7 / 109.5
SF-1	Punching vu / φvc	96.0 / 189.7 psi	96.0 / 189.7
SF-1	Bearing φBn; ℓd of #6	1432 kips; 17.08 in	1432; 17.08
SF-2	Partial contact, e > L/6: q max = 2P/(3B(L/2 − e))	5.645 ksf	5.645
SF-3	Biaxial, inside the kern: corner pressure	3.625 ksf	3.625

CHAPTER 111

Pole footing

A pole, post or column embedded in earth, or in a concrete footing in earth, resisting lateral load by soil bearing: the depth of embedment of IBC 2024 Section 1807.3, with the presumptive soil values of Section 1806. The method is allowable stress only, because the Table 1806.2 values are allowable values. Loads combine with ASCE 7-22 2.4.1.

Inputs of the pole footing

- **Restraint at the ground surface:** nonconstrained (1807.3.2.1, Eq. 18-1) or constrained by a rigid floor or slab-on-ground (1807.3.2.2, Eq. 18-2 and 18-3). Whether a floor truly restrains the post is your judgment; the floor or slab itself is not designed here.
- **Installation:** a hole backfilled with concrete, a column embedded in a poured concrete footing, clean sand, or controlled low-strength material (1807.3.3).
- **Footing and post:** shape and size of each, and the provided embedment d. With a concrete or CLSM installation, **Width b for lateral bearing** chooses the footing or the post. The code defines b as the diameter of a round post or footing, or the diagonal of a square one (the program takes the diagonal itself). The California DSA Bulletin 09-06 lists conditions for taking b as the footing; the choice and those conditions are yours. Sand backfill always uses the post, because the sand is part of the soil.
- **Backfill concrete f'c**, **concrete unit weight**, and whether the **footing weight** counts in the bearing check.
- **Soil:** a class of Table 1806.2, or entered values from a geotechnical report.
- **Isolated pole: twice the lateral bearing (1806.3.4).** Off by default. It applies to isolated poles such as flagpoles or signs, and to poles of buildings not adversely affected by a 1/2 in. motion at the ground surface under short-term lateral loads. That judgment is yours; the program never assumes it.
- **15× cap of 1806.3.3:** 15 times the tabular value (prefilled, the usual reading, and the conservative one with the doubling) or 15 times the doubled value.
- **Increase for W or E combinations:** 1.0 prefilled. IBC 1806.1 allows one-third more only with the alternative combinations of IBC 1605.2, which are not computed here; a geotechnical report may state its own increase.
- **Loads** per load type, unfactored: lateral force P at its height h above grade, moment M at grade in the same sense as P·h, vertical load V (downward positive; the weight of the post is yours to enter as D). Wind and seismic act both ways. A lateral force in a gravity load type keeps the sign you enter.
- **Factor of safety, uplift**, and the **code options** on brittle materials (1807.3.1).

Auto-design (editor toolbar) varies the embedment d (1/2-ft steps) and the footing diameter or side (2-in. steps); the post, soil, restraint, installation, loads and increases are kept. Objectives: least volume (footing area × embedment), or shallowest embedment and then least volume.

Checks performed

Check	Basis
Embedment, nonconstrained	Eq. 18-1, d = 0.5A{1 + [1 + 4.36h/A]^½}, A = 2.34P/(S1·b), S1 at one-third of the depth. The sheet shows one pass at the provided depth and the required depth, where the trial and computed depths agree
Embedment, constrained	Eq. 18-2 (P·h) and Eq. 18-3 (moment at grade), $d^2 = 4.25 \cdot Mg/(S3 \cdot b)$, S3 at the depth d, solved the same way
Depth for the pressure	At most 12 ft (definition of d in 1807.3.2.1); a note appears when it governs
Depth increase and its cap	1806.3.3, at most 15 times the tabular value; a note appears when it governs
Vertical bearing	1807.3.2.3 and Table 1806.2, on the footing area (with or without its weight) or on the end of the post for sand and CLSM
Uplift	Footing weight with the D factor of the combination, divided by the factor of safety; side friction not counted
Backfill	1807.3.3: f'c at least 2,000 psi, hole at least 4 in. larger than the post diameter or diagonal
Brittle materials	1807.3.1 item 2: not supported laterally by the post unless bracing limits the deflection

The result also gives the lateral force (or the moment at grade, when constrained) that the provided depth allows.

The drawings (screen, report, DXF and PDF sheet) show the elevation with the post, grade or the floor, the footing or hole, the lateral forces at their heights, the dimensions and the depths where S1 and S3 are taken, and the footing plan. Moments and vertical loads are listed in the sheet notes. Reinforcement and the post section are not drawn, because the calculation does not design them.

Limitations

- The post or pole itself, the footing reinforcement, the transfer of forces into the concrete and the floor or slab at grade are not designed. A note gives the moment at grade for the column calculation. Wood poles must meet the AWPA U1 treatment of 1807.3.1 item 3, which is not checked.
- The IBC method only: no ASABE EP 486.3 and no other approved method (1807.3.2), no deflection calculation, no sloping ground, no layered soil, no water table. Friction under a footing (1806.3.1) is not added to the lateral resistance.
- Eq. 18-1 needs a lateral force: a load combination with a moment at grade and no lateral force is not feasible in the nonconstrained case.
- The 1605.2 alternative combinations are not computed. Local amendments are not applied (the City of Phoenix, for example, limits presumptive values and the ratio of depth to width without a geotechnical report; the California DSA has its own requirements).

Verification examples

Verification status: 26 values compared in 6 examples (6 published, 0 hand calculations from the cited equations): 26 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published worked examples (backend/tests/test_verify_pole_footing.py), all on earlier IBC editions with the same equations. The published forces are the design level, so they are entered as load type L (factor 1.0 in D + L).

PDHonline Course S235 (J. W. Andrew, IBC 2006, pp. 3 to 5): a sign on a 32 in. footing in sandy clay, nonconstrained, twice the lateral bearing, 1,200 lb at 16 ft, 775 lb vertical.

What it checks	Published	ILX Calcs
Trial 8.00 ft: S1, A, computed depth	533 psf, 1.97, 6.93 ft	533.3, 1.974, 6.94
Trial 7.50 ft: S1, A, computed depth	500 psf, 2.10, 7.19 ft	500.0, 2.106, 7.20
Trial 7.31 ft: S1, A, computed depth	487 psf, 2.16, 7.31 ft	487.3, 2.161, 7.31
Required embedment	7.31 ft	7.31
Vertical pressure / allowable	139 / 1,500 psf	138.8 / 1,500
15 × the tabular value	1,500 psf	1,500

CalcBook, Pole Foundation Design (IBC 2021, web page, Steps 1 to 4): a 6 kip vehicle load at 2.25 ft on a 2 ft footing in clay, constrained, no increase.

What it checks	Published	ILX Calcs
Footing weight at 6.6 ft	3.11 kips	3.11
Vertical pressure	0.99 ksf	0.99
S3 at 6.6 ft	0.66 ksf	0.66
Depth from Eq. 18-2 at that trial	6.59 ft	6.59
Required embedment, solved	6.60 ft	6.60

Maplesoft, Maple Flow flagpole footing (IBC 2018, pp. 2 to 3): 30 kips at 3 ft on a 4.5 ft footing, 350 psf/ft, constrained, isolated-pole factor 2. The published values are in Pa and m.

What it checks	Published	ILX Calcs
Moment at grade	9.00×10^4 lb-ft	90,000
Embedment d	1.509 m	1.509 m (4.952 ft)
S3	1.660×10^5 Pa	3,466 psf (1.660×10^5 Pa)
S1	5.532×10^4 Pa	1,155 psf (5.532×10^4 Pa)
A	13.501	13.501

Connecticut DAS, Soils and Foundations (2016 OEDM Career Development, IBC 2012, p. 20): a 12 in. utility pole with 9 ft of embedment in sand, 200 lb at 40 ft. The slide stops after one pass of the equation.

What it checks	Published	ILX Calcs
S1 at the 9 ft embedment	450 psf (in the step for A)	450
A	1.04	1.040
Computed depth, one pass	7.3 ft	7.27

Hand calculations of the 12 ft limit, the 15× cap, the 2× factor, square diagonals, uplift and Metric input are in backend/tests/test_pole_footing.py. The text of IBC 2024 Sections 1807.3.2.1, 1806.3.3 and 1806.3.4 was read in the 2025 California Building Code, which is based on it. Equations 18-2 and 18-3, Section 1806.1 and Table 1806.2 were read only in the secondary sources above and in earlier editions; see the project's documents-needed list.

CHAPTER 112

Wall footing

A continuous reinforced footing under a wall, designed per foot of wall to ACI 318-19 (Chapters 7, 13, 22, 24 and 25) with ASCE 7-22 load combinations. The soil checks use the service (ASD) combinations of ASCE 7-22 2.4.1 against the allowable pressure you enter; the concrete is designed for the factored combinations of 2.3.1. ACI 318 has no allowable-stress method for concrete, so the type has no LRFD / ASD switch.

Inputs of the wall footing

- **Wall:** concrete, masonry or steel studs on a sill plate. The choice sets the critical section for moment (Table 13.2.7.1): the face of a concrete wall; halfway between the center and the face of a masonry wall. The table has no row for studs, so the program uses the row of a column on a steel base plate, halfway between the stud face and the sill edge, and says so in a note. Choose another wall type if you judge otherwise. Also the wall thickness and its offset e from the footing center, positive toward +x.
- **Footing:** width B, thickness h, the depth of soil over the footing, the soil and concrete unit weights, a surcharge on the soil with its load type (D or L), and whether the soil weight counts over the wall width too. Off: the soil and surcharge act beside the wall only, because the wall weight is part of your D. On: over the full width, as many hand calculations do.
- **Loads per foot of wall at the top of the footing,** per load type, unfactored: P (compression positive, at the wall centerline), M (moves the resultant toward +x) and V (at the top of the footing, toward +x; it adds V × h at the base). Wind and seismic loads act in both directions.
- **Soil:** allowable bearing pressure qa and its basis (gross pressure, or net pressure above the original overburden γs Df), base friction μ, allowable passive pressure on the footing face (equivalent fluid, 0 to ignore it), and the factors of safety for sliding, overturning and uplift (1.5 prefilled; with the 0.6D combinations some engineers use 1.0, which is your choice).
- **Concrete and reinforcement:** f'c, fy, λ, the exposure and clear cover (3 in cast against the ground, Table 20.5.1.3.1), transverse bars and their spacing, longitudinal bars and their number across B, which layer is at the bottom, straight or hooked ends of the transverse bars, epoxy coating.
- **Code options:** the size effect factor λs, which 13.2.6.2 lets you neglect for one-way shallow foundations (prefilled); the crack control spacing of Table 24.3.2, which 13.3.2.1 and 7.7.2.2 reach for a strip footing and is checked as written unless you decide otherwise (with 3 in of cover and Grade 60 bars it limits the spacing to 7.5 in); and the reduction of ℓd for excess reinforcement (25.4.10, straight bars only), where As,required is the larger of the steel for the moment and As,min.

Auto-design (editor toolbar) varies the width B (1/2-ft steps), thickness h (1-in. steps), the transverse bars (size and spacing) and the longitudinal bars (size and count). Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost; narrowest footing, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Soil bearing pressure	Rigid footing, linear pressure: full contact inside the kern, partial contact (no tension) outside it; worst 2.4.1 combination against qa (gross or net); an advisory flags partial contact under gravity loads
Resultant within the footing, overturning, sliding, uplift	13.2.6.1 with your factors of safety. Overturning about either edge: the applied M and V h (and net uplift) against the moment of the wall load and weights. Sliding resistance μP plus passive
Effective depth ≥ 6 in	13.3.1.2
Cover	Table 20.5.1.3.1 for the exposure you choose
Flexure of the transverse bars	Both overhangs, critical section of Table 13.2.7.1, moment of the forces on that side (13.2.6.6): soil pressure minus the factored footing, soil and surcharge weights over it; Whitney block, φ by εt (21.2.2)
Minimum steel, tension control, spacing	7.6.1.1 through 13.3.2.1 (0.0018 Ag), 7.3.3.1, 7.7.2.3 (3h, 18 in), clear spacing of 25.2.1 (1 in, db; the aggregate limit is yours to confirm)
Crack control spacing (option)	7.7.2.2, Table 24.3.2 with fs = 2/3 fy (24.3.2.1)
Top tension of an overhang	Where an overhang loses soil contact the weights bend it upward and there are no top bars: plain concrete, Mn = 5λ√f'c Sm, φ = 0.60, h − 2 in (14.5.2.1, 14.5.1.7)
One-way shear	At d from the critical section for moment (13.2.7.2, 7.4.3.2), both sides, Table 22.5.5.1(c) (no shear reinforcement) with ρw of the transverse bars, λs as chosen
Development of the transverse bars	ℓd (25.4.2.4, Ktr = 0) or ℓdh (25.4.3.1), from the critical section to the bar end on the shorter side (13.2.8.3); optional reduction of 25.4.10
Longitudinal steel	24.4.3.2 (0.0018 Ag over B h), spacing 24.4.3.3 (5h, 18 in)
Bearing of the wall on the footing	22.8.3.2 with A2 limited by the footing width; Bu = Pu + 6 Mu / t, the extreme-fiber stress of the wall base

Moments and shears in the check tables are for a 12 in strip of wall. The key result Mu is per unit length of wall.

Limitations

- No top reinforcement: where top tension fails its plain-concrete check, the program tells you and you design the top bars by hand or change the footing. Plain concrete footings, stepped or sloped footings and thickened sections are not covered.
- The wall load acts at its centerline and the footing is free to rotate, so the moment of an offset wall goes to the soil. If a floor or a slab restrains the wall, enter what you judge it takes as a load moment.
- Wall dowels, wall reinforcement, splices of the longitudinal bars and the wall's own bearing are not designed.
- Seismic foundation provisions (18.13) are not included. The 0.5L permitted in ASCE 7-22 2.3.1 exception 1 is not applied: 1.0L is used.
- Concentrated loads along the wall and longitudinal bending of the footing (differential settlement, discontinuous walls) are not analyzed; the design is per foot of a uniformly loaded wall.

Verification examples

Verification status: 78 values compared in 13 examples (7 published, 6 hand calculations from the cited equations): 63 within tolerance, 15 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published worked examples (backend/tests/test_verify_wall_footing.py). Most are to an earlier ACI edition or use a simplified formula; where ACI 318-19 or the method differs the value is a deviation, explained on the Verification page.

Example	What it checks	Published	ILX Calcs
StructurePoint strip footing (Wight 15-1, ACI 318-14), 13 in	Factored pressure; d	6.19 ksf; 9.75 in	6.19; 9.75
same	Vu at d; Mu at the face	7.87 kips; 13.4 kip-ft per ft	7.87; 13.44
same	As required; As,min; bar length	0.316, 0.281 in²; 22 in	0.316, 0.281; 22.0
same	Steel along the wall and its spacing	1.45 in²; 13.84 in	1.45; 13.84
same	φVc (deviation: ACI 318-14 vs 318-19)	9.61 kips	5.50
same	ℓd of #4 (deviation: simplified table vs general equation)	21.9 in	13.1
same, 12 in trial	Net pressure; required width; Vu at d	4.37 ksf; 5.15 ft; 8.38 kips	4.37; 5.15; 8.39
UET Peshawar Example 9.1	Vu; Mu; As; As,min; steel along the wall	8.38; 13.433; 0.35; 0.26; 1.34	8.39; 13.44; 0.353; 0.259; 1.34
same	φVc (deviation: 2√f'c used without stirrups)	9.32 kips	5.67
ASDIP Foundation (offset wall, wind)	Max bearing; e / B	2.9 ksf; 0.16	2.93; 0.161
same	Overturning and sliding factors of safety	2.73; 2.93	2.74; 2.93
same, 1.2D + 1.6L	Mu each side; φMn	1.7, 1.5; 11.7 kip-ft per ft	1.70, 1.47; 11.70
same, 1.2D + 1.6L	Vu, − side	1.1 kips	1.05
same	Vu, + side; φVc (deviation: not reproduced to the printed decimal)	0.9; 5.0 kips	0.80; 4.93
same	Plain concrete top φMn (deviation: φ = 0.55 and full h vs φ = 0.60 and h − 2 in)	3.6 kip-ft per ft	2.74
same	ℓd with the reduction (deviation: As,required)	12.0 in	13.7
CE-Ref Example 10 (ACI 318-05, masonry)	Net pressure; factored pressure; d	2.308, 3.04 ksf; 6.75 in	2.308, 3.04; 6.75
same	Mu, vu (deviation: critical sections not per Table 13.2.7.1)	1.27 kip-ft per ft; 40.5 psi	1.78; 19.5
same	φvc (deviation: 2,500 psi and older formula)	75 psi	38.8
same	Steel along the wall (deviation: 0.002 on 8.75 in)	0.525 in²	0.54

Hand calculations of the same file cover the default design, the masonry critical section, the closed-form partial contact pressure, the size effect, the stud case and the stability of a wind moment. Under ACI 318-19 the StructurePoint and UET footings fail one-way shear with the thickness those older methods accepted: the 318-19 strength of a member without stirrups depends on the flexural steel and is much lower for a lightly reinforced footing.

CHAPTER 113

Combined footing

A rectangular footing under two columns, designed to ACI 318-19 (13.3.4 and Chapter 8) with ASCE 7-22 loads. The footing is analysed as a beam along its length under the factored net soil pressure and the column loads; the columns' transverse beams, punching and bearing are checked at each column.

Inputs of the combined footing

- **Footing:** length L, width B, thickness h, soil cover and unit weights. For a uniform soil pressure under D + L the length should be twice the distance from the left end to the resultant of the column loads; the calculation suggests that length when yours differs.
- **Columns:** the centre of each column from the left end (x1 < x2) and its size (c1 along L, c2 across), and the columns' concrete f'c. The columns are on the footing's centreline.
- **Column loads** per column and load type, unfactored: P and the moment M in the long direction (positive moves the resultant toward the right end).
- **Soil:** allowable pressure and its basis (gross or net).
- **Reinforcement:** top and bottom longitudinal bars (size and number across B), transverse bottom bars (size and spacing), which bottom layer is outermost, covers; λs option (13.2.6.2 lets you neglect it).

Auto-design (editor toolbar) varies the width (1/2-ft steps), thickness (2-in. steps), longitudinal bottom and top bars and transverse bars; the length when unlocked (column positions kept). Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Kept unless you select them: length L. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Soil bearing, resultant within the footing, net uplift	Linear pressure along L (uniform, trapezoidal or triangular), 2.4.1 combinations, 13.3.1.1 and 13.3.4.3
Shear and moment along L	Exact for the linear net pressure; diagrams of the combination governing the negative moment
Top steel (negative moment) and bottom steel (positive moment at the column faces and overhangs)	22.2 with φ by εt; As,min 8.6.1.1; tension control; spacing 8.7.2.2
One-way shear along L	At d from the column faces (13.2.7.2), Table 22.5.5.1(c)
Transverse beams under each column	Width c1 + d/2 on each side (limited by the footing end), Pu/B over the cantilever from the column face; flexure and shear
Punching at each column	22.6 with the edge (three-sided) section where the column is close to the end; moment transfer 8.4.4.2
Bearing and dowels	22.8.3.2 on the footing and on each column base; 16.3.4.1 note

Check	Basis
Minimum transverse steel, development	8.6.1.1 per foot; 25.4.2.4 for the bottom, top (ψt = 1.3) and transverse bars

The drawings (screen, report, DXF and PDF sheet) show the footing plan with the two columns (marks C1, C2) and the bottom bars both ways, section A along L with the top and bottom longitudinal bars, the transverse bars at their spacing and the soil cover, the factored shear and moment diagrams along L under it, and section B across the footing at column 2. Bars are drawn over the full length as designed; cut-offs, hooks, splices and column dowels are not part of the calculation and the sheet notes say so.

Limitations

- Rectangular footings only (no trapezoidal or strap footings); columns on the centreline; moments across the width are not entered.
- The transverse-beam width (c1 + d/2 each side) is common textbook practice, not a code rule; confirm it against your practice.
- Verified by hand calculation only; no published combined-footing example is on file.

Verification examples

Verification status: 12 values compared in 3 examples (2 published, 1 hand calculations from the cited equations): 11 within tolerance, 1 recorded deviations. **Thin: only 2 published examples.** Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

Published worked example (backend/tests/test_verify_combined_footing.py): the free StructurePoint example that reproduces Wight, *Reinforced Concrete Mechanics and Design*, 7th Ed., Example 15-5 (ACI 318-14): 25 ft 4 in × 8 ft, an exterior 16 × 24 in column at the property line and an interior 24 in column 20 ft away.

Example	What it checks	Published	ILX Calcs
SP-CF, h = 36 in	Punching vu, interior column / φvc	80.2 / 164.3 psi	80.2 / 164.3
SP-CF, h = 36 in	Punching vu, exterior column (three-sided, γv Munb about the section centroid)	192 psi	191.7
SP-CF	Negative moment between the columns	2100 kip-ft (diagram)	2112
SP-CF, h = 40 in	As,min = 0.0018 b h	6.91 in²	6.91
SP-CF, h = 40 in	Punching vu, exterior column (deviation: the column sides are within d of the long edges, so the program also checks the section open to them, which governs)	157 psi	242.6

This example found an error, now fixed: the moment transferred by eccentric shear at an edge column was taken as Vu times the eccentricity of the column; it is now the moment of the column load and of the soil reaction inside the section about the section's centroid (ACI 318-19 R8.4.4.2.3), which raised vu at the exterior column from 184 to 192 psi.

Hand calculation of the default design: 21.5 ft × 7 ft × 36 in, columns at 1 ft and 17 ft.

What it checks	Hand	ILX Calcs
Service soil pressure (D + L)	3.296 ksf	3.296
Largest negative moment, M at V = 0	692.5 kip-ft	692.5
φMn of 11 #8 top bars	1274 kip-ft	1274
One-way Vu at d from column 2 / φVc	136.9 / 149.8 kips	136.9 / 149.8
Transverse moment under column 2	195.0 kip-ft	195.0
Length for uniform pressure (suggestion)	21.51 ft	21.51

CHAPTER 114

Strap footing

An exterior column at a property line, on an eccentric footing, tied by a strap beam to an interior footing. The strap holds the exterior footing level, so its reaction acts at its center: more than the column load, by what the interior footing is relieved. Use it where a combined footing would be too long.

ACI 318-19 has no section on straps. The strap is a beam by Chapter 9, each footing is by Chapter 13, and the rigid body statics are the method in the textbooks.

Inputs of the strap footing

- **Column spacing, footing sizes and offsets.** Column 1 is the exterior one. **e1** runs from column 1 to the center of the exterior footing, toward column 2. **e2** runs from column 2 to the center of the interior footing, usually 0.
- **Strap width and depth.** The strap is drawn with its top at the top of the footings.
- **Columns:** sizes along the strap and across, and their concrete strength.
- **Column loads:** per column and load type, unfactored, at the top of the footing. **M** acts along the strap, and positive moves the resultant toward the interior column. **M across** acts out of the plane of the strap, and tilts the pressure across the footing.
- **Soil:** cover, unit weights, allowable pressure as gross or net.
- **Reinforcement:** strap bars, hooked or straight at their far ends, stirrups, and the bars of each footing both ways.
- **Size effect:** neglected for the footings by default (13.2.6.2). The strap uses Table 22.5.5.1 with stirrups.

Auto-design varies the footing widths (1/2-ft steps), thicknesses and strap depth (2-in. steps), the strap bars, the stirrup spacing and the footing bars. The column spacing, footing lengths and offsets are kept: they set where the footing sits at the property line.

Checks performed

Check	Basis
Reactions	Rigid body statics, about the center of each footing, with the weights of footings, soil and strap
Uplift, resultant within the footing	13.2.6.1 at every ASD combination
Soil bearing, each footing	13.3.1.1, uniform along the strap, with the moment across it
Strap, top and bottom flexure	22.2, 9.6.1.2. Moments along the whole length under the factored column loads, the net reactions and the strap's own weight
Strap, shear with stirrups, section limit, stirrup spacing, minimum stirrups	22.5, 9.6.3, 9.7.6.2.2. At d from the column faces
Exterior footing, flexure and one-way shear across the strap	13.3.3, 13.2.7.2. A cantilever from the strap face

Check	Basis
Exterior footing, bars along the strap	7.6.1.1, 7.7.2.3. Minimum and spacing
Interior footing, flexure and one-way shear both ways	13.3.3, 13.2.7.2. From the column faces
Interior footing, two-way shear	22.6, 8.4.4.2. The section takes the soil reaction outside it
Bearing of the columns	22.8, without the frustum increase
Development of strap and footing bars	25.4
Strap stiffer than the footing	Advisory: the strap's I over the exterior footing's I above 2, a textbook practice and not ACI

Limitations

- The rigid strap is the model. It takes no soil pressure, and each footing's reaction stays at its center along the strap. Whether your strap is rigid enough, and how to keep it from bearing on the soil, is your judgment; the advisory gives the usual ratio.
- Column 1 sits at the end of the strap, over the exterior footing. Its load goes to the strap, so it has no two-way shear check here. A column much wider than the strap needs your own check of that load path.
- At column 2 the two-way section takes the soil outside it, by equilibrium, because the strap carries its force to the column region. This is a reasoned reading, not a code clause: **check it if a reference disagrees.**
- The strap section is used alone inside the footings for moment and shear, which is conservative where the footing would help.
- Lateral forces on the footings, sliding and transverse beams at the exterior footing are not covered. Enter shears at the top of the footing as moments if you need their effect.
- Dowels are stated, not designed.

Verification examples

Verification status: 4 values compared in 1 example (1 published, 0 hand calculations from the cited equations): 0 within tolerance, 4 recorded deviations. **Thin: only 1 published example. None of the compared values is within tolerance;** the reasons are below. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

The textbook examples (Bowles, *Foundation Analysis and Design*; Wight, *Reinforced Concrete: Mechanics and Design*) were not on hand. One free example was found, and it is only a **partial** check:

Example	Source	Values
Exterior column at a property line, service pressures	ASDIP, *Strap Footing Example Using ASDIP FOUNDATION* (2024), a software output read from its screenshots	Pressures 3.2 / 2.3 and 3.5 / 2.5 ksf. Differences documented below. The spread across the footing agrees

The post does not state the soil weight and shows the water table at the surface, so the weights are not known and the pressures differ, by up to 6 %. Those are documented differences, not fitted. The split of its live, roof and wind loads is not stated either, so its factored strap diagrams cannot be run.

Everything else is a hand calculation in the tests: the reactions, the strap moment under the footing, the footing moments and shears, the moment across the strap, uplift, and the net bearing. **A published, complete strap example is still needed.**

CHAPTER 115

Mat foundation

A rectangular mat under many columns, analysed by the conventional rigid method (ACI 336.2R approach) and designed to ACI 318-19 (13.3.4, Chapter 8, 22.6) with ASCE 7-22 loads. The strip design is approximate, as the method is; the chapter says where it stops.

Inputs of the mat foundation

- **Mat:** Lx, Ly, thickness h, soil cover and unit weights.
- **Columns:** each column's centre (from the mat's lower-left corner), size and unfactored gravity loads (D, L, Lr, S). **Overturning moments** of the wind or seismic system at the mat base (W, E; both directions are checked).
- **Column line tolerance:** columns closer than this in one coordinate form one column line (one strip).
- **Soil:** allowable pressure and basis; the modulus of subgrade reaction ks for the rigidity check only (0 skips it).
- **Reinforcement:** bottom and top bars each way (size, spacing), covers, λs option (13.2.6.2 lets you neglect it).

Auto-design (editor toolbar) varies the thickness (2-in. steps) and the bottom and top mats (bar size and spacing, each way). Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Full contact	Planar pressure q = P/A + P ex x/Iy + P ey y/Ix; a corner in tension under any combination is not feasible (outside the rigid method)
Soil bearing	Largest corner pressure under the 2.4.1 combinations against qa; the largest pressure under a column is listed
Rigidity (advisory)	ACI 336.2R: column spacing ≤ 1.75/λ, λ = (ks b / (4 Ec I))^¼
Punching at every column	Most critical admissible section (interior, edge or corner where a face is within d of an edge), Table 22.6.5.2, the shear's eccentricity about the section centroid (8.4.4.2)
Strip flexure, top and bottom, both directions	Strips along the column lines, bounded midway between lines; column loads and soil reaction adjusted to their average; moment diagram from both ends blended linearly; per foot of strip width; As,min 8.6.1.1 (13.3.4.4), tension control, spacing 8.7.2.2
Strip one-way shear	At d from the column faces, per foot, Table 22.5.5.1(c)

The drawings (screen, report, DXF and PDF sheet) show the mat plan with the columns, the column lines and their dimensions, section A along x through the column line of the strip with the largest moment (columns, top and bottom bars each way at their spacing, covers, soil cover) and that strip's factored moment diagram under it. The smallest sheet scale is 1/16" = 1'-0" (1:200): a mat longer than about 55 ft (17 m) needs the 11 × 17 sheet. Cut-offs, splices, added column-strip bars and dowels are not part of the calculation and are not drawn.

Limitations

- Rigid method only: partial contact, flexible mats (plate on springs) and irregular mats need a plate analysis.
- Strip moments are spread uniformly over the strip width; the concentration at the columns is not modelled. Provide continuous top and bottom bars both ways (R13.3.4.4).
- Column moments are not entered; the edge and corner punching checks use the shear's eccentricity (conservatively added to any moment).
- The edge rule for punching (an edge or corner section where a column face is within d of the edge) and the rigidity criterion follow common practice (ACI 336.2R, ACI 421.1R); they are marked for verification against those documents.

Verification examples

Verification status: 11 values compared in 2 examples (1 published, 1 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

Published worked example (backend/tests/test_verify_mat_foundation.py): Das, *Principles of Foundation Engineering*, mat by the conventional rigid method (16.5 m × 21.5 m, 12 columns, 11,000 kN, resultant 0.44 m and 0.1 m off the centre), as reproduced in free university lecture notes. The published pressures are those of the column loads (the mat's weight is uniform and is subtracted for the comparison).

Point	Published	ILX Calcs
A / B / C (top edge)	36.81 / 31.86 / 26.91 kN/m²	36.75 / 31.83 / 26.91
D / E / F (bottom edge)	25.19 / 30.16 / 35.09 kN/m²	25.26 / 30.18 / 35.10

Das's strip moments are not compared: they are taken at the column centrelines under a uniform average pressure, the program takes them at the column faces under the planar pressure. No free published example of the punching or strip design of a mat to ACI 318-19 was found (ACI 336.2R and the CRSI Design Handbook would provide one).

Hand calculation of the default design: 54 ft × 54 ft × 42 in, 3 × 3 columns at 24 ft.

What it checks	Hand	ILX Calcs
Service soil pressure (D + L)	1.698 ksf	1.698
Punching vu, interior column	97.3 psi	97.3
Punching vu, corner column (two-sided section, moment about the section centroid)	116.9 psi	116.9
φvc	189.7 psi	189.7
Interior x-strip, negative moment per foot	107.1 kip-ft	107.0

The strip moment differs by 0.05 % because the diagram is sampled at 100 stations.

CHAPTER 116

Pile cap

A rigid pile cap on 2 to 9 piles in a standard pattern, or on any layout by coordinates, designed to ACI 318-19 (13.4.6, Chapters 22 and 25) with ASCE 7-22 loads. Pile reactions follow the rigid-cap group formula.

Inputs of the pile cap

- **Piles:** the pattern (2 in line, 3 triangle, 4 square, 5 square plus centre, 6 two rows of 3, 7 hexagon plus centre, 8 staggered 3-2-3, 9 grid) with spacing s, or entered coordinates from the column centre; the edge distance (pile centre to cap edge); the pile diameter and shape; the allowable compression, tension (0 = none allowed) and lateral load per pile, from the pile design.
- **Cap:** thickness h, the distance from the cap bottom to the bottom bars (pile embedment plus the clearance above the pile tops), soil cover and unit weights. The cap is the rectangle around the piles plus the edge distance.
- **Column:** concrete, or steel on a base plate, as for the spread footing; **loads** per load type at the top of the cap (P, Mx, My, Vx, Vy).
- **Reinforcement:** bars each way (size and number), bottom layer, straight or hooked ends, side cover.

Auto-design (editor toolbar) varies the thickness (2-in. steps) and the bottom bars each way (size and count); the pile layout is kept. Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Pile compression, tension, lateral	Rigid cap: R = a + b x + c y with ΣR = P, ΣRx = Mx, ΣRy = My (any layout); 2.4.1 combinations against the allowable values
Effective depth ≥ 12 in	13.4.6.1
Flexure both directions	At Table 13.2.7.1's section; reactions at the pile centroids (13.4.6.2) beyond the section, minus the cap weight
One-way shear both directions	At d; a pile contributes fully when its centre is dpile/2 or more outside the section, nothing dpile/2 or more inside, linearly between (13.4.6.5); λs applies
Punching at the column	Section at d/2 with the 13.4.6.5 shares, Table 22.6.5.2, moment transfer 8.4.4.2
Punching around the most loaded pile	Pile as the square of equal area; corner or edge section to the cap edges; an advisory flags overlapping perimeters (R13.2.7.2)
Deep cap (advisory)	Piles within 2h of the column face: a strut-and-tie check (13.2.6.5, Chapter 23, βs = 0.60λ per 13.4.6.4) is recommended
Minimum steel, spacing, tension control, development, bearing	8.6.1.1, 8.7.2.2, 8.3.3.1 (by analogy), 25.4, 22.8.3.2

The drawings (screen, report, DXF and PDF sheet) show the cap plan with the piles (dashed), the column or steel column on its base plate, the bottom bars both ways, the pile rows and edge distances, and section A along x through the column with the piles below the cap, the clearance to the bottom bars, the bars (hooked ends when chosen) and the soil cover. Pile embedment and reinforcement, column dowels and anchor rods are not part of the calculation and are not drawn.

Limitations

- No strut-and-tie design is performed; for deep caps (most caps on closely spaced piles) the sectional shear checks may be unconservative (R13.4.6.5) and the advisory asks for a strut-and-tie check.
- The cap is rectangular (triangular caps with clipped corners are drawn and weighed as their bounding rectangle).
- Batter piles and the pile design itself are not covered; lateral load is shared equally.
- Two piles in a line cannot resist a moment across the line: that case is not feasible (brace the cap with grade beams).
- The minimum steel and spacing of 8.6.1.1 and 8.7.2.2 are applied to caps by analogy with two-way footings.

Verification examples

Verification status: 17 values compared in 4 examples (3 published, 1 hand calculations from the cited equations): 15 within tolerance, 2 recorded deviations. Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

Published worked example (backend/tests/test_verify_pile_cap.py): the free StructurePoint example that reproduces Fanella, *Reinforced Concrete Structures*, 2nd Ed., Example 10.13 (ACI 318-14): an 8.5 ft × 7.75 ft × 33 in cap on eight 12 in piles under a 22 in column, D = 425, L = 250 kips. The example neglects the cap's own weight; the program includes it, which nearly cancels in the moments and in punching.

What it checks	Published	ILX Calcs
Factored moment, short direction / long direction	584.3 / 541.5 kip-ft	585.4 / 538.4
As,min, short / long direction	6.06 / 5.52 in²	6.06 / 5.52
Punching Vu at the column	684 kips	682.3
φVc at the column (deviation: ACI 318-19 size effect λs = 0.745 applies to pile caps)	947 kips	706.0
One-way Vu east-west, d = 26 in (deviation: cap weight; published share rounded up)	96 kips	93.7

Hand calculation of the default 4-pile cap.

What it checks	Hand	ILX Calcs
Service pile reaction (D + L)	85.06 kips	85.06
Factored moment at the column face	160.9 kip-ft	160.9
Punching vu at the column with the 13.4.6.5 shares	12.7 psi	12.7
Punching vu around a corner pile	56.7 psi	56.7
φvc with λs	141.2 psi	141.2
One-way shear (all piles inside d)	0	0

CHAPTER 117

Slab on grade

Point loads on a plain (unreinforced or shrinkage-reinforced) concrete slab on ground: rack posts on base plates or lift-truck wheels, with an optional uniform storage load beside an aisle. The design follows the approach of ACI 360R and the PCA method (Packard): the working flexural tensile stress from Westergaard's theory of a slab on a dense-liquid (Winkler) subgrade must not exceed the modulus of rupture divided by a factor of safety. Under a rack post the calculation also checks two-way shear and bearing of the plain concrete to ACI 318-19 Chapter 14.

This is a working-stress (allowable stress) design: the loads are unfactored and the factor of safety is yours, so the LRFD / ASD switch is hidden. Only the plain-concrete shear and bearing checks use a load factor, which you enter.

Inputs of the slab on grade

- **Concentrated load:** a rack post on a base plate (plate x along the joint, plate y) or a wheel load (tire contact pressure q, single or dual tires with their centre spacing). P is the working load of one post or one wheel. The contact area is the plate area or P / q; it is treated as a circle of equal area (radius a). Dual tires are replaced by one circle (Huang, *Pavement Analysis and Design*, 2nd Ed., 4.1.3) carrying both loads.
- **Other loads nearby:** the back-to-back post of a double rack, the next posts along the rack, the other wheel of the axle: their positions (x, y from the design load) and loads. They are superposed at the interior position (see Checks).
- **Load positions:** interior (away from joints), edge (at a joint or free edge), corner (joint intersection). Check every position where the load can stand.
- **Joint load transfer:** the reduction of the edge and corner stresses for load transfer across the joint. 0 % is a free edge or an open joint; up to about 25 % is used for doweled joints or tight sawcut joints with aggregate interlock (ACI 360R, UFC 3-320-06). It is never assumed.
- **Edge and corner formulas:** Westergaard's 1948 edge formula for a circular load with the Ioannides et al. (1985) corner formula (the default: both agree with finite-element results), or Westergaard's 1926 edge and corner formulas, which ACI 360R and the PCA papers quote. The interior formula is Westergaard's 1926 formula in both.
- **Joint spacing x, y:** the panel size, for the drawing and for the check that the panel is large enough for the theory.
- **Slab and subgrade:** thickness h, f'c, the modulus of rupture (9√f'c, the PCA value used with its charts; 7.5√f'c, ACI 318-19 19.2.3.1; or a tested flexural strength), Ec (57,000√f'c, ACI 318-19 19.2.2.1, or entered: the PCA and Corps of Engineers charts use 4,000 ksi), Poisson's ratio (0.15 usual; the Corps uses 0.20), the modulus of subgrade reaction k (from plate load tests or the geotechnical report; usual values by soil type in UFC 3-320-06 Table 2-2) and the factor of safety on the modulus of rupture (lift-truck wheels 1.7 to 2.0, rack posts 1.7 to 2.0 in ACI 360R and 2.0 to 5.0 in UFC 3-320-06 Table 3-7; more for unlimited repetitions or impact).
- **Shear and bearing (rack posts):** on or off; the load factor on P (1.6 for storage live load, ASCE 7-22 2.3.1); whether h is reduced by 2 in for shear because the slab is cast against soil (ACI 318-19 14.5.1.7).
- **Uniform storage load:** the load w on both sides of an aisle, the aisle layout (variable: the critical aisle width governs; fixed: the aisle width entered, valid only while the layout does not change) and its factor of safety (2.0 usual, 1.4 the lower limit in UFC 3-320-06 Table 3-7).

Auto-design (editor toolbar) varies the slab thickness (1/2-in. steps); the loads, the base plate, the subgrade, the concrete and the factors of safety are kept. Objective: the thinnest slab. See Auto-design.

Checks performed

Check	Basis
Radius of relative stiffness	ℓ = [Ec h³ / (12 (1 − ν²) k)]^¼ (Westergaard 1926, Eq. 1)
Equivalent radius	b = √(1.6a² + h²) − 0.675h when a < 1.724h, else a (Westergaard 1926, Eq. 8)
Interior stress	σi = 3(1 + ν)P / (2πh²) [ln(ℓ/b) + 0.6159] (Westergaard 1926; ACI 360R form 0.316 P/h² [4 log(ℓ/b) + 1.069])
Loads nearby (interior)	Moments of each nearby load from the infinite-plate solution: Mr = P/(2π)[kei″ + ν kei′/x], Mt = P/(2π)[kei′/x + ν kei″], x = r/ℓ (Hertz, Westergaard 1926 pp. 104-106); added in x, y and shear, and the largest principal stress is checked
Edge stress	Default: Westergaard 1948 circular load, σe = 3(1 + ν)P / (π(3 + ν)h²) [ln(Eh³/(100ka⁴)) + 1.84 − 4ν/3 + (1 − ν)/2 + 1.18(1 + 2ν)a/ℓ] (Ioannides et al. 1985); or 1926: 0.572 P/h² [4 log(ℓ/b) + 0.359]; times (1 − load transfer)
Corner stress	Default: σc = 3P/h² [1 − (c/ℓ)^0.72], c = √(contact area) (Ioannides et al. 1985); or 1926: 3P/h² [1 − (a√2/ℓ)^0.6]; times (1 − load transfer)
Allowable flexural stress	MR / FS (ACI 360R, PCA)
Uniform load beside an aisle	Beam on elastic foundation (Hetényi; Rice 1957, PCA): λ = [k/(4EcI)]^¼, I = h³/12, aisle-centre moment M = w/(2λ²) e^(−λc) sin(λc) (UFC 3-320-06 Eq. 5-5); variable layout at the critical aisle λc = π/4 (2.209ℓ), which is UFC Eq. 5-4, w = 257.9 (MR/FS) √(kh/Ec) psf
Two-way shear under the base plate	ACI 318-19 Table 14.5.5.1(b), (c): Vn = min(4/3 + 8/(3β), 2.66) λ√f'c bo h, perimeter at h/2 (14.4.3.4.1) for the smallest perimeter of the positions checked (interior: four sides; edge: three; corner: two), h − 2 in when chosen (14.5.1.7), φ = 0.60 (Table 21.2.1(i))
Bearing under the base plate	ACI 318-19 Table 14.5.6.1: 0.85 f'c A1 × min(√(A2/A1), 2), A2 of the 1:2 frustum within h, cut by the joints at an edge or corner, φ = 0.60
Panel size (advisory)	Smaller joint spacing at least 3.5ℓ (interior), 5ℓ (edge), 4ℓ (corner) for Westergaard's solutions to hold (Ioannides et al. 1985)
Nearby loads at an edge or corner (advisory)	Shown when a nearby load is within 3ℓ and an edge or corner position is checked: they are not superposed there

Limitations

- Westergaard's theory: an elastic slab of uniform thickness in full contact with a dense-liquid subgrade, large compared with ℓ. Curling and warping stresses, loss of support at joints and settlement are not included.
- Nearby loads are superposed at the interior only. No free published method gives the influence of a nearby load on an edge or corner stress; the edge and corner stresses are those of the design load alone (an advisory says so). ACI 360R and the PCA charts treat the back-to-back post at the interior in the same way.

- The PCA design charts (Packard 1976, ACI 360R Appendix A) are graphs and are not reproduced; the closed-form Westergaard solutions they rest on are. Fatigue is covered by the factor of safety you choose, not by Miner's rule. The PCA "variable layout" table for uniform loads (UFC Table 5-4) is not implemented: the critical aisle width (UFC Eq. 5-4, PCA Table 5-5) governs the variable layout here.
- Reinforced, post-tensioned, shrinkage-compensating and fiber-reinforced slabs, wall and line loads, and the subgrade reaction inside the shear perimeter (neglected, conservative) are not covered.
- The shear and bearing checks are made only for rack posts (tire pressures do not govern).

Verification examples

Verification status: 50 values compared in 28 examples (26 published, 2 hand calculations from the cited equations): 48 within tolerance, 2 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published examples and tables (backend/tests/test_verify_slab_on_grade.py), all from free documents: Westergaard's 1926 paper (HRB Proceedings 5, Tables II to IV and the four-wheel example), Huang's Examples 4.1 to 4.4 as reproduced in a free university lecture, the Pavement Interactive calculation example, the finite-element results of Ioannides, Thompson and Barenberg (1984, AFOSR-TR-85-0083) and UFC 3-320-06 (2025) with the PCA tables it reprints.

Example	What it checks	Published	ILX Calcs
Westergaard 1926, h = 6 in, k = 100 pci, a = 2 / 4 / 6 in	Corner (Table II)	619 / 509 / 420 psi	619 / 509 / 420
same	Interior (Table III)	461 / 395 / 335 psi	461 / 395 / 335
same	Edge (Table IV)	721 / 601 / 493 psi	721 / 601 / 493
Westergaard 1926, h = 10 in, a = 2 / 6 in	Corner; edge	239 / 182; 261 / 212 psi	239 / 182; 261 / 212
Westergaard 1926, four 10-kip wheels at 66 in (h = 7 in, k = 50 pci)	One wheel; two wheels; four wheels	279; 301; 282 psi	279; 301; 282
Huang 4.1, 4.2 (h = 10 in, k = 100 pci, a = 6 in)	ℓ; b; interior; corner (1926)	42.97 in; 5.804 in; 143.7; 186.6 psi	42.97; 5.80; 143.7; 186.6
Huang 4.3	Edge, Westergaard 1948 circle	279.4 psi	279.3
Huang 4.4, duals 14 in apart	a; corner; interior; edge	7.85 in; 166.8; 130.8; 244.2 psi	7.85; 166.8; 130.8; 244.1
Pavement Interactive, h = 9 in, k = 200 pci, 9,000 lb, a = 5.9 in	ℓ; interior; edge; corner (1926)	35.3 in; 150; 226; 193 psi	35.3; 150; 226; 193
Ioannides 1984, finite elements, corner, runs C001, C002, C004, C005, C007	Corner stress	307.5, 270.9, 282.1, 259.9, 159.9 psi	310.0, 271.0, 282.6, 257.8, 160.5
Ioannides 1984, runs C003, C006 (deviation: formula fit)	Corner stress	242.7, 563.6 psi	237.3, 554.9
UFC 3-320-06 Table 5-3 (k = 100 pci, Ec = 4,000 ksi, FS = 2)	Allowable uniform load, h = 7 / 6 / 20 in	1109 / 868 / 2018 psf	1109 / 869 / 2018
UFC 3-320-06 Example B-1.1 (k = 150 pci)	Allowable uniform load	1358 psf	1358

Example	What it checks	Published	ILX Calcs
PCA Table 5-5 (in UFC 3-320-06)	Critical aisle width; allowable load (three rows)	5.4 / 5.6 / 5.6 ft; 950 / 610 / 2065 psf	5.39 / 5.59 / 5.63; 948 / 612 / 2063

The Ioannides corner formula is a fit to finite-element results; two of the seven runs are 1.5 and 2.2 % below them (deviation SOG-IOANNIDES-FIT on the Verification page).

Hand calculations in closed form:

Example	What it checks	Hand	ILX Calcs
SOG-1 (the defaults)	Two-way shear φVn, edge perimeter with h − 2 in	18.17 kips	18.17
SOG-1	Bearing φBn = 0.6 × 0.85 f'c A1 × 2	146.9 kips	146.9
SOG-2	Uniform load beside a fixed 4 ft aisle	1247 psf	1247

The behaviour tests also check the Kelvin-function moments against Westergaard's printed coefficients (Mr/P = −0.0211 and Mt/P = 0.0181 at 1.813ℓ; −0.0186 and 0.0058 at 2.564ℓ). ACI 360R-10 and its worked examples (Appendix A), the PCA IS195 paper and Ringo and Anderson's book are not on file; their charts are graphical and are not used.

CHAPTER 118

Beam on elastic foundation

A reinforced-concrete grade beam (or continuous footing) of finite length resting on the soil, analysed as a beam on a Winkler foundation and designed to ACI 318-19 with ASCE 7-22 loads. The soil is a bed of independent springs: the contact pressure at a point is the modulus of subgrade reaction ks times the settlement there, so a stiff beam spreads its loads and a flexible one concentrates the pressure under them. The calculation gives the settlement, soil pressure, shear and moment along the beam for every load combination, checks the peak service pressure against the allowable bearing pressure and the settlement against your limit, and designs the concrete section for flexure and shear.

Inputs of the beam on elastic foundation

- **Beam:** length L, width b in contact with the soil, depth h, and whether the self-weight is added to the dead load. The beam is free at both ends: the soil alone holds it up (lengths up to 80 ft, 24 m).
- **Loads**, unfactored and per load type (D, L, Lr, S, R, W, E): point loads P (downward positive) with an optional couple M (clockwise positive) at any position x from the left end, and line loads, uniform or linearly varying, over the full length or any part of it (From = To = 0 loads the full length). The ASCE 7-22 2.4.1 combinations are used for the soil pressure and the settlement, 2.3.1 for the concrete; wind and earthquake act in both directions.
- **Modulus of subgrade reaction ks.** It is not a soil constant: it depends on the loaded width and shape and on the soil, so it is your input, with the value the geotechnical engineer gives for this beam. When you only have a 1 ft (0.3 m) plate-load test value ks1, choose "From a 1 ft plate-load test": the calculation converts it to the beam width with Terzaghi's (1955) expressions, ks = ks1 ((B + 1)/2B)² for granular soil and ks = ks1 (1/B)(m + 0.5)/(1.5 m) for cohesive soil (B = b in ft, m = L/B), and shows the steps. Nothing is converted unless you choose it.
- **Soil springs:** compression only (the default: the soil cannot pull the beam down, so the springs that would be in tension are removed and the beam re-analysed until the contact zone settles, per combination) or linear (the classical Heténi solution; any tension in the springs is then reported as a failed contact check).
- **Allowable bearing pressure** (gross, from the geotechnical report) and the **settlement limit** (your limit, commonly 1 in. total for buildings).
- **Beam stiffness:** Ec by ACI 318-19 19.2.2.1(b) (57,000√f'c), 19.2.2.1(a) (wc^1.5 × 33√f'c) or entered; the moment of inertia gross, a fraction of Ig, or entered. A stiffer beam attracts more moment and spreads the pressure; a cracked (softer) beam concentrates the pressure under the loads: Table 6.6.3.1.1(a) gives 0.35Ig for beams at factored loads and 6.6.3.2.2 lets you use 1.4 times that (0.49Ig) for service loads. Consider both; the calculation tells you when the service moment exceeds the cracking moment.
- **Concrete and reinforcement:** f'c, fy, λ, clear covers (3 in. cast against earth at the bottom), top and bottom bars (size and number, continuous over the length), stirrups (size, legs, spacing) or none.

Auto-design (editor toolbar) varies the width (4-in. steps) and depth (2-in. steps), bottom and top bars (size and count), stirrup size, spacing and legs; the length, loads, soil and limits are kept. Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Analysis	Beam finite elements with consistent Winkler springs, nodes at every load, at least 60 elements and λ·le ≤ 0.05; λ = (k/4EI)^¼ with k = ks·b; λL and Hetényi's classification (short and rigid below π/4, long above π) reported
Soil bearing pressure	Largest service pressure ks·y under the 2.4.1 combinations ≤ the allowable (gross)
Contact	Linear springs: no tension in the soil; compression-only springs: the length that lifts off is reported; a net upward load is not feasible
Settlement	Largest service settlement ≤ your limit
Flexure, top and bottom bars	22.2 with φ by εt (Table 21.2.2); εt ≥ 0.004 (9.3.3.1); As,min 9.6.1.2 or 4/3 of the required area (9.6.1.3)
Bar spacing	Clear spacing 25.2.1 (3/4 in. aggregate); crack control 24.3.2 with fs = 2/3 fy
One-way shear	22.5 (Table 22.5.5.1, λs; Vs = Av fyt d/s), section limit 22.5.1.2, Av,min 9.6.3.1, stirrup spacing along and across 9.7.6.2.2; the largest factored shear anywhere along the beam is used (no reduction at d from the loads)
Advisories	Development length of the top and bottom bars from the section of largest moment to the nearer end (25.4.2.4); service moment against the cracking moment (stiffness choice); skin reinforcement when h > 36 in. (9.7.2.3)

The drawings (screen, report, DXF and PDF sheet) show the beam elevation on its soil springs with the loads by type (point loads keyed P1, P2, ... with their values in the sheet notes when there are more than three positions), the load positions and the length, the service soil pressure envelope (with the settlement at the labelled peaks) and the factored shear and moment envelopes under it, and the typical section with the bars, stirrups and cover. Bars are drawn continuous over the length as designed; cut-offs, hooks, splices and column dowels are not part of the calculation and the sheet notes say so.

Limitations

- Winkler model: the springs are independent, so the soil next to a loaded area does not settle with it (no shear coupling, no elastic half-space); the results are only as good as ks. Vary ks (for example half and twice the value) when it is uncertain.
- Constant section and constant ks along the beam; no axial load, no torsion, no supports other than the soil (piles or walls under the beam are not modelled); the length is limited to 80 ft (24 m).
- Shear is checked at its largest value anywhere, conservatively; punching and bearing under the columns are not checked (use the spread or combined footing calculations where they govern).
- The plate-load conversion follows Terzaghi as given in the textbooks (Bowles, Foundation Analysis and Design, 9-6); the original paper is not on file and the value should be confirmed by the geotechnical engineer.

Verification examples

Verification status: 20 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 18 within tolerance, 2 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

backend/tests/test_verify_elastic_foundation_beam.py; the finite elements are also compared with Hetényi's closed forms for the infinite beam (force and couple), the semi-infinite beam and the finite beam with free ends in backend/tests/test_elastic_foundation_beam.py (within 0.1 %).

Example	What it checks	Published	ILX Calcs
Hetényi (1946) §18: 120 in. wood beam, free ends, 5000 lb and a partial uniform load	Deflection at A / C / B	0.03036 / 0.05193 / 0.00628 in.	0.03035 / 0.05197 / 0.00632
Hetényi §18	Foundation pressure at A / C / B	6.07 / 10.39 / 1.26 psi	6.07 / 10.39 / 1.26
Hetényi §18	Moment and shear at the middle C	9623.5 lb-in. / 75.3 lb	9574 / 74.4 (the publication sums interpolated table values)
Hetényi §12, Ex. 1: equal loads spaced π/2λ on a long beam	Moment under a load / (P/4λ): two loads; middle and outer of three	0.7921; 0.5842 and 0.7489	0.792; 0.584 and 0.749
CodeCogs rail example (10 tons per rail, k = 400 lb/in.²)	1/λ; half the depressed length; maximum moment	54.8 in.; 129 in.; 137 ton-in.	54.8; 129.1; 136.9
Structville: 10 m × 0.4 m × 0.6 m concrete beam, 300 kN at 3 m	λ	0.2828 per m	0.2828
Structville	End deflection; moment under the load (deviation: the hand calculation rounds Roark's constants)	12.23 mm; 228.4 kN-m	12.39; 231.0 (exact, as an independent closed-form solution)
Default design (hand)	φMn of 6 #8 top bars; As,min by 9.6.1.2	679.1 kip-ft; 3.96 in.²	679.1; 3.96
Plate-load conversion (hand)	ks from ks1 = 250 pci on sand, B = 3 ft	111.1 pci	111.1

CHAPTER 119

Pile group

The loads on each pile of a group under a rigid cap: vertical and battered piles in any layout, axial load, moments about both axes and horizontal loads, checked against the pile capacities the engineer enters. Two analyses are offered (USACE EM 1110-2-2906, 4-5): the classic rigid-cap formula with axial piles, and the stiffness method of Hrennikoff and Saul (the method of the Corps program CPGA) with lateral pile springs. The cap itself is designed with the pile cap type; the piles' capacities come from the geotechnical and structural pile design (the laterally loaded pile type gives a lateral capacity).

Inputs of the pile group

- **Pile layout:** a rectangular grid (piles along x and y, spacings sx and sy, centred on the load point and numbered row by row from −y, each row from −x), or entered coordinates from the load point. Entered piles may be battered: the batter n (n vertical to 1 horizontal; 0 = vertical) and the plan direction the pile leans toward going down (0° = +x, 90° = +y). The pile diameter or width and shape (drawing, spacing and edge checks).
- **Analysis:**
  - *Rigid cap, axial piles*: all piles alike and acting axially. With vertical piles the reactions follow R = P/n ± Mx·y/Σy² ± My·x/Σx² about the group centroid (solved in general form, so any layout works) and the horizontal load is shared equally. With battered piles the axial-only stiffness solution resolves the horizontal load through the batter (an elastic-center type solution); a load the batter cannot resolve is NOT FEASIBLE.
  - *Rigid cap, elastic piles (stiffness method)*: each pile is an axial spring ka = C33·A·E/L and the lateral springs of a long pile in a soil of constant modulus Es (Hetenyi's semi-infinite beam on springs, as in CPGA: 4EIβ³, 2EIβ², 2EIβ for a head fixed into the cap, 2EIβ³ for a pinned head, β = (Es/4EI)^¼). The 6 × 6 stiffness of the cap gives its displacements and every pile's axial force, shear and head moment. Inputs: pile length L, area A, modulus E, moment of inertia I, the axial stiffness factor C33 (about 1.0 for end bearing to 2.0 for friction piles, EM 1110-2-2906 4-5; a load test is best) and the soil modulus Es (horizontal subgrade modulus times the pile width, constant with depth, reduced by the engineer for close spacing).
- **Loads:** per load type (D, L, Lr, S, R, W, E), unfactored, at the load point: P (down), Mx (adds compression on the +y piles), My (adds compression on the +x piles), Hx, Hy. Applied at the top of the cap (H then adds H·h at the pile heads) or at the pile-head level.
- **Cap:** the edge distance (pile center to cap edge; the cap is the pile group plus the edge distance), thickness h, and optionally the cap and soil-cover weight as dead load.
- **Capacities (per pile):** compression, tension (0 = no tension allowed), lateral (0 = not checked) and, for fixed heads in the stiffness method, the head moment (0 = not checked); the group capacity (block failure, efficiency; 0 = not checked). With ASD (ASCE 7-22 2.4.1) these are allowable loads; with LRFD (2.3.1) factored resistances.

Auto-design (editor toolbar) varies the piles along x and y and their spacing (grid layout; the y spacing keeps its ratio to the x spacing): fewest piles, then the smallest cap. The pile type, capacities, loads and analysis are kept; piles at entered coordinates are kept as entered. See Auto-design.

Checks performed

Check	Basis
Pile compression and tension (uplift)	Largest and smallest axial force of all combinations against the capacities entered
Pile lateral load	Vertical piles, rigid method: $\sqrt{(Hx^2 + Hy^2)}/n$; stiffness method: the shear across each pile
Pile head moment (stiffness method, fixed heads)	Largest head moment against the capacity entered (else reported)
Group capacity	Total vertical load against the group capacity entered (block failure, efficiency)
Pile spacing (advisory)	Closest piles at least 3 diameters (EM 1110-2-2906 4-2g(1))
Long-pile assumption (advisory, stiffness method)	$\beta L \geq 3$ for the semi-infinite lateral springs
Mechanism (NOT FEASIBLE)	One pile, piles in a line with a moment across it, battered piles whose axes meet at one point, horizontal load with no batter in the rigid method

The drawings show the cap plan (piles dashed with their marks, arrows toward the toe of battered piles, pile rows, edge and overall dimensions, the load point) and section A along x with the piles below the cap at their batter. The sheet notes list each pile's largest and smallest axial force by mark. Groups of more than 20 piles mark the first pile of each row.

Limitations

- The cap is rigid; flexible caps and mats on piles are not covered.
- No torsion about the vertical axis is applied, and the piles carry no torsion.
- The lateral springs assume a long pile in a soil of constant modulus; p-y (nonlinear) springs, group reduction factors and pile-soil-pile interaction are not modelled (enter a reduced Es).
- The pile capacities, block failure and group efficiency are the engineer's inputs; nothing is assumed.
- Rigid method with battered piles: piles act axially only (the horizontal load goes to the batter).

Verification examples

Verification status: 18 values compared in 5 examples (5 published, 0 hand calculations from the cited equations): 18 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_pile_group.py):

Source	What it checks	Published	ILX Calcs
UFC 3-220-01A, Eq. 2-2 and Fig. 2-1: 8 piles, ex = 0.8 ft, ey = 3.0 ft, Qg = 100 kips	Largest pile load / tension	34.2 / 9.2 kips	34.17 / 9.17
FHWA GEC 12 Examples (NHI-16-064), Pier 2, 20 piles, Table D-60	Piles 5, 16 and 1	257.3 / 88.3 / 177.7 kips	257.3 / 88.3 / 177.7
GEC 12 Examples, North Abutment, 2 rows of 9, Table D-13	Service I per pile, front / back	229.9 / −25.7 kips	229.9 / −25.7
GEC 12 Examples, North Abutment, Strength I	Front row per pile	323 kips	323.2

Source	What it checks	Published	ILX Calcs
USACE ITL-89-3, CPGA Example Problem 1 (Hrennikoff 1950): 5 timber piles, 3 battered 3:1, fixed heads	Axial forces, piles 1 to 5	37.6, 34.5, 31.3, 10.8, 5.4 kips	37.63, 34.47, 31.31, 10.77, 5.44
same	Shear / head moment (vertical piles)	1.53 kips / 36.0 in-k	1.52 / 36.0
same	Cap displacements Dx, Dz, rotation	−0.1032 in, 0.02787 in, 0.0002095 rad	−0.1032, 0.02787, 0.0002095

In CPGA's sign convention a positive MY adds compression on the −x piles; the example is entered with My = −173.4 kip-ft.

CHAPTER 120

Laterally loaded pile

The ultimate lateral resistance of a single vertical pile by Broms' method (1964a cohesive soil, 1964b cohesionless soil), checked against the service lateral load with the engineer's factor of safety, with an estimate of the deflection at grade. FHWA GEC 9 (2018) 6.5 presents the method and its limits: it suits short, stiff piles with a free head; long or fixed-head piles should be confirmed with a p-y analysis.

Inputs of the laterally loaded pile

- **Soil:** cohesive (undrained shear strength cu; no resistance over the top 1.5D, then 9·cu·D per unit length) or cohesionless (effective unit weight γ' and friction angle φ'; 3·Kp·γ'·z·D per unit length with the Rankine Kp = tan²(45° + φ'/2)). One uniform soil; the values come from the geotechnical report.
- **Pile:** head condition (free, with the load at the height e above grade; or fixed by a cap at grade that prevents rotation: the engineer's judgment of the restraint), embedment L, the section: a steel pipe (D, t, Fy; My = Fy·Z with the plastic modulus Z = (D³ − di³)/6) or an entered section (width D, yield moment My, E, I; for a prestressed concrete pile FHWA HI-97-013 uses My = f'c·S), and the factor of safety on Hu (FHWA HI-97-013 9.7.3.2 uses 2.5; 2 to 3 is common).
- **Loads:** per load type, unfactored: H at the load point and, for a free head, a head moment M (same sense as H·e; it raises the effective load height to e + M/H). The ASCE 7-22 2.4.1 combinations give the service loads.
- **Deflection estimate (optional):** the horizontal subgrade modulus kh (clay, constant with depth; about 67·cu/D per Davisson) or its gradient nh (sand, kh = nh·z/D) and the deflection limit set by the supported structure.

Auto-design (editor toolbar) varies the embedment in 1-ft steps (the shortest that passes). The section, soil, head and factor of safety are kept: a long pile governed by its yield moment needs a stronger section, which the engineer chooses. See Auto-design.

Checks performed

Check	Basis
Lateral load H ≤ Hu / FS	Broms: every mode is computed and the governing one reported
Free head, cohesive	Short: H(e + 1.5D + 0.5f) = 2.25·cu·D·g², f = H/(9cuD), L = 1.5D + f + g; long: Mmax = H(e + 1.5D + 0.5f) = My
Fixed head, cohesive	Short: Hu = 9cuD(L − 1.5D) while Hu(0.5L + 0.75D) ≤ My; intermediate (hinge at the cap); long: Hu = 2My/(1.5D + 0.5f)
Free head, cohesionless	Short: Hu = 0.5γ'DL³Kp/(e + L); long: H(e + 2f/3) = My, f = √(2H/(3γ'DKp))
Fixed head, cohesionless	Short: Hu = 1.5γ'DL²Kp while γ'DKpL³ ≤ My; intermediate: Hu = 0.5γ'DL²Kp + My/L; long: Hu = 3My/f

Check	Basis
Deflection at grade (advisory estimate)	Broms subgrade reaction: clay β = (khD/4EI)^¼, short (βL ≤ 1.5) 4H(1 + 1.5e/L)/(khDL) free, H/(khDL) fixed; long (βL ≥ 2.5) 2Hβ(eβ + 1)/(khD) free, Hβ/(khD) fixed; sand η = (nh/EI)^⅕, short (ηL ≤ 2) 18H(1 + 1.33e/L)/(L²nh) free, 2H/(L²nh) fixed; long (ηL ≥ 4) 2.40H/(nh^⅗(EI)^⅖) free (e = 0 only), 0.93H/(nh^⅗(EI)^⅖) fixed; linear between
Method limits (advisory)	GEC 9 6.5: confirm long or fixed-head piles with a p-y analysis

The intermediate depths follow the exact passive distribution (f = √(2H/(3γ'DKp)), the published 0.82 and 0.544 are its rounded coefficients). The drawings show the pile elevation (grade, soil, load, embedment, the 1.5D zone in clay) and the Broms soil reaction and moment along the pile at Hu, which stop at a plastic hinge.

Limitations

- **p-y analysis is not included.** The free sources on hand (FHWA GEC 9, FHWA HI-97-013, GEC 12, EM 1110-2-2906) describe the p-y method but give no complete set of curves with a worked example the program could reproduce; FHWA-IP-84-11 (Reese 1984) and the COM624P manual (FHWA-SA-91-048), which have them, could not be obtained. Use a p-y program (LPILE, COM624P) for long, fixed-head or important piles.
- One uniform soil, level ground, static loads; no cyclic degradation, scour, layered soil or sloping ground.
- No axial load or P-delta; the pile's structural design and axial capacity are not checked.
- Group effects are not included (enter reduced soil values for close spacing).
- The deflection is an estimate (GEC 9 considers Broms' deflections rough); a free-head long pile in sand with the load above grade has a chart solution only and is not estimated.

Verification examples

Verification status: 15 values compared in 4 examples (4 published, 0 hand calculations from the cited equations): 12 within tolerance, 3 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published worked examples (backend/tests/test_verify_lateral_pile.py):

Source	What it checks	Published	ILX Calcs
Hubbell / Chance Helical Foundation Manual, Step 5, Example 5.1: 6-5/8 in pipe, clay cu = 1,000 psf, free head, e = 1 ft, L = 6.98 ft	Hu (2 × 4,400 lb), f, Mmax	8.8 kips, 1.771 ft, 23.88 kip-ft	8.79, 1.770, 23.85
Kouretzis, Fundamentals of Foundation Engineering (open textbook), Example 6.12: 400 × 5 mm pipe, su = 100 kPa, fixed head, L = 10 m	My; short / intermediate / long Hf	351 kNm; 3384 / 1333.6 / 527.03 kN	351.1; 3384 / 1333.6 / 527.1
same	Deflection at 263.5 kN (deviation: β rounded in the example)	0.019 m	0.0199
Kouretzis, Example 6.13: same pipe, dry sand φ' = 33°, fixed head	Kp; short / intermediate / long Hf	3.39; 3661.2 / 1255.5 / 343.8 kN	3.392; 3663.5 / 1256.3 / 343.8
FHWA HI-97-013 Student Exercise 8: 356 mm prestressed pile, sand φ = 35°, fixed head, L = 11 m	Kp	3.69	3.690

Source	What it checks	Published	ILX Calcs
same	Qu (deviation: read from Broms' chart)	295 kN	280.6
same	Deflection at 118 kN (deviation: read from Broms' chart)	11.0 mm	11.2

No free published example of a free-head pile in sand was found; that case is verified against the closed forms in backend/tests/test_lateral_pile.py.

CHAPTER 121

Retaining wall

A cantilever retaining wall, per foot of wall: earth pressures, stability under service loads, and the stem, heel and toe designed to ACI 318-19 (13.3.6, Chapters 7, 22, 24 and 25).

Inputs of the retaining wall

- **Wall:** stem height above the footing (the backfill is level with the top of the stem), stem thickness at the top and base (the back face is vertical, a thicker base batters the front face), footing thickness, toe and heel lengths, and an optional shear key (depth, width, position).
- **Backfill and soil:** the earth pressure theory (Rankine active with a sloped backfill, Coulomb active with the wall friction δ on the vertical plane through the heel, or at rest K0 = 1 − sin φ for level backfill), φ, β, γ, the water table above the base with the saturated unit weight, a uniform surcharge (dead or live), and the soil over the toe (its weight can be counted or ignored).
- **Loads on the stem top:** dead and live line loads.
- **Stability and bearing:** allowable pressure, base friction μ, the passive resistance factor and the top depth neglected, factors of safety for sliding and overturning (1.5 per IBC 1807.2.3), and whether the resultant must be in the middle third.
- **Concrete and reinforcement:** covers, stem vertical bars (earth face), heel top and toe bottom bars, stem horizontal and footing longitudinal bars; the load factor on H (1.6 prefilled) and λs for the toe and heel.

Auto-design (editor toolbar) varies the heel and toe (1/2-ft steps), stem base and footing thickness (2-in. steps) and every bar mat (size and spacing); the height, key, soil and loads are kept. Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis		
Earth pressure	On the vertical plane through the heel: Rankine Ka (acting parallel to the slope), Coulomb Ka (at δ), or K0; effective unit weight and water pressure below the water table; surcharge K q		
Sliding	(μN + Pp)/ΣH ≥ FS; Rankine passive Kp over the toe soil, footing and key, times the reduction factor, the top depth neglected; a live surcharge over the heel is not counted as resisting		
Overturning	Mr/Mo about the toe ≥ FS; uplift (linear from the heel to zero at the toe) overturns		
Bearing, middle third	Trapezoidal or triangular base pressure under D + H and D + H + L against qa;	e	≤ B/6 when required
Stem flexure and shear	At the stem-footing interface (13.3.6.3) and, for a tapered stem, the most critical of nine levels; λs applies; As,min 7.6.1.1, tension control, spacing 7.7.2.3; crack control spacing Table 24.3.2 at the earth face (fs = 2/3 fy, 7.7.2.2; an option, on by default)		

Check	Basis
Heel and toe	Factored base pressure (1.4D + 1.6H, 1.2D + 1.6H + 1.6L and 0.9D + 1.6H, ASCE 7-22 2.3.1 with H; the lateral pressure of a surcharge is part of H, 2.2): heel top bars and shear at the stem face, toe bottom bars and shear at d; crack control spacing Table 24.3.2 of the heel top and toe bottom bars
Shrinkage and temperature steel	Stem horizontal and footing longitudinal bars, 24.4.3.2 and 24.4.3.3
Development	Stem bars hooked into the footing (25.4.3.1, R13.3.6.3): bars along the wall with ψo = 1.0, and the end bars at a wall end or joint with the side cover entered (ψo = 1.25 under 6db, Table 25.4.3.2); heel bars (top bar factor) and toe bars (25.4.2.4)

The drawings (screen, report, DXF and PDF sheet) show the wall section: stem (battered front face), toe, heel and key, the backfill (level or sloped), the soil over the toe, the water table and the surcharge; the stem vertical bars with their standard hook into the footing, the heel top and toe bottom bars, the stem horizontal bars (one or both faces) and the footing longitudinal bars at their spacings; the service lateral pressure (earth + water + surcharge) on the vertical plane through the heel; and the service bearing pressure under the base, aligned under the section. Bar lengths, cut-offs, splices, the hook direction, key reinforcement, drainage and joints are not part of the calculation and are not drawn.

Limitations

- Seismic earth pressure (Mononobe-Okabe), compaction pressures, cohesive backfill and global (slope) stability are not included; the key's own strength is not designed; counterfort walls are not covered.
- The backfill reaches the top of the stem; at-rest pressure is for level backfill only.
- The uplift distribution (full at the heel, zero at the toe) is an assumption for a drained toe.
- The ASCE 7-22 load factors on H (1.6 adding, and H at 1.0 in the service combinations) are entered as inputs; they are marked for verification against the ASCE 7-22 text, which is not on file. IBC 1807.2.3 (FS 1.5) is quoted from memory and marked the same way.

Verification examples

Verification status: 118 values compared in 7 examples (6 published, 1 hand calculations from the cited equations): 97 within tolerance, 21 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Published worked example (backend/tests/test_verify_retaining_wall.py): the free StructurePoint example (ACI 318-19) that reproduces Darwin, Dolan and Nilson, *Design of Concrete Structures*, 16th Ed., Example 16.8: 13.5 ft tapered stem, 9.75 ft × 18 in footing, key, φ = 30°, 400 psf surcharge, f'c = 4500 psi. The example's stem is battered on the earth side and the program's on the front face, so the weights, the overturning factor of safety, the base pressures and the toe and heel moments are not compared.

What it checks	Published	ILX Calcs
Horizontal thrust, earth + surcharge	6.49 kips/ft	6.50
Overturning moment about the toe	37.46 kip-ft/ft	37.50
Stem Mu at the base	45.7 kip-ft/ft	45.68

What it checks	Published	ILX Calcs
As,min of the toe and heel (0.0018 b h)	0.39 in²/ft	0.389
Stem φVc (deviation: the program neglects the Nu/(6Ag) term, conservative)	10.98 kips/ft	10.67

Hand calculation of the default wall: 12 ft stem, 3 ft toe, 6 ft heel, 24 in key, Rankine, 250 psf live surcharge.

What it checks	Hand	ILX Calcs
Earth pressure ½ Ka γ H'²	3.645 kip/ft	3.645
Overturning ratio Mo·FS/Mr	0.427	0.427
Sliding ratio ΣH·FS/(μN + Pp)	0.812	0.812
Toe pressure N/B (1 + 6e/B)	1.748 ksf	1.748
Stem Mu / Vu at the base	28.03 kip-ft / 6.21 kips	28.03 / 6.21

CHAPTER 122

Basement (restrained) retaining wall

A restrained (basement) retaining wall of concrete, per foot of wall, spanning vertically between the floor diaphragm at the top and the slab on grade or footing at the base: lateral earth, water and surcharge pressure, axial load from above, reactions to the diaphragm and the base, and the wall designed to ACI 318-19 (Chapters 6, 11 and 22).

Inputs of the basement wall

- **Wall:** the span L between the base support and the diaphragm, the retained soil height (up to the span: grade may be below the floor), the thickness h, and the span condition: a propped cantilever (fixed at the footing, pinned at the floor) or a simple span (pinned at both). The real base lies between fixed and pinned; the choice is the engineer's. The wall is analysed as a one-way vertical strip of constant stiffness.
- **Lateral pressure:** an equivalent fluid pressure from the geotechnical report (with a second value below the water table that includes the water), at rest K0 = (1 − sin φ) OCR^sin φ (Jaky; Mayne and Kulhawy for an overconsolidated or compacted backfill, as in FHWA GEC 2 and AASHTO 3.11.5.2), or Rankine / Coulomb active for a wall that can yield (sloped backfill allowed, horizontal component used). A restrained wall does not move enough for active pressure; at rest or the report's value is the usual choice. With K, the water table adds hydrostatic pressure and the soil below it takes its effective unit weight. A uniform surcharge (dead or live) adds K q (with an equivalent fluid pressure, q × EFP / γ: the surcharge as an equivalent height of soil); compaction pressure can be entered the same way (2 ft of soil is common for short walls).
- **Axial load from above:** dead and live line loads at the top and their eccentricity toward the interior face (a moment P e at the top, tension on the exterior face); the wall's own weight is added below the top.
- **Concrete and reinforcement:** f'c, fy, λ, one or two layers of vertical bars (exterior and interior face, or one centered layer), covers, horizontal bars, and whether the wall is a single-story basement wall (the 11.7.2.3 exception to two layers).
- **Code options:** the load factor on H (1.6 prefilled, ASCE 7-22 2.3.1), the effective length factor k and the unsupported height ℓu for slenderness (k = 1.0 is permitted for nonsway members, 6.6.4.4.3), and the shear critical section (at d from the supports where the reaction compresses the member end, or at the supports).

Auto-design (editor toolbar) varies the wall thickness (2-in. steps) and the vertical bars of each face and the horizontal bars (size and spacing); the span, soil, water, loads and span condition are kept. Objectives: lowest cost index (concrete, formwork, reinforcement); least concrete, then cost. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Analysis	Simple-beam moment of the pressure, plus the top end moment P e, plus the base moment from compatibility for the propped cantilever (zero rotation at the fixed base, constant EI); reactions to the diaphragm and the base per load type (service) and factored
Load combinations	ASCE 7-22 2.3.1 with H at its entered factor: 1.4D + H, 1.2D + H + 1.6L, 0.9D + H; earth, water and surcharge pressure are H (2.2), the surcharge pressure present with its own load type

Check	Basis
Flexure with axial load	11.5.2.2 and 22.4: strain compatibility at the factored axial force of the section (wall weight above it included), both layers of bars, φ by Table 21.2.2 (tied); exterior face at the base or top, interior face in the span
Slenderness	6.2.5.1(b) with r = 0.3h and M1/M2 = 0 (34); beyond it the moment is magnified by 6.6.4.5 (Cm = 1.0 with transverse load, EI = 0.4EcIg/(1 + βdns), M2,min); kℓu/r over 100 is not feasible (6.2.6); second-order moment at most 1.4 times the first order (6.2.5.3)
Maximum axial strength	22.4.2.1, Pn,max = 0.80 Po, φ = 0.65
Out-of-plane shear	11.5.5 and 22.5.5.1: Vc = [8λsλ(ρw)^(1/3)√f'c + Nu/(6Ag)]bw d ≤ 5λ√f'c bw d, with the bars in tension at the section and the axial force of the same combination
Reinforcement limits	Table 11.6.1 minimum vertical and horizontal ratios; spacing ≤ min(3h, 18 in.) (11.7.2.1, 11.7.3.1); two layers in walls thicker than 10 in. unless a single-story basement wall (11.7.2.3)

The drawings (screen, report, DXF and PDF sheet) show the wall section with the floor diaphragm and the slab or footing as stubs by others, the backfill grade, the water table, the surcharge, the vertical bars of each face and the horizontal bars at their spacings, and the service lateral pressure on the wall; beside it the factored shear and moment along the height under the combination with the largest moment. The floor, the footing, the slab on grade, dowels, splices, waterproofing and drainage are not part of the calculation.

Limitations

- The footing (bearing, sliding) is not designed: the slab and the footing carry the reactions the calculation reports (see the wall footing and spread footing calculations).
- Seismic earth pressure, two-way action near corners and openings, crack control, deflection and the dowels into the footing are not included. Bar spacing follows Chapter 11 (11.7), which does not call for Table 24.3.2; ACI MNL-17(21) Retaining Walls Example 1 nevertheless checks Table 24.3.2 crack control on its basement wall. Whether to add that check here is an open item for the owner. Compaction pressure is entered as a surcharge or within the equivalent fluid pressure.
- At-rest and equivalent fluid pressures are for level backfill (a sloped backfill raises them by an amount the geotechnical engineer gives); Coulomb's vertical component of the thrust is not added to the axial force (conservative for a tension-controlled wall).
- A masonry (TMS 402) basement wall is not offered: TMS 402-22 is not on file (only TMS 402-13), and its slender wall provisions belong to the masonry wall calculations.

Verification examples

Verification status: 25 values compared in 4 examples (2 published, 2 hand calculations from the cited equations): 21 within tolerance, 4 recorded deviations. **Thin: only 2 published examples.** Code basis: not determined. The code is cited, but no record shows a section-by-section read for this module.

Published worked example (backend/tests/test_verify_basement_wall.py): ACI E702.4, *Buried Concrete Basement Wall Design* (ACI E702 Example Problems, 2013, free from concrete.org): 12 in. wall, fixed base and pinned top, 15 ft between supports, equivalent fluid pressure 60 pcf with 2 ft of soil as surcharge, D = 2.5 and L = 1.5 kip/ft, #6 at 8 in. each face, f'c = 4000 psi. The example is written to ACI 318-11; the analysis values and the factored moment do not depend on the edition.

What it checks	Published	ILX Calcs
Diaphragm reaction (service)	2.02 kip/ft	2.025
Shear at the base (service)	6.53 kip/ft	6.525
Moment at the base (service)	16.9 kip-ft/ft	16.88
Maximum span moment (service)	7.88 kip-ft/ft	7.88
Factored moment at the base, 1.6H	27.0 kip-ft/ft	27.0
Factored shear at the base	10.4 kip/ft	10.44
Design moment strength at Pu (spColumn)	31.6 kip-ft/ft	30.4 (deviation: 0.9D + 1.6H governs with less axial load; at the example's axial load 31.64)

Hand calculations: a simple span under at-rest pressure (reactions W/3 and 2W/3, maximum moment $2WL/(9\sqrt{3})$ and its factored value) and a propped wall under an eccentric top load (the carry-over moment $-Mt/2$ at the fixed base and the reaction $(Mb - Mt)/L$), all within 0.1 %.

CHAPTER 123

Gravity retaining wall

A gravity retaining wall of mass (plain) concrete or stone masonry, per foot of wall: the earth pressure, the stability of the wall on its base, and the strength of the stem at five sections (plain concrete to ACI 318-19 Chapter 14, or stone masonry by the no-tension rule and an allowable stress).

Inputs of the gravity wall

- **Wall:** height H from the base bottom to the top, top width, front batter and back batter (horizontal per vertical; a back batter slopes out under the backfill), an optional base slab (thickness, toe and heel projections), the material (plain concrete with f'c and λ, or stone masonry with the allowable compressive stress of its joints) and its unit weight.
- **Backfill and soil:** the earth pressure theory: Coulomb on the back of the wall (wall friction δ, 2/3 φ is common; the back's inclination θ comes from the batter) or Rankine on the vertical plane through the heel (the soil over the heel and over a sloped back moves with the wall and counts as weight, the thrust acts parallel to the slope); φ, β, γ, a uniform surcharge (dead or live) and the soil depth in front for passive resistance.
- **Stability and bearing:** allowable bearing pressure, base friction coefficient μ, the passive reduction factor and the neglected top depth, the factors of safety for sliding and overturning, the eccentricity limit (B/6 keeps the whole base in compression), and the load factor on H for the concrete sections.

Auto-design (editor toolbar) varies the top width (1/2-ft steps), the front batter and the toe and heel of a base slab; the height, slab thickness, soil and loads are kept. Objective: least wall area. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Earth pressure	Coulomb Ka(φ, δ, β, θ) with the thrust at δ + θ below the horizontal, or Rankine Ka(φ, β) on the vertical plane through the heel; Pa = ½ K γ H² at H/3; surcharge K q H at H/2 (× cos θ / cos(θ − β) for Coulomb with a sloped backfill); drained backfill
Sliding	(μN + Pp) / T ≥ FS; a live surcharge over the wall or heel does not resist
Overturning	Resisting over overturning moment about the toe ≥ FS (the vertical earth thrust resists)
Bearing and eccentricity	Trapezoidal or triangular base pressure under D + H and D + H + L (ASCE 7-22 2.4.1) against the allowable; eccentricity within the entered limit
Plain concrete sections	At five levels of the stem, under 1.4D + H, 1.2D + H + 1.6L and 0.9D + H (2.3.1): tension face Mu/Sm − Pu/Ag ≤ φ5λ√f'c (14.5.3.1), shear Vu ≤ φ(4/3)λ√f'c b h (14.5.5.1), φ = 0.60
Masonry joints	At five levels, service: resultant in the middle third (no tension) and the maximum compressive stress against the allowable

The drawings (screen, report, DXF and PDF sheet) show the wall section (stem and base slab, backfill, soil in front, surcharge, the horizontal earth pressure on the back plane, the height, base and top dimensions) and the service bearing pressure under the base. Drainage, weep holes, joints and any base slab reinforcement are not part of the calculation.

Limitations

- A drained backfill is assumed: no water pressure or uplift. Seismic earth pressure and global stability are not included.
- Coulomb on the back of the wall is not offered with a heel (the soil over the heel moves with the wall: use Rankine).
- The base slab is taken as part of the mass; its bending strength as a cantilever toe or heel is not checked (keep it thick enough, or design it as a cantilever wall footing).
- ACI 318-19 14.5.1.7 (thickness less 2 in. for concrete cast against soil) is not applied to the stem sections.

Verification examples

Verification status: 11 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

No free published gravity-wall example with complete data was available (research/DOCUMENTS_NEEDED.md lists the textbooks and the USACE and NAVFAC manuals). Hand calculations in closed form (backend/tests/test_verify_gravity_wall.py):

What it checks	Hand	ILX Calcs
Rankine Ka, φ = 32°	0.3073	0.3073
Rectangular wall 4 × 8 ft: overturning moment / resisting moment ÷ 2	3.146 / 4.640 kip-ft	3.146 / 4.640
Sliding force / toe pressure	1.180 kip/ft / 2.340 ksf	1.180 / 2.340
Plain concrete tension at the base, 0.9D + 1.6H	5.86 psi	5.86
Coulomb Ka, δ = 21°, vertical back	0.2751	0.2751
Trapezoidal wall: overturning / resisting ÷ 2	6.422 / 8.138 kip-ft	6.422 / 8.138
Sliding force / resistance ÷ 1.5	1.798 / 2.176 kip/ft	1.798 / 2.176

CHAPTER 124

Segmental retaining wall

A conventional (gravity) segmental retaining wall of dry-stacked concrete units, per foot of wall, by the NCMA approach to external stability: the units with their infill as a battered gravity wall, plus the interface shear between units from the manufacturer's tests.

Inputs of the segmental wall

- **Units:** number of courses, unit height and depth, setback per course (the batter ω = atan(setback / unit height)), the in-place unit weight with the infill, the embedment (soil in front of the base) and the leveling pad thickness (drawn).
- **Retained soil:** φ, the wall friction δ on the back of the units (NCMA uses 2/3 φ for a conventional wall), the backfill slope, γ and a uniform surcharge (dead or live: a live surcharge drives the wall but does not resist).
- **Stability:** the allowable bearing pressure, the base friction coefficient μ, the interface shear intercept and angle a_u and λ_u from the manufacturer's ASTM D6916 tests, the passive resistance factor (NCMA neglects it: 0), the factors of safety (NCMA minimums prefilled: sliding 1.5, overturning 2.0, interface shear 1.5) and the eccentricity limit.

Auto-design (editor toolbar) varies the unit depth (12 to 30 in.); the number of courses, unit height, setback, soil, interface values and loads are kept. Objective: least unit volume. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Earth pressure	Coulomb Ka(φ, δ, β, θ) on the back of the units, θ = −ω (the plane through the bottom unit's heel and the top unit's back corner), Pa = ½ Ka γ H², thrust at δ − ω below the horizontal; surcharge Ka q H
Base sliding	(μN + Pp) / T ≥ FS
Overturning	About the toe ≥ FS
Bearing and eccentricity	Base pressure under D + H and D + H + L (ASCE 7-22 2.4.1) against the allowable; eccentricity within the entered limit
Interface shear	At each joint between courses: (a_u + N tan λ_u) / T ≥ FS; the worst joint is reported
Local overturning	At each joint: the units above about the joint's front edge ≥ FS

The drawings show the wall section (the units with their setbacks, the leveling pad, the backfill, the soil in front, the surcharge, the horizontal earth pressure on the back plane, the main dimensions) and the service bearing pressure. The leveling pad design, drainage aggregate, drain pipe, cap units and adhesive are not part of the calculation.

Limitations

- Conventional (unreinforced) walls only. Geogrid-reinforced walls (internal stability, pullout, connection strength, the reinforced mass) are not offered: the NCMA Design Manual for Segmental Retaining Walls is not on file and no free worked example allowed a verified implementation.
- The NCMA bearing capacity (Meyerhof with B' = B − 2e) is not computed: enter the allowable bearing pressure.
- Water, seismic loads and global stability are not included; the passive resistance of the embedment is neglected by default.

Verification examples

Verification status: 7 values compared in 1 example (0 published, 1 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

The NCMA design manual and its worked examples are not on file (research/DOCUMENTS_NEEDED.md). Hand calculation in closed form (backend/tests/test_verify_segmental_wall.py): three 8 × 18 in. courses with a 1 in. setback, φ = 32°, δ = 21.3°, a_u = 1000 lb/ft, λ_u = 30°.

What it checks	Hand	ILX Calcs
Batter ω	7.125°	7.125
Coulomb Ka on the battered back	0.2426	0.2426
Sliding force / resistance ÷ 1.5	0.0558 / 0.1566 kip/ft	0.0558 / 0.1566
Overturning moment / resisting ÷ 2	0.0372 / 0.1691 kip-ft	0.0372 / 0.1691
Interface shear capacity at the first joint ÷ 1.5	0.766 kip/ft	0.766

CHAPTER 125

Gabion retaining wall

A gabion retaining wall, per foot of wall: stacked courses of stone-filled baskets, optionally battered back, checked as a gravity wall on its base and at every joint between courses.

Inputs of the gabion wall

- **Gabion courses:** the number of courses and their height, the base course width, the width step per course (each course narrower than the one below; one basket module is 1.5 ft) down to the narrowest width, and the stepped face: at the front (the backs flush, the usual arrangement) or at the back (the front flush). The batter angle tilts the whole wall back (about 6° is common); the base then dips toward the backfill. The gabion unit weight is $\gamma g = \gamma s (1 - n)$ from the stone unit weight and the porosity of the fill.
- **Backfill and soil:** φ, the wall friction δ on the rough gabion back (2/3 φ to φ, the engineer's choice), the backfill slope, γ, a uniform surcharge (dead or live) and the soil depth in front for passive resistance.
- **Stability and bearing:** allowable bearing pressure, the base friction coefficient μ, the friction coefficient between courses and the allowable compressive stress in the gabions (the manufacturer's values), the passive reduction factor and neglected depth, the factors of safety and the eccentricity limit.

Auto-design (editor toolbar) varies the base course width (1.5-ft modules) and the width step per course; the number and height of the courses, the narrowest width, the face, the batter, the soil and the loads are kept. Objective: least gabion volume. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Earth pressure	Coulomb $K_a(\varphi, \delta, \beta, \theta)$ on the plane through the heel of the bottom course and the back corner of the top course (θ negative for a battered wall), $P_a = \frac{1}{2} K_a \gamma H^2$ at H/3, the thrust at $\delta + \theta$ below the horizontal; surcharge $K_a q H \cos\theta / \cos(\theta - \beta)$ at H/2
Sliding	Along the (inclined) base: $(\mu N + P_p) / T \geq FS$, with N normal to the base and T along it
Overturning	About the toe $\geq FS$
Bearing and eccentricity	Pressure along the base under D + H and D + H + L (ASCE 7-22 2.4.1) against the allowable; eccentricity within the entered limit
Joints between courses	For the courses above each joint (the earth pressure on the plane above it): sliding between the baskets, overturning about the joint's front edge, eccentricity and the compressive stress on the joint against the allowable; the worst joint of each check is reported

The drawings show the wall section (the courses with their setbacks and batter, backfill, soil in front, surcharge, the horizontal earth pressure on the back plane, the main dimensions) and the service bearing pressure. The basket mesh, lacing, filter fabric and drainage are not part of the calculation.

Limitations

- A drained backfill is assumed (no water pressure); seismic loads and global stability are not included.
- The steps of the back are replaced by the plane through the heel and the top back corner; the soil between the steps and that plane is not counted as weight (conservative).
- One course height for the whole wall; a custom arrangement of baskets is not offered.

Verification examples

Verification status: 10 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

No free gabion design manual with a complete worked example could be retrieved (research/DOCUMENTS_NEEDED.md). Hand calculations in closed form (backend/tests/test_verify_gabion_wall.py):

What it checks	Hand	ILX Calcs
Gabion unit weight 165 × (1 − 0.30)	115.5 pcf	115.5
Two courses, no batter: sliding force / resistance ÷ 1.5	0.555 / 1.333 kip/ft	0.555 / 1.333
Overturning moment / resisting ÷ 2	1.110 / 6.096 kip-ft	1.110 / 6.096
Joint: sliding force / resistance ÷ 1.5	0.139 / 0.510 kip/ft	0.139 / 0.510
One course battered 6°: Coulomb Ka with θ = −6°	0.2348	0.2348
Force along the inclined base / resistance ÷ 1.5	−0.100 / 0.845 kip/ft	−0.100 / 0.845

CHAPTER 126

Soldier pile wall

A cantilever soldier pile and lagging wall (no anchors or braces), per pile: the embedment by the simplified free-earth method of the Caltrans Trenching and Shoring Manual (AASHTO LRFD 3.11.5.6), the pile moment, shear and top deflection, the steel pile designed to AISC 360-22 and the lagging (timber to NDS 2024, or a concrete panel to ACI 318-19).

Inputs of the soldier pile wall

- **Wall and piles:** excavation height H, pile spacing, the W or HP shape and its steel, the installation (a drilled hole filled with concrete below the excavation, whose diameter is the effective width, or a driven pile, whose flange width is), and the provided embedment D below the excavation.
- **Soil and surcharge:** one granular soil above and below the excavation (φ, γ), the backfill slope (Rankine Ka of the sloping backfill, horizontal component), a uniform surcharge q (adds Ka q over the full depth) and a lateral construction surcharge above the excavation (Caltrans minimum 72 psf).
- **Embedment method:** Kp (Rankine by default; Caltrans recommends a log-spiral value, which can be entered), a factor of safety on passive, the arching factor f (0.08φ ≤ 3 for granular soil by default, Caltrans Eq. 7-2-1), the embedment factor D/D0 (1.2), the point of fixity for the deflection (0.25D below the excavation) and the engineer's deflection limit.
- **Pile design:** the unbraced length (0 where the lagging against the front flange braces the compression flange), Cb, and the load factor on H for LRFD (ASD uses the service values).
- **Lagging:** timber boards (actual thickness, width, spacing, bearing length on the flange, reference values Fb, Fv and Fc⊥ and the factors CD, CM, CF and Cfu of the NDS Supplement Table 4A; DF-L No. 2 values prefilled) or a concrete panel (thickness, bars, f'c); the lagging pressure factor (0.6 where soil arching can develop, Caltrans 6-5).

Auto-design (editor toolbar) varies the embedment (1/2-ft steps), the W or HP pile (lightest first) and the lagging thickness; the excavation, spacing, soil and surcharges are kept. Objective: least steel and lagging. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Pressures	Above the excavation Ka γ z, Ka q and the lateral surcharge over the spacing; below it the active pressure over the effective width b and the passive pressure Kp γ y / FS over f b ≤ spacing (Caltrans 7-2, 7-5.02)
Embedment	D0 from moments about the rotation point O (the reaction R at O drops out); D = 1.2 D0 against the provided embedment (Caltrans 7-5.02, AASHTO 3.11.5.6)
Moment and shear	Maximum moment at the depth of zero shear below the excavation; maximum shear along the pile (at O it is the reaction R, as Caltrans reports it)

Check	Basis
Pile flexure	AISC 360-22 F2 (lateral-torsional buckling with the entered Lb and Cb) and F3 (flange local buckling); LRFD with H × its factor, ASD at service
Pile shear	AISC 360-22 G2.1
Top deflection	Moment-area from the point of fixity to the top, the steel pile alone, against the engineer's limit
Timber lagging	Design pressure = factor × the earth pressure at the excavation level; bending over the clear span plus the bearing length required (Caltrans Eq. 6-5-11), shear 3V/2A, bearing on the flange; NDS 2024 adjusted values Fb CD CM CF Cfu, Fv CD CM, Fc⊥ CM
Concrete lagging	Simple span between the flanges per foot of height: ACI 318-19 flexure, minimum steel, spacing and one-way shear

The drawings (screen, report, DXF and PDF sheet) show the pile elevation with the retained grade, the excavation, the pile and its drilled hole, the provided and required embedment, the driving pressure above the excavation and the passive resistance below it (per pile); the shear and moment along the pile; and a plan of three piles with the lagging between the flanges. Pile tips, caps, anchors, drainage and the lagging installation are not part of the calculation.

Limitations

- Cantilever walls only: anchored or braced walls are not covered.
- One granular soil; layered and cohesive soils, groundwater and sloping ground in front of the wall are not covered. Global stability, basal heave and seismic loads are not included.
- The required embedment uses the simplified method (no rigorous method); the strength of the concrete in the drilled hole is not checked.
- The deflection is a beam approximation from a point of fixity (Caltrans 7-3); a soil-structure analysis (p-y) is needed where movement matters.

Verification examples

Verification status: 14 values compared in 3 examples (2 published, 1 hand calculations from the cited equations): 14 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_soldier_pile_wall.py), Caltrans Trenching and Shoring Manual (July 2025, free at dot.ca.gov):

Example 7-1B, cantilevered soldier pile wall by the simplified method: W14x120 at 8 ft in 2 ft concrete-filled holes, H = 15 ft, φ = 35°, γ = 125 pcf, 72 psf construction surcharge.

What it checks	Published	ILX Calcs
Arching factor f = 0.08φ	2.8	2.8
D0 from moments about O	12.272 ft	12.265
Embedment D = 1.2 D0	14.73 ft	14.72
Depth of zero shear below the excavation	5.997 ft	5.996

What it checks	Published	ILX Calcs
Maximum moment	379.70 kip-ft	379.73
Maximum shear (at O)	137.73 kips	137.62

The example's deflection (1.64 in.) comes from the Caltrans program with its own pile stiffness assumptions and is not compared.

Section 6-5.01, timber lagging: 4x12 DF-L No. 2 at 12 in., piles at 8.5 ft with 12 in. flanges, 580 psf × 0.6.

What it checks	Published	ILX Calcs
F'b = Fb CD CF Cfu	1252 psi	1252
Bending stress	1283 psi	1286
F'v / fv	201 / 50 psi	201 / 50
F'c⊥ / fc⊥	419 / 39 psi	419 / 39

Hand calculation: a driven pile with an entered arching factor: D0 from the closed-form moment balance about O and the depth of zero shear from the quadratic force balance, within 0.1 %.

CHAPTER 127

Bearing capacity and settlement

The geotechnical check of a shallow foundation: the ultimate bearing capacity of a strip, square, rectangular or circular footing on one bearing stratum, by the general bearing capacity equation in the form of FHWA GEC 6 (2002) Eq. 5-14 with the factors of the method the engineer chooses, checked against the bearing pressure on Meyerhof's effective footing; and the settlement of a layered profile below the base (immediate settlement of granular layers, primary consolidation of clay layers). The footing's concrete design is done in the footing types (spread, wall, combined footing); this type answers the soil questions they take as inputs.

Inputs of the bearing capacity check

- **Footing:** shape (strip: loads per ft of length), width B (diameter of a circle), length L (rectangular), depth Df from grade to the base, and, when its weight is included as dead load, the footing thickness (concrete at 150 pcf, the backfill above at the unit weight above the base).
- **Bearing stratum:** cohesion c and friction angle φ (undrained: c = su with φ = 0; drained: c', φ'), the total unit weight below the base and above it, and the groundwater depth below grade (the highest anticipated level). These come from the geotechnical report; GEC 6 Figs. 4-1 and 4-2 correlate SPT values with φ.
- **Method (engineer's choice):** Terzaghi (1943; his own shape factors 1.3 / 0.8 / 0.6), Meyerhof (1963), Hansen (1970) or Vesić (1973), the set FHWA GEC 6 Table 5-1 and AASHTO use, with the shape factors of GEC 6 Table 5-2. Depth factors are off by default: GEC 6 5.2.3.3 includes them only when the cover is high-quality compacted granular fill that will stay in place.
- **Factor of safety** (ASD, 3.0 usual, GEC 6 5.2.6) or **resistance factor φb** (LRFD; 0.45 prefilled, the AASHTO value for the theoretical method in sand, to confirm against the code text). The method switch picks the ASCE 7-22 2.4.1 (ASD) or 2.3.1 (LRFD) combinations.
- **Loads:** per load type, unfactored, at the base center: P down, MB (moves the resultant across B) and ML (along L).
- **Settlement:** the stress below the center (Boussinesq or 2:1), the part of L in the settlement load (sustained live load for consolidation), net or gross pressure, the layers below the base (granular: Es; clay: Cc, Cr, e0, OCR = σ'p/σ'0) and the limit set by the structure.

Auto-design varies the footing width B in 0.5-ft steps (a rectangular footing keeps its L/B ratio); depth, soil, method, loads and factor of safety are the engineer's. See Auto-design.

Checks performed

Check	Basis
Bearing: P / (B'L') ≤ qult / FS (ASD) or φb qult (LRFD), every combination	GEC 6 Eq. 5-14: qult = c Nc sc dc + q Nq sq dq Cwq + 0.5 γ B' Nγ sγ dγ Cwγ

Check	Basis
Factors	Vesić: Nq = e^(π tan φ) tan²(45° + φ/2), Nc = (Nq − 1) cot φ (5.14 at φ = 0), Nγ = 2(Nq + 1) tan φ; Meyerhof Nγ = (Nq − 1) tan 1.4φ; Hansen Nγ = 1.5(Nq − 1) tan φ; Terzaghi Nq = e^(2(3π/4 − φ/2) tan φ)/(2 cos²(45° + φ/2))
Shape factors	GEC 6 Table 5-2: sc = 1 + (B/L)(Nq/Nc) (1 + B/5L at φ = 0), sq = 1 + (B/L) tan φ, sγ = 1 − 0.4 B/L; Meyerhof with Kp; Hansen sq with sin φ
Groundwater	GEC 6 Eqs. 5-9, 5-10: Cwq = 0.5 + 0.5 Dw/Df ≤ 1, Cwγ = 0.5 + 0.5 (Dw − Df)/(1.5B) between 0.5 and 1
Eccentric load	Meyerhof: B' = B − 2eB, L' = L − 2eL; the resultant inside the footing (else not feasible); an advisory when it leaves the middle third
Settlement ≤ limit	Δσ below the center (Boussinesq: Newmark's corner solution, strip and circle closed forms; or 2:1); granular Δσ H/Es; clay Cc H/(1 + e0) log((σ'0 + Δσ)/σ'0), with Cr up to σ'p

The drawings show the section (footing at its depth, grade, groundwater, the settlement layers) and, for a finite footing, the plan with the effective area of the governing combination hatched.

Limitations

- One bearing stratum: a weaker layer within about 2B below the base (punching) is not checked; enter the weaker layer's strength or use a layered method.
- Vertical loads only: load inclination factors are not applied (GEC 6 notes they should not be combined with shape factors); sliding and overturning are checked in the footing types.
- Level ground; footings on or near slopes (GEC 6 5.2.3.5, Ncq and Nγq) and rock are not covered.
- A moment on a circular footing is not covered (enter an equivalent square).
- Terzaghi's Nγ uses Coduto's closed approximation of his chart (to confirm against the source).
- The settlement is the immediate and primary consolidation settlement below the center; time rate, secondary compression and differential settlement are not computed. Δσ H / Es is a one-dimensional estimate with the modulus the engineer enters.

Verification examples

Verification status: 20 values compared in 8 examples (3 published, 5 hand calculations from the cited equations): 19 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_bearing_capacity.py):

Source	What it checks	Published	ILX Calcs
FHWA GEC 6 (2002) Table 5-1	Nc, Nq, Nγ at 30° and 35°	30.1, 18.4, 22.4; Nq 33.3, Nγ 48.0	≈ 30.14, 18.40, 22.40; 33.30, 48.03 (hand-checked)
FHWA GEC 6 Appendix B Example 1, Step 7 (interior pier, φ = 35°, Df = 2.3 m)	sq, sγ, surcharge term	1.7, 0.6, 2,553 kPa	≈ 1.700, 0.6, 2,552 kPa

Source	What it checks	Published	ILX Calcs
same	unit weight term at Bf = 6 m (deviation: the example rounds Cwγ to 0.9)	1,524 kPa	≈ 1,487 kPa
GEC 6 Tables 5-1, 5-2	undrained Nc and sc of a square	5.14, 1.2	5.14, 1.2

Hand calculations cover Terzaghi's, Meyerhof's and Hansen's factors, Meyerhof's effective width of an eccentric strip, and the consolidation of a normally consolidated clay layer under the 2:1 stress. No published worked settlement example with these inputs was found on disk (the open textbook of Kouretzis has settlement examples, but their numbers are figures in the text copy).

CHAPTER 128

Pile axial capacity

The static axial capacity of a single vertical pile through a layered soil profile: side resistance summed over the layers plus toe resistance, for driven piles by the methods of FHWA GEC 12 (2016) and for drilled shafts by FHWA GEC 10 (2018), checked against the largest compression and tension of the ASCE 7-22 combinations with the engineer's factors of safety (ASD) or resistance factors (LRFD). Each layer is split at the groundwater level and the pile head and evaluated at its mid-depth, as the GEC illustrative examples do: thinner layers follow a varying profile more closely.

Inputs of the pile capacity check

- **Pile:** driven pile or drilled shaft; round (closed-end pipe, precast round, shaft), square (precast) or an entered perimeter and toe area (H-pile or open pipe: whether the box or the steel area applies is the engineer's call, GEC 12 7.10.7); the embedded length, the depth of the head below grade (no side resistance above it: cap, scour, excavation), the groundwater depth.
- **Soil layers** (top down from grade, to below the tip): clay (total unit weight, undrained strength su; an α per layer when the engineer reads it from the Tomlinson curves) or sand (φ', N60 for drilled shafts; β and Nt for driven piles, from local experience or the ranges of GEC 12 Table 7-9 after Fellenius 2014).
- **Driven piles:** α in clay by API RP 2A (Randolph and Murphy 1985: α = 0.5ψ^-0.5 for ψ = su/σ'v ≤ 1, 0.5ψ^-0.25 above, ≤ 1) or entered; an optional limit on the toe resistance in sand (the β-method itself has none).
- **Drilled shafts:** the test the su comes from (CIUC as entered; UU and UC converted by GEC 10 Eqs. 10-17, 10-16), the exponent m of Eq. 10-11 (0.6 clean quartzitic sand, 0.8 silty sand), the neglected top of the shaft in clay (5 ft is common practice, GEC 10 10.3.5.2).
- **Factors:** ASD factors of safety for compression and uplift; LRFD resistance factors for side resistance in clay and sand, toe and uplift (GEC 10 values prefilled; the AASHTO driven-pile values are in the hints, to confirm against the code text); the uplift side resistance as a part of the compression value; the pile weight against uplift (buoyant below the water table; 0.6 of it in ASD, 0.9 in LRFD, as the dead-load factors of the uplift combinations).
- **Loads:** per load type, unfactored, compression positive and tension negative.

Auto-design varies the embedded length in 1-ft steps within the soil profile (and the width when unlocked); the soil, coefficients, loads and factors are the engineer's. See Auto-design.

Checks performed

Check	Basis
Compression ≤ Rn / FS or φsc Rs,clay + φss Rs,sand + φp Rp	Rn = Σ fs·perimeter·thickness + qp·At
Driven, clay	fs = α su; toe 9 su (GEC 12 α-method)
Driven, sand	fs = β σ'v (GEC 12 Eq. 7-19); toe Nt σ'p (Eq. 7-20)

Check	Basis
Shaft, clay	fs = α su(CIUC), α = 0.30 + 0.17/(su/pa) (GEC 10 Eq. 10-18); base N*c su, N*c of Table 10-2 (9 at 2,000 psf and up), Eq. 10-20 when the depth is less than 3B
Shaft, sand	β = (1 − sin φ')(σ'p/σ'v)^sin φ' tan φ' ≤ Kp tan φ', σ'p/pa = 0.47 N60^m, β at its 7.5-ft value above that depth (Eqs. 10-8 to 10-13); base 0.60 N60 tsf ≤ 30 tsf (Eq. 10-14)
Tension ≤ ratio × Rs / FS + 0.6 W (ASD) or φup ratio × Rs + 0.9 W (LRFD)	only when a combination gives tension

The drawings show the pile in the soil profile (layers, groundwater, head and tip) and the unit and cumulative side resistance along the pile.

Limitations

- A single vertical pile: group effects and block failure, downdrag (negative skin friction), settlement and the pile's structural capacity are not checked; drivability (wave equation), set-up and relaxation are not covered.
- Nordlund's method and the Tomlinson α curves are chart based: enter α (and β) per layer when using them.
- Rock sockets, cased zones (casing reduction factors) and open-end plugging are not modelled; enter equivalent values.
- Layers are evaluated at mid-depth; split thick layers where the stress or strength changes much.
- The resistance factors prefilled are GEC 10's for drilled shafts; for driven piles enter the agency's factors.

Verification examples

Verification status: 12 values compared in 5 examples (3 published, 2 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_pile_capacity.py):

Source	What it checks	Published	ILX Calcs
FHWA GEC 10 (2018) Illustrative Example 10-1: 5-ft shaft, silty sand 0 to 15 ft and 15 to 33 ft, water at 15 ft	side resistance of layers 1 and 2	197.2, 635.6 kips	≈ 196.6, 631 kips (hand-checked; the example's layer 2 line uses 1.08 for its β of 0.98)
Example 10-2: stiff clay 33 to 60 ft, UU su = 2,000 psf	su(CIUC), side resistance	2,571 psf, 479.7 kips	≈ 2,571 psf, 479.8 kips
Example 10-3	base qBN, RBN	23.1 ksf, 453.6 kips	≈ 23.14 ksf, 454.3 kips

Hand calculations cover the driven-pile α (API RP 2A) and β / Nt methods, the toe in clay, the uplift resistance and the LRFD sum. The GEC 12 design examples print DRIVEN program output rather than hand calculations, so no driven-pile example is reproduced.

CHAPTER 129

Laterally loaded pile (p-y)

A single vertical pile in layered soil under lateral load, head moment and axial compression, analysed by the p-y method of FHWA GEC 9 (2018): the pile is an elastic beam-column on nonlinear soil springs, solved by finite differences for every load combination. The type reports the deflection, slope, moment, shear and soil reaction along the pile, checks a steel pipe (or an entered section) for combined axial load and flexure, and checks the head deflection at the service loads against the engineer's limit. The Broms method stays in the Laterally loaded pile type.

Inputs of the p-y pile

- **Pile:** head condition (free, or fixed against rotation by a cap: the engineer's judgment of the restraint), the height of the pile top above grade (the loads act at the top), the embedded length, and the section: a steel pipe (D, t, Fy, E) or an entered section (width, E, I, nominal Mn and Pn with φ or Ω). The width D is the one the p-y curves use.
- **Soil:** the water table (effective stresses below it use γsat − 62.4 pcf), static or cyclic curves, and the layers top down, each with its own p-y curve family and parameters:
  - *Soft clay, Matlock (1970)*: su, ε50 and J (GEC 9 A.1; J = 0.5 soft, 0.25 medium);
  - *Stiff clay without free water, Welch and Reese (1972)*: su and ε50 (GEC 9 A.3, Table A-2);
  - *Sand, API*: φ' and the initial modulus k (GEC 9 Table A-3: 25 / 90 / 225 pci above water, 20 / 60 / 125 pci below, for loose / medium / dense sand);
  - *Linear*: a constant subgrade modulus kh (Es = kh·D), for an elastic check.
- **Loads:** per load type, unfactored: H and M at the pile top and the axial compression P. The ASCE 7-22 2.3.1 (or 2.4.1) combinations give the strength cases; the 2.4.1 combinations give the service cases for the deflection.
- **Head deflection limit:** set by the supported structure.
- **Pile segments:** the number of finite-difference segments over the whole pile (100 by default, as LPILE).

Auto-design varies the embedded length in 2-ft steps (the shortest that passes). The section, soil, head and limits are kept. See Auto-design.

Checks performed

Check	Basis
A p-y solution exists at every strength combination	GEC 9 Ch. 6: NOT FEASIBLE when the deflection grows past two pile widths (the soil cannot carry the load, or the axial load buckles the pile) or the iteration does not settle in 300 steps or 4 s
Pile section in axial load and flexure	AISC 360-22 H1.1 with Mn by F8 (round HSS: compact, noncompact or slender wall) and Pn by E7 with no length effect (the soil braces the embedded pile); an entered section uses its Mn, Pn, φ and Ω
Head deflection at the service loads	The engineer's limit, ASCE 7-22 2.4.1 combinations
Short (rigid) pile (advisory)	The deflection does not change sign along the pile: the result depends on the tip soil

The solver writes EI·y'''' + P·y'' + Es·y = 0 at each node with central differences, two imaginary nodes at each end, head shear EI·y''' + P·y' = H, head moment EI·y'' = M (free) or slope zero (fixed), and zero moment and shear at the tip. Es = p/y is the secant of each node's p-y curve, iterated until the largest change of deflection is below 10^{-5} in. (or 10^{-5} of the largest deflection); the step is halved if the iteration oscillates.

The drawings show the pile in its soil profile with the load and dimensions, and the deflection, moment and soil reaction along the pile under the governing strength combination.

Limitations

- Stiff clay with free water (Reese et al. 1975) is not offered: its As and Ac factors are read from a chart. The Reese (1974) sand curve is replaced by the API tanh form, which uses the same ultimate resistance. Rock, liquefied sand and cemented soils are not offered.
- The API sand tanh form and its factor A are written from knowledge (API RP 2A is not on disk; to confirm against the code text).
- Group effects (p-multipliers), sloping ground, scour and cyclic degradation beyond the cyclic curves are the engineer's (enter reduced strengths).
- Constant EI: no cracking of a concrete pile (enter a cracked I) and no change of section along the pile.
- Buckling of the stick-up above grade under axial load is not checked separately.

Verification examples

Verification status: 6 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 6 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No published p-y example with a complete input set is on disk (LPILE and COM624P manuals, Reese and Van Impe 2011 and FHWA-IP-84-11 are listed as needed documents). The solver and the curves are checked by hand calculation (backend/tests/test_verify_lateral_pile_py.py):

Source	What it checks	Published	ILX Calcs
Hetényi (1946), long elastic pile, free head: 12.75 × 0.375 in pipe, Es = 637.5 psi, H = 10 kips	Ground-line deflection 2Hβ/Es, Mmax = 0.3224 H/β	0.3716 in, 22.69 kip-ft	(test run)
same, fixed head	Deflection Hβ/Es, head moment H/(2β)	0.1858 in, 35.18 kip-ft	(test run)
GEC 9 Eqs. A-1, A-2, soft clay su = 500 psf, γ' = 50 pcf, b = 12 in	pu at 5 ft and 20 ft	3.0, 4.5 kip/ft	(test run)

CHAPTER 130

Drilled pier

A straight drilled shaft (drilled pier, caisson) in layered soil and rock: the axial side and base resistance by FHWA GEC 10 (FHWA-NHI-18-024, 2018) Chapter 10, uplift, the moment and shear along the shaft from a p-y analysis of the lateral loads, and the structural design of the shaft as a circular reinforced concrete member by ACI 318-19 13.4.3.

Inputs of the drilled pier

- **Shaft:** diameter B, length L, head condition for the lateral analysis (free, or fixed by a cap: the engineer's judgment), and whether the shaft weight (150 pcf) is added to the dead load.
- **Soil and rock:** the water table and the layers top down (the last runs on):
  - *cohesionless*: N60 (for σ'p and the base) and φ' (GEC 10 Eq. 10-9 relates it to (N1)60), and the sand type that sets the preconsolidation correlation: clean sand σ'p/pa ≈ 0.47·N60^0.6, silty sand 0.47·N60^0.8 (Eq. 10-11), gravel 0.15·N60 (Eq. 10-12);
  - *cohesive*: su and the test it came from (UU and UC values are converted to CIUC by Eqs. 10-16, 10-17);
  - *rock*: the uniaxial compressive strength qu of intact core;
  - per layer a side factor (0 where permanent casing or scour removes the side resistance; GEC 10 suggests 0.60 to 0.75 for permanent casing in contact with the soil), and the p-y parameters ε50 (clay) and k (sand API modulus, or the linear spring of rock).
- **Exclusions:** side resistance neglected over the top (GEC 10 10.3.5.2: 5 ft is common practice, or the depth of seasonal moisture change) and, if chosen, over one diameter above a base in clay (AASHTO practice).
- **Loads:** per load type at the top: axial P (compression positive, uplift negative), lateral H and moment M.
- **Resistance factors (LRFD)** from GEC 10 Table 8-4 (side 0.55 / 0.45 / 0.55, base 0.50 / 0.40 / 0.50, uplift 0.45 / 0.35 / 0.40 for cohesionless / cohesive / rock), or the **factor of safety (ASD)**; the rock coefficients C (Eq. 10-21) and N*cr (Eq. 10-23).
- **Shaft reinforcement:** shaft type for ACI Table 13.4.3.2 (uncased cast-in-place 0.55; in rock or a permanent casing 0.60), f'c, fy, the number and size of longitudinal bars, ties or spiral and their spacing, clear cover, and the seismic design category.

Auto-design varies the diameter (6-in steps) and the length (1-ft steps) for the least concrete; the bars, soil, factors and loads are kept. See Auto-design.

Checks performed

Check	Basis
Axial compression	Σφs·RSN + φB·RBN ≥ the factored load (GEC 10 Eq. 10-2), or (ΣRSN + RBN)/FS ≥ the service load
Side, cohesionless	fs = β·σ'v, β = (1 − sinφ')(σ'p/σ'v)^sinφ'·tanφ' ≤ Kp·tanφ' (Eq. 10-13), β not above its value at 7.5 ft
Side, cohesive	fs = α·su(CIUC), α = 0.30 + 0.17/(su/pa) (Eq. 10-18), taken not more than 1.0

Check	Basis
Side, rock	fs = C·pa·√(qu/pa) with qu ≤ f'c (Eq. 10-21)
Base	Cohesionless 0.60·N60 tsf ≤ 30 tsf (Eq. 10-14); cohesive N*c·su (Eq. 10-19, Table 10-2; Eq. 10-20 below 3B); rock N*cr·qu (Eq. 10-23)
Uplift	Σφup·RSN (side only) against the largest net tension (the weight helps as dead load at 0.9)
Lateral p-y solution	FHWA GEC 9 Ch. 6 with API sand, Matlock (su < 1,000 psf) or Welch-Reese clay, and a linear spring in rock; gross EI; NOT FEASIBLE if the soil cannot hold the load
Shaft axial	Pu ≤ φ·Pn,max with φ of ACI 318-19 Table 13.4.3.2
Shaft axial and flexure	The largest p-y moment with its axial load inside the φPn-φMn curve (strain compatibility, φ of Table 21.2.2)
Shaft shear	Vu ≤ 0.75(Vc + Vs), bw = D, d = 0.8D (22.5.2.2), two legs of the tie
Reinforcement required	ACI 318-19 13.4.4.1 where there is uplift or moment
Seismic minimum steel (advisory)	ACI 318-19 18.13.5: ρ ≥ 0.0025 (SDC C), 0.005 (SDC D-F) (to confirm against the code text)

Each layer's resistance is computed at its mid-depth (as the GEC 10 examples do); split a thick layer to follow a change of strength with depth. The drawings show the shaft in its soil profile with its length, diameter and bars, the shaft section with the bars and ties, and the moment and shear along the shaft.

Limitations

- No bells, no settlement analysis (GEC 10 10.3.7 load-transfer curves), no group effects, downdrag or scour.
- The Hoek-Brown base resistance of fractured rock (Eqs. 10-24 to 10-29) and the caving-rock reduction (Eq. 10-22) are not offered; enter a reduced C or N*cr.
- The cap of α at 1.0 is ILX's limit on Eq. 10-18 for very soft clay, where the fit leaves the data of Figure 10-6.
- The lateral analysis uses the gross EI; the moment is taken with the largest axial load at the top (conservative for the interaction); bars are assumed over the full length.
- ACI 318-19 18.13 detailing (length of the confined region, transverse steel) is the engineer's; only the minimum longitudinal ratio is shown.

Verification examples

Verification status: 14 values compared in 5 examples (5 published, 0 hand calculations from the cited equations): 14 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples (backend/tests/test_verify_drilled_pier.py), FHWA GEC 10 (2018):

Source	What it checks	Published	ILX Calcs
Example 10-1: 5 ft shaft, silty sand N60 9 and 27, φ' 38° and 41°	β; RSN of the two layers	0.97, 0.98; 197.2, 635.6 kips	(test run)
Example 10-2: stiff clay su = 2,000 psf (UU)	su(CIUC), α, RSN	2,571 psf, 0.44, 479.7 kips	(test run)
Example 10-3	qBN, RBN	23.1 ksf, 453.6 kips	(test run)

Source	What it checks	Published	ILX Calcs
Example 10-5	Factored demand and ΣφR	766, 855.4 kips	(test run)
Example 10-4: 6 ft socket in sandstone, qu = 360 ksf	fs, RSN, RBN	27.6 ksf, 5,202 kips, 25,447 kips	(test run)

The example's own rounding of β puts its sand side resistances 0.3 to 0.6 % above the unrounded values. The shaft structural checks use the shared ACI 318-19 column library already verified for the concrete column type.

CHAPTER 131

Helical pile

A single helical pile (screw pile) with one or more helices on a round or square steel shaft, in compression and tension: the geotechnical capacity by the individual bearing and cylindrical shear methods, the installation torque from the capacity-to-torque ratio Kt, the shaft in compression with buckling in soft soil and in tension, the helix plates at the manufacturer's strength, and the IBC 2024 1810.3.3.1.9 limits. The methods follow the Iowa DOT Helical Pile Foundation Guide for Bridge Structures (TR-800, 2023), which draws on Perko (2009) and DFI (2019).

Inputs of the helical pile

- **Pile:** round hollow shaft (outside diameter and wall) or square solid bar, Fy and E from the manufacturer's evaluation report, the corrosion loss per exposed face (ICC-ES AC358 sets it by exposure and design life), the depth of the lead helix, the helix diameters from the lead up, the helix spacing (2.4 to 3.6 helix diameters is common, about 3 is efficient) and whether the shaft adhesion above the top helix counts (often ignored).
- **Soil:** the water table and the layers top down: clay with its undrained strength su, sand with φ'.
- **Loads:** axial per load type, compression positive, combined by ASCE 7-22 2.4.1 (ASD) or 2.3.1 (LRFD).
- **Capacity and torque:** the factor of safety (IBC 1810.3.3.1.9: 2) or the engineer's resistance factor (no code value; DFI 2019 suggests driven-pile factors), Kt in 1/ft (ICC-ES AC358 defaults about 10 for 1-1/2 and 1-3/4 in. square, 9 for 2-7/8 in., 7 for 3-1/2 in., 6 for 4-1/2 in. round shafts, or the manufacturer's report), the shaft's installation torque rating, a recorded final torque and a load-test capacity when they exist, and the helix plate nominal strength with its φ or Ω.
- **Shaft buckling:** the horizontal modulus kh of the weakest soil the shaft passes, the unbraced length above firm soil (exposed, scoured, or N < 5) and K.

Auto-design varies the number of helices (the entered sizes, repeating the largest, up to six) and the lead depth (1-ft steps). See Auto-design.

Checks performed

Check	Basis
Compression and tension, helix bearing	Individual bearing Pu = Σ q·An (+ adhesion); cylindrical shear Pu = q·A(lead or top) + τ·(n − 1)s·π·Davg (+ adhesion); the lesser governs (Iowa DOT 4.6.1, 4.6.2, 4.6.7). Clay q = 9·su; sand q = q'·Nq, Nq = 1 + 0.56(12φ)^(φ/54); cylinder τ = su or σ'v(1 − sinφ)tanφ. Net helix area (helix less shaft). Available Pu/FS or φ·Pu
Required installation torque within the rating	T = Pu,required / Kt (Hoyt and Clemence 1989)
Capacity from the recorded torque	Kt·T / FS or φ·Kt·T (IBC 1810.3.3.1.9 item 2)

Check	Basis
Capacity from a load test	Ptest / FS (item 3)
Helix plates	n × the manufacturer's strength with φ or Ω (item 5)
Shaft compression	AISC 360-22 E3 with Pe = 2√(kh·d·EI) for a shaft in soft soil, or π²EI/(K(Lu + 1.4R))² with R = (EI/(kh·d))^¼ when part of it is unbraced (Davisson and Robinson 1965)
Shaft tension	AISC 360-22 D2 yielding
Top helix at least 5 diameters deep	Iowa DOT 4.6.8 (deep behaviour; the torque correlation needs it)
Helix spacing (advisory)	2.4 to 3.6 diameters (Iowa DOT 3.4)

The drawing shows the pile in its soil profile with the helices at their depths, the depths and spacing dimensioned, and the shaft and helix sizes called out; its notes state the final installation torque.

Limitations

- The bearing factors (Nc = 9, the deep Nq expression), the cylinder shear strength in sand (K0 = 1 − sinφ) and the buckling expressions are written from knowledge: the CHANCE Technical Design Manual, Perko (2009), DFI (2019), ICC-ES AC358 and IBC 2024 are not on disk (to confirm against the code text).
- No grouted shafts, battered piles or lateral capacity (use the laterally loaded pile types), no group effects (AC358 asks for 4 helix diameters between piles), couplings or cap brackets.
- Bearing on rock and torque correlation in rock are outside the methods.

Verification examples

Verification status: 7 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No published helical pile design example is on disk. Hand calculations from the equations (backend/tests/test_verify_helical_pile.py):

Source	What it checks	Published	ILX Calcs
Hand: 12 in. helix on 3-1/2 in. shaft, clay su = 1,000 psf	Pu = 9·su·An; ASD available	6.467, 3.234 kips	(test run)
Hand: 10 in. helix at 15 ft, dry sand φ' = 30°	Pu = q'·Nq·An	12.43 kips	(test run)
Hand: two 12 in. helices 18 in. apart in clay	Individual, cylindrical (governs)	12.93, 11.18 kips	(test run)
Hand: Kt = 7, T = 5,000 lb-ft	Kt·T/2	17.5 kips	(test run)
Hand: 3-1/2 × 0.300 in. shaft, kh = 20 pci	Pe = 2√(kh·d·EI)	177.8 kips	(test run)

CHAPTER 132

Pile capacity from CPT

The axial capacity of a single pile directly from a cone penetration test (CPT) sounding, by the method the engineer chooses: LCPC (Bustamante and Gianeselli 1982) as UFS 3-220-20 (DM7.2, 2026) 6-5.4.4 gives it, or Eslami and Fellenius (1997) as FHWA GEC 12 (FHWA-NHI-16-009, 2016) 7.2.1.3.6 gives it. The other method is reported beside it for comparison.

Inputs of the CPT pile

- **Pile:** round or square, width or diameter b, toe depth L (in the sounding's depths), and the depth above which no side resistance counts (pre-drilled, scoured or downdrag zone).
- **LCPC:** the pile group of UFS 3-220-20 6-5.4.4 (I-A drilled shafts, CFA and gravity-grouted micropiles; I-B drilled shafts with permanent casing; II-A driven or jacked concrete; II-B driven or jacked steel), and careful construction (the bracketed fp values of Table 6-22, only with careful execution: the engineer's judgment).
- **Eslami and Fellenius:** the cone area ratio a and the toe influence zone (8b above to 4b below the toe for a pile driven through weak soil into denser soil, 2b above to 4b below for a pile into weaker soil).
- **CPT sounding:** one row per reading: depth, qc, fs (recorded only), u2, and the soil type from the engineer's classification of the sounding (it sets Cs and the LCPC category).
- **Loads:** axial per load type (compression positive), the resistance factor φ (LRFD; GEC 12 suggests the factor of the field verification method) or the factor of safety (ASD).

Auto-design varies the toe depth in 1-ft steps within the sounding. See Auto-design.

Checks performed

Check	Basis
Axial compression	(Rs + Rb)·φ or (Rs + Rb)/FS against the largest compression
Tension	Rs·φ or Rs/FS against the largest tension
LCPC side	fs = qc/ks ≤ fp (Eq. 6-32) by category (soil type and qc/pa) and group, Tables 6-21, 6-22
LCPC base	qb = kt·qca (Eq. 6-33), qca the average over 1.5b above and below the base: values above outside 70 to 130 % of the average and values below over 130 % dropped until none is; kt by Table 6-23
Eslami-Fellenius side	fs = Cs·qE, qE = qt − u2, qt = qc + u2(1 − a) (Eqs. 7-24 to 7-26), Cs from Table 7-11
Eslami-Fellenius toe	qp = Cp·qEg (Eq. 7-27), qEg the geometric average of qE in the toe zone, Cp = 12/b for b > 16 in. (Eq. 7-28)

The side resistance is integrated between the readings (trapezoids, cut at the toe and at the top of the counted zone). The drawings show the pile beside the cone resistance and the unit side resistance along the depth.

Limitations

- The soil type of each reading is the engineer's entry; the soil behaviour type is not derived from the CPT.
- UFS 3-220-20 Table 6-22 prints its group columns compactly; the column assignment used here (I-A, I-B, II-A, II-B) follows Bustamante and Gianeselli's table as recalled (to confirm against the original).
- GEC 12 states qE without subtracting the effective overburden; this follows GEC 12 as printed.
- No group effects, downdrag, setup or relaxation, structural capacity or driveability.

Verification examples

Verification status: 9 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published worked example of either method is on disk. Uniform soundings make each value a hand calculation from the published equations and tables (backend/tests/test_verify_cpt_pile.py):

Source	What it checks	Published	ILX Calcs
Hand, LCPC: 12 in. steel pile, 20 ft, sand qc = 200 ksf	Rs, Rb	106.4, 78.54 kips	(test run)
Hand, Eslami-Fellenius: same	Rs, Rb	50.27, 157.1 kips	(test run)
Hand, LCPC: 24 in. drilled shaft, 30 ft, clay qc = 40 ksf	Rs (normal, careful), Rb	139.6, 188.5, 43.98 kips	(test run)
Hand, Eslami-Fellenius: same with u2 = 10 ksf	Rs, Rb (Cp = 0.5)	301.6, 50.27 kips	(test run)

CHAPTER 133

Braced excavation

A flexible excavation wall (sheeting, or soldier piles and lagging) supported by struts or rakers inside the cut, or by ground anchors: the apparent earth pressure diagrams of UFS 3-220-20 (DM7.2, 2026) 4-8.3, the support loads by the tributary area or the hinge method, the wall and wale moments, and the stability of the base against heave (DM7.2 2-4.3.3).

Inputs of the braced excavation

- **Excavation:** soil (sand, or undrained clay whose envelope follows the stability number N0 = γH/su: stiff clay when N0 ≤ 4, soft to medium clay above), support (internal bracing, or ground anchors with the FHWA 1999 diagrams; DM7.2 leaves the use of the FHWA diagrams for preloaded struts to the engineer), depth H, width B (and length L for Bjerrum and Eide), the support levels, their horizontal spacing, and the method for the support loads.
- **Soil and water:** γ, φ' (sand) or su above and below the base (clay); for sand below the base γ2 and φ'2 (Figure 2-7(a)); the depth to a firm layer below the base (B1 = 0.7B for deep clay); whether deep soft normally consolidated clay underlies the cut (m = 0.4·FBH in Figure 4-52); the stiff clay coefficient (0.4, or 0.2 for small movements and a short construction period); a uniform surcharge with its pressure coefficient (Ka for sand; the engineer's choice for clay); and the water table behind the wall with the cut dewatered to the base.
- **Base stability:** the method for clay (DM7.2 Figure 2-7(b) flexible wall, 2-7(c) very stiff wall with adhesion α, Terzaghi 1943, or Bjerrum and Eide 1956), the wall embedment below the base, and the required factor of safety (1.5, DM7.2 2-4.3.3).
- **Wall and wale:** arching in the lower spans (DM7.2 4-8.3.1: pressures may be reduced to 80 % in all but the upper span), and the section modulus and Fy of the wall per foot and of the wale (0 reports the moment only).

Auto-design varies the horizontal spacing of the struts or anchors (1-ft steps) for the entered wale. See Auto-design.

Checks performed

Check	Basis
Apparent pressure, sand	Internal: uniform 0.65·Ka·γ·H; anchors: trapezoid σH = 0.65·Ka·γ·H²/(H − H1/3 − Hn+1/3) (Figure 4-51)
Apparent pressure, soft to medium clay	Internal: Ka = 1 − m·4su/γH, σH = Ka·γ·H reached at 0.25H; anchors: Ka = 0.22 (4 < N0 ≤ 5.14) or 1 − 4su/γH + 2√2(d/H)(1 − 5.14su,b/γH) (Figure 4-52)
Apparent pressure, stiff clay	0.2 to 0.4·γ·H over the middle (internal) or between the anchor ramps (Figure 4-53)
Support loads	Tributary area (half the distance to the next support) or hinge method (simple spans, the top cantilever to the first level, the lowest span to the base)
Wall moment	The top cantilever exactly; spans w·l²/10 with the largest pressure on the span; first yield Fy·S with 1.6 H (LRFD)

Check	Basis
Wale moment	T·s²/10 for a continuous wale; first yield
Base stability, sand	FBH = 2·Nγ·(γ2/γ1)·Ka·tanφ'1 (Figure 2-7(a), Vesić Nγ)
Base stability, clay	(5.7·su,b·B1 + su,h·H [+ (1 + α)·su,d·D]) / (γ·H·B1 + q·Bs) (Figure 2-7(b), (c)); Terzaghi 5.7·su·B1/(γHB1 + qB1 − su·H); Bjerrum-Eide Nc·su/(γH + q)
Force on buried sheeting	When FBH is below the required value: P'H of Figure 2-7(b)

The drawings show a section through one side of the cut (wall, retained ground, base, supports with their loads per foot, water table) and the pressure diagram used.

Limitations

- Ground movements and their effect on neighbours (DM7.2 2-4.4, Table 2-8), seepage and piping, point and line surcharges (DM7.2 Figure 4-18), and the strut, anchor and connection design are the engineer's.
- One soil above the base: layered retained soil should be averaged by the engineer, as the apparent diagrams assume.
- Terzaghi (1943) and Bjerrum and Eide (1956) are written from knowledge (to confirm against the original papers); the Skempton Nc expression used for Bjerrum-Eide caps H/B at 2.5.
- The apparent diagrams suit deep (over about 20 ft) and relatively wide cuts (DM7.2 4-8.3).

Verification examples

Verification status: 9 values compared in 2 examples (1 published, 1 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Published worked example and hand calculations (backend/tests/test_verify_braced_excavation.py):

Source	What it checks	Published	ILX Calcs
UFS 3-220-20 Figure 4-55: 12 ft wide, 20 ft deep cut in soft clay, su = 400 psf	FBH, Ka, σh	1.35, 0.64, 1.54 ksf	(test run)
same	Resultant and its height, force on the buried sheeting	27 kips/ft, 8.8 ft, 1.6 kips/ft	(test run)
Hand, Figure 4-51: sand φ' = 30°, H = 24 ft, levels 4, 12, 20 ft	σH, tributary loads	624 psf, 4.99 kips/ft	(test run)

The example's 10-kip point surcharge is not modelled; its wall pressure from that load (Figure 4-18) is not compared.

CHAPTER 134

Slope stability

The factor of safety of a simple slope against sliding on circular surfaces by Bishop's simplified method, with the ordinary method of slices for comparison (UFS 3-220-10, DM7.1 2026, 7-4). The type searches a grid of centers and radii for the critical circle, checks one entered circle, or solves an entered slice table as DM7.1 Figure 7-4 tabulates it.

Inputs of the slope

- **Analysis:** search, one circle, or a slice table; Bishop simplified or the ordinary method of slices (the other is reported); the required factor of safety (the engineer's: 1.5 long-term static is common, 1.3 at the end of construction).
- **Slope:** height H and the face slope n horizontal to 1 vertical, with level ground at the toe (elevation 0) and above the crest; the firm stratum elevation (no circle passes below it); horizontal layers to their bottom elevations with γ, γsat, and c' and φ' (or su with φ = 0 for an undrained analysis).
- **Pore water:** dry, a horizontal water table (hydrostatic pore pressure at the slice base; γsat below it) or the pore pressure ratio ru (DM7.1 7-5: pore pressures belong in effective stress analyses only).
- **Surcharge:** a uniform load on the crest over a width from the crest.
- **Search:** the number of grid centers per side and radii per center. The grid spans from in front of the toe to past the crest and from just above the crest to well above it; the radii are tangent to depths from the firm stratum to the toe, plus the circle through the toe. A second pass refines around the best center.
- **Circle:** center (x from the toe, y elevation) and radius. **Slices:** b, W, α, c', φ' and u·b per foot of slope.

Checks performed

Check	Basis
Factor of safety ≥ the required value	Bishop: F = Σ[c'b + (W − ub)tanφ']/mα ÷ ΣW·sinα, mα = cosα(1 + tanα·tanφ'/F), iterated (DM7.1 Figure 7-4); ordinary method: Σ[c'l + (W·cosα − ul)tanφ'] ÷ ΣW·sinα with the effective normal force not below zero
Search limits (advisory)	At most 6,000 circles and 12 s; a capped search says that a lower factor may exist
Critical center on the grid edge (advisory)	A lower factor may lie outside the grid

Hard limits keep every input finite: 30 grid centers per side, 25 radii, 100 slices, 100 Bishop iterations per circle. Circles whose mα falls below 0.2 at any slice are skipped (the numerical difficulty of the simplified method at steep exit angles); a search that finds no admissible circle is NOT FEASIBLE. The drawing shows the slope, layers, firm stratum, water table and surcharge with the critical circle, its center, radius and factor of safety.

Limitations

- Circular surfaces only; no Spencer or Morgenstern-Price (DM7.1 recommends a full equilibrium method and a thorough search for final design).
- Simple slope geometry with horizontal layers; no benches, berms, sloping layers or water table, flow nets, external water, tension cracks, reinforcement or seismic (pseudo-static) loading.

Verification examples

Verification status: 3 values compared in 2 examples (1 published, 1 hand calculations from the cited equations): 3 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Source	What it checks	Published	ILX Calcs
UFS 3-220-10 Figure 7-4: nine slices, c' = 90 psf, φ' = 32°	ΣW·sinα, Bishop F	12.3 kips, 1.3	(test run)
Hand, same slices	Ordinary method of slices	1.174	(test run)

The search is checked in backend/tests/test_slope_stability.py against the infinite slope (dry sand, tanφ/tanβ) and Taylor's stability number for undrained clay (about 5.5 for deep circles at 45°).

CHAPTER 135

Liquefaction triggering

The simplified liquefaction triggering procedure layer by layer from SPT or CPT data, with the earthquake demand from the ASCE 7-22 11.8.3 peak ground acceleration PGA_M: the cyclic stress ratio, the cyclic resistance by Boulanger and Idriss (2014) or, for SPT, Youd et al. (2001), the factor of safety of each layer, and an estimate of the free-field post-liquefaction settlement (Idriss and Boulanger 2008, after Yoshimine et al. 2006).

Inputs of the liquefaction check

- **Earthquake:** PGA_M = FPGA·PGA for the site class (ASCE 7-22 11.8.3; the USGS values) and the moment magnitude M (from the USGS deaggregation: mean or modal, the engineer's choice).
- **Site:** SPT or CPT; the procedure (Youd et al. 2001 is offered for SPT only); the water table during the earthquake; the layers top down, each evaluated at its mid-depth: thickness, total unit weight (saturated below water), N60 (corrected for energy, rod length, sampler and borehole) or qc, fines content FC, and whether the layer is susceptible (plastic or clayey soils by the engineer's judgment).
- **Criteria:** the required factor of safety (1.0 marks triggering; 1.1 to 1.3 is common for design), the Kσ exponent f of Youd et al., and the settlement limit set by the structure.

Checks performed

Check	Basis
Lowest factor of safety ≥ the required value	FS = CRR·MSF·Kσ / CSR, CSR = 0.65·PGA_M·(σv/σ'v)·rd
B&I 2014, SPT	(N1)60cs = CN·N60 + ΔN(FC), CN = (pa/σ'v)^m ≤ 1.7, m = 0.784 − 0.0768√(N1)60cs (iterated); CRR = exp(N/14.1 + (N/126)² − (N/23.6)³ + (N/25.4)⁴ − 2.8); MSFmax = 1.09 + (N/31.5)² ≤ 2.2; Cσ = 1/(18.9 − 2.55√N) ≤ 0.3
B&I 2014, CPT	qc1Ncs = CN·qc/pa + Δqc1N(FC), m = 1.338 − 0.249·qc1Ncs^0.264; CRR = exp(q/113 + (q/1000)² − (q/140)³ + (q/137)⁴ − 2.80); MSFmax = 1.09 + (q/180)³ ≤ 2.2; Cσ = 1/(37.3 − 8.27q^0.264) ≤ 0.3
B&I 2014 common	rd = exp(α(z) + β(z)·M) (Idriss 1999) to 34 m; MSF = 1 + (MSFmax − 1)(8.64·e^(−M/4) − 1.325); Kσ = 1 − Cσ·ln(σ'v/pa) ≤ 1.1
Youd et al. 2001, SPT	CN = (pa/σ'v)^0.5 ≤ 1.7; (N1)60cs = α + β(N1)60; CRR = 1/(34 − N) + N/135 + 50/(10N + 45)² − 1/200 for N < 30 (too dense to liquefy above); rd by Liao and Whitman; MSF = 10^2.24/M^2.56; Kσ = (σ'v/pa)^(f−1) ≤ 1
Settlement estimate	εv = 1.5·exp(−2.5Dr)·min(0.08, γmax), γmax from FS and Dr (Yoshimine et al. 2006), Dr from the penetration resistance; Σ εv·t against the limit

The drawings show the soil column with each layer's factor of safety and the CSR, CRR·MSF·Kσ and FS by depth.

Limitations

- The equations of Boulanger and Idriss (2014), Youd et al. (2001) and Idriss and Boulanger (2008) are written from knowledge; the reports are not on disk (to confirm against the published text).
- Fines content is entered (no Ic-based estimate from the CPT); qc is used as entered (enter qt where the pore pressure correction matters); no thin-layer or transition corrections.
- No lateral spreading, residual strength or flow failure, and no effect on foundations (downdrag, loss of bearing); the settlement is free-field volumetric settlement only.

Verification examples

Verification status: 10 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Hand calculations from the equations (backend/tests/test_verify_liquefaction.py), with a layer at one atmosphere of effective stress so that CN and Kσ are 1:

Source	What it checks	Published	ILX Calcs
Hand, B&I 2014 SPT N60 = 20, PGA_M 0.2 g, M 7.5, z = 10.34 m	rd, CSR, CRR, MSF, FS, settlement	0.8913, 0.2317, 0.2059, 1.000, 0.888, 10.43 in.	(test run)
Hand, B&I 2014 CPT qc1Ncs = 100	CRR	0.1373	(test run)
Hand, Youd et al. 2001 SPT N = 20, M 6.5	rd, CRR, MSF	0.898, 0.2154, 1.442	(test run)

CHAPTER 136

Ground improvement

Aggregate columns (vibro stone columns or rammed aggregate piers) under an embankment or a wide load, by FHWA GEC 13 (FHWA-NHI-16-027, 2017) Chapter 5: the unit cell and area replacement ratio, the stresses on the columns and the soil with the engineer's stress concentration ratio, the column bearing resistance, the settlement of the treated ground by Priebe's improvement factor, by stress reduction or by the two-layer method for rammed aggregate piers, and the composite strength for a stability analysis.

Inputs of the ground improvement

- **Columns:** the system, the grid (triangular, square, or an entered ratio A/Ac), spacing s and diameter d (GEC 13: 3.0 to 3.6 ft columns at 5 to 12 ft are common), the stress concentration ratio n (2 to 5 for stone columns, 2.5 for preliminary design; 5 to 10 for rammed aggregate piers under embankments, about 10 under rigid footings), the column friction angle φc (40 to 45° stone, 45 to 50° rammed piers), the column unit weight and length.
- **Soil and load:** the applied pressure q (fill unit weight times height for an embankment), the compressible layer's thickness, γsat, water table, su, and its settlement model: normally consolidated clay (Cc, e0) or elastic (Es).
- **Design criteria:** the settlement method, the pier stiffness modulus kg (75 to 360 pci under rigid footings, less for embankments), the column bearing factor Nc (22 for stiffer inorganic soils, 18 for peat, organic or very soft clay) with a factor of safety of 3, an allowable soil pressure between the columns (0: no check), and the settlement limit.

Auto-design varies the spacing (6-in steps) for the fewest columns. See Auto-design.

Checks performed

Check	Basis
Column stress ≤ Nc·su / FS	GEC 13 Eqs. 5-5, 5-6
Soil stress ≤ the allowable (optional)	GEC 13 Eq. 5-4
Settlement of the treated ground ≤ the limit	Untreated: Cc·H/(1 + e0)·log((σ'0 + Δq)/σ'0) or Δq·H/Es (Eqs. 5-14, 5-15); Priebe: ÷ n0 = 1 + as[(5 − as)/(4Kac(1 − as)) − 1]; stress reduction: × σsoil/q; rammed piers: qg/kg + the settlement below the piers (Eqs. 5-11 to 5-13)
Composite strength (reported)	τ = (1 − as)·cu + as·σv·tanφc, γ = γsc·as + γsoil(1 − as) (Eqs. 5-7, 5-8)
Columns shorter than the layer (advisory)	The soil below the columns settles unimproved

The drawings show a plan of the column grid with the spacing and the replacement ratio, and a section through the treated layer with the column length.

Limitations

- Priebe's closed form is written from knowledge (Priebe 1995 is not on disk; GEC 13 shows it only as a curve); it is an upper-bound estimate (GEC 13 4.1.2.6). Eq. 5-7 is printed unclearly in GEC 13 and is used as $\tau = (1 - as)\cdot cu + as\cdot\sigma v\cdot\tan\varphi c$ (to confirm against the code text).
- No time rate of settlement (radial drainage), liquefaction mitigation, bulging of single columns, group or global stability; the field verification program is the engineer's.

Verification examples

Verification status: 8 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

FHWA GEC 13 (2017) 4.3 Design Examples (backend/tests/test_verify_ground_improvement.py):

Source	What it checks	Published	ILX Calcs
4.3.1 Rammed aggregate piers: 2.75 ft at 5 ft, n = 6, kg = 65 pci, q = 2,500 psf	σ'0, untreated settlement, Ra, qg, suz	432 psf, 22 in., 0.27, 6,383 psf, 0.68 in.	(test run)
4.3.2 Stone columns: A/Ac = 3.6, q = 1,875 psf, 50 ft clay	σ'0, untreated settlement, settlement ratio	1,440 psf, 27 in., 2.7 (chart)	(test run)

The example rounds Ra to 0.27 (exact 0.2744 for the triangular grid), which puts its qg 0.9 % above the exact value. Its settlement ratio is read from the Priebe curve; the closed form gives 2.81 with φc = 40°.

CHAPTER 137

Micropile

One micropile by the allowable stress method of FHWA NHI-05-039, *Micropile Design and Construction* (2005): the grout-to-ground bond capacity of the bond zone, the structural capacity of the cased length and of the uncased bond zone in compression and tension, buckling where the pile crosses a void or very soft soil, and the bearing of the top plate on the cap by ACI 318-19 22.8. Service loads use the ASCE 7-22 2.4.1 combinations, the cap bearing the 2.3.1 combinations.

The FHWA manual is not on disk: its equations, the αbond ranges of its Table 5-3 and the strain-compatibility limit are written from knowledge and are marked "to confirm against the code text" in the report.

Inputs of the micropile

- **Loads:** unfactored axial load per load type, compression positive, tension negative.
- **Micropile:** the casing (common API N80 sizes), its yield, the cased length, the casing area left at the threaded joint as a fraction (from the supplier's joint detail), a corrosion allowance on the outside face, the center bar and its yield, the grout strength.
- **Bond zone:** grouted diameter and length, the ground and the grouting method (Type A to D), the ultimate bond αbond (yours, from the geotechnical report and tests; the Table 5-3 range is shown) and the factor of safety (2.0).
- **Buckling:** an unsupported length (0 = laterally supported), K and a factor of safety.
- **Cap connection:** cap f'c, the square top plate and the cap concrete beyond it (for A2).

Auto-design varies the bond length (10 to 60 ft), the casing and the center bar; loads, ground, αbond, grout and factors of safety are kept.

Checks performed

Check	Basis
Bond capacity, compression and tension	αbond π Db Lb / FS (FHWA NHI-05-039 Ch. 5); αbond against Table 5-3 (advisory)
Cased length, compression	0.40 f'cg Ag + 0.47 Fy (Abar + Acasing at the joint)
Uncased bond zone, compression	0.40 f'cg Ag + 0.47 Fy Abar
Tension	0.55 Fy Abar
Fy	Least of bar and casing yield, not more than 87 ksi (grout strain 0.003)
Buckling (where entered)	Euler over K Lu with the steel EI, divided by the factor of safety
Cap bearing	ACI 318-19 22.8.3.2: 0.65 × 0.85 f'c A1 √(A2/A1), √(A2/A1) ≤ 2, factored load

The drawing is an elevation of the cap, the cased length and the bond zone with the bar.

Limitations

- Group effects and spacing, settlement, lateral load and bending, the plunge length, the grout-to-bar bond and the cap design (punching, reinforcement) are not covered.
- Buckling ignores the lateral support of the soil (conservative over the length entered); a rational analysis (Davisson, p-y) may give more.
- FHWA NHI-05-039 not on disk: every FHWA equation and range is to confirm against the code text.

Verification examples

Verification status: 8 values compared in 7 examples (0 published, 7 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

No published micropile example is on disk. Hand calculations (closed form) check the bond capacity, the cased and uncased compression, tension, the 87 ksi limit, the joint and corrosion area, Euler buckling and the cap bearing.

CHAPTER 138

Ground anchor

One prestressed ground anchor (tieback) of an anchored wall or a tiedown by FHWA GEC 4, *Ground Anchors and Anchored Systems* (FHWA-IF-99-015, 1999): the design load, the bond zone capacity and the bond length it needs, the tendon against 60, 70 and 80 % of its specified minimum tensile strength, the unbonded length, the overburden over the bond zone and the movement expected at the test load. Loads are service loads: GEC 4 designs anchors with factors of safety.

Inputs of the ground anchor

- **Design load:** entered, or from the anchor row's horizontal load per length of wall TH (from the apparent earth pressure envelope, GEC 4 5.2, not part of this calculation), the spacing and the inclination: DL = TH s / cos i.
- **Bond zone:** soil or rock and the ultimate load transfer rate: the presumptive values of Table 6 (small-diameter gravity-grouted soil anchors) or Table 8 (rock), a rate you enter (geotechnical report, preproduction tests), or an average ultimate bond stress of Table 7 times the drilled perimeter. The factor of safety is yours: GEC 4 uses 2.0 in soil and 3.0 in rock, and the calculation says so when you enter less.
- **Tendon:** 0.6 in. Grade 270 strands (Table 10) or ASTM A722 bars (Table 9); the maximum test load (1.33 DL for performance and proof tests) and the lock-off load as fractions of DL; the corrosion protection class (a note).
- **Geometry:** the unbonded length, the distance from the head to the critical potential failure surface along the anchor (from the stability analysis, or the active wedge at 45° + φ'/2), the wall height, the depth of the head and the jack length.

Auto-design varies the bond length (15 to 60 ft) and the tendon (1 to 19 strands, or the Table 9 bars); the load, the ground, the transfer rate, the factor of safety, the unbonded length and the geometry are kept.

Checks performed

Check	Basis
Tendon at the design load, the test load and lock-off	DL ≤ 0.6 SMTS, TL ≤ 0.8 SMTS, lock-off ≤ 0.7 SMTS (GEC 4 5.3.8)
Bond zone capacity	DL ≤ Lb × rate / FS (5.3.6); the bond length DL needs
Factor of safety and bond length range (advisories)	2.0 soil, 3.0 rock; 15 to 40 ft in soil, 10 to 33 ft in rock (5.3.6)
Unbonded length	≥ 15 ft (strand) or 10 ft (bar) (5.3.4); ≥ distance to the failure surface + max(H/5, 5 ft) (5.3.2)
Overburden (soil anchors)	≥ 15 ft over the center of the bond zone (5.3.7)
Test acceptance	Elastic movement at the test load for the apparent free length range, jack + 0.8 Lfree to jack + Lfree + 0.5 Lb (7.4.5.3, Eq. 49)

The drawing is a section through the wall with the anchor at its inclination, the unbonded length, the bond zone and the failure surface where it crosses the anchor.

Limitations

- The earth pressure and TH, the stability of the anchored system (internal and external, 5.4 and 5.5), the wall, the waler or connection, group effects and creep testing are yours.
- PTI DC35.1 is not on disk: the limits are those GEC 4 quotes from PTI (1996) (to confirm against the code text). The tendon modulus (28,500 ksi strand, 29,000 ksi bar) and the alignment load of 0.05 DL used for the expected movement are our values (to confirm against the code text).
- Presumptive transfer rates are for preliminary design; every production anchor is proven by its test.

Verification examples

Verification status: 17 values compared in 8 examples (6 published, 2 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	What it checks	Published	ILX Calcs
GEC 4 design example 1, upper anchor	DL = 168 kN/m × 2.5 m / cos 15°	435 kN	434.8 kN
same	Allowable bond, 12 m at 100 kN/m, FS 2.0	600 kN	600 kN
same	32 mm Grade 150 bar at 0.6 SMTS	501 kN	500.4 kN
Lower anchor	DL; bond length 445 × 2.0 / 100; 3 strands at 0.6 SMTS	445 kN; 8.9 m; 469 kN	same
Tables 9 and 10	Bar and strand capacities	as printed	same

Hand calculations: the bond stress route (τ π d) and the depth of the bond zone center.

CHAPTER 139

Masonry shear wall

A reinforced concrete masonry (CMU) shear wall loaded in its plane, designed by strength design to TMS 402-22 Chapter 9, with the prescriptive seismic reinforcement of Chapter 7 and ASCE 7-22 2.3.1 load combinations. The design section is the base of the wall: axial load with in-plane moment by strain compatibility, in-plane shear, the reinforcement limits, and the wall type the seismic design category permits.

The calculation is by strength design only, so the LRFD / ASD switch is not shown. Allowable stress design (TMS 402-22 Chapter 8) is not offered. A calculation saved with ASD is designed by strength design.

Edition. The calculation is written from TMS 402-13 / TMS 602-13 (code, commentary and specification), checked provision by provision on 2026-11-01. Section numbers below are the TMS 402-22 numbers from TMS's own cross-reference list; the 402-22 text itself is not on hand. φ, γg, the maximum reinforcement and shear friction follow 402-22 as TMS's own 2022 handouts state them; items not yet confirmed in the code are listed under the limitations.

Inputs of the masonry shear wall

- **Wall:**
  - The nominal thickness: 6, 8, 10 or 12 in. units, with specified thicknesses of 5.625, 7.625, 9.625 and 11.625 in.
  - The wall length Lw and height h.
  - The grout: fully grouted, or grouted only at the reinforced cells. For a partially grouted wall the net section is the two face shells (ASTM C90 minimums: 1.0 in. for 6 in. units, 1.25 in. for the others) plus 8 in. of full thickness per grouted cell, centered on the bar. The webs are neglected, which is conservative.
- **Materials:**
  - f'm, entered directly or by the unit strength method from the unit strength and the mortar type (TMS 602-22 Table 2).
  - Strength design requires f'm of at least 1,500 psi (Table 4.3.1); below that the calculation is not feasible. A specified f'm above 4,000 psi is permitted, but 4,000 psi is the value used for the nominal strengths of concrete masonry, and the steps say so.
  - The unit strength method assumes ASTM C90 units sampled and tested, bed joints of 5/8 in. or less, grout to TMS 602 Article 2.2 and units at least 4 in. high (TMS 602-13 Article 1.4 B.2.b).
  - Reinforcement of Grade 60 or Grade 40.
- **Vertical reinforcement:**
  - The number of end bars at each end (one bar per cell, 8 in. apart) and their size.
  - The distance from the wall end to the first bar (4 in. is the center of the end cell).
  - The distributed bar size and their maximum spacing. The distributed bars are spaced evenly between the innermost end bars, at no more than that spacing.
  - Whether the compression bars are laterally tied. They are untied in the usual case, and then bars in compression are neglected, both in the axial strength and in the strain compatibility.

- **Horizontal reinforcement:**
  - Bond-beam bars (size and spacing), or joint reinforcement (two W1.7 wires at 8 or 16 in.), or none.
  - Joint reinforcement counts toward the prescriptive minimum of ordinary and intermediate walls but is not counted as shear reinforcement (Vns = 0).
- **Loads at the base:** one row per load type (D, L, Lr, S, R, W, E):
  - P is the axial compression (concentric).
  - V is the in-plane shear.
  - M is the base moment; left blank, it is V × h.
  - E is the horizontal seismic load effect as you enter it, ρQE.
  - SDS adds the vertical seismic effect Ev = 0.2SDS D (ASCE 7-22 12.4.2.2) to the 2.3.6 combinations: (1.2 + 0.2SDS)D + Eh + L + 0.15S and (0.9 − 0.2SDS)D + Eh. It is 0 by default (Eh only); a note says so while E is present.
  - The optional self-weight adds the wall weight (psf) × Lw × h to D. NCMA TEK 14-13B values for normal-weight fully grouted units are in the hint.
- **Seismic detailing:**
  - The shear wall type (ordinary, intermediate or special reinforced): the type you assumed for R in ASCE 7-22 Table 12.2-1.
  - The seismic design category.
  - For intermediate and special walls, whether the design uses R ≤ 1.5. That allows the exemption from the maximum reinforcement where Mu/(Vu dv) ≤ 1 (402-16 9.3.3.2.4 (to confirm against the code text: at 402-22 9.3.5.6.1)). Without it, such a wall with Mu/(Vu dv) < 1 is checked with α = 1.5.
  - The shear friction coefficient μ: 1.00 on concrete with an unfinished or intentionally roughened surface, which the drawings must then specify; 0.70 for all other conditions, prefilled (to confirm against the code text: 9.3.5.5).
  - The area crossing the base Asp. 0 takes every vertical bar, as the TMS example does; enter less when not every bar is doweled into the support.
  - The mortar type also matters in SDC D, E and F (7.4.4.2.2). It is an input only with the unit strength method; with f'm entered, the check is a reminder.

Auto-design (editor toolbar) varies the nominal thickness, distributed vertical bars (size and spacing), end bars and horizontal bars. Objectives: lowest cost index (units, grout, reinforcement); least reinforcement. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Axial strength	9.3.3.1.1 (402-13 Eqs. 9-19 / 9-20): Pn = 0.80 [0.80 f'm (An − Ast) + fy Ast] (1 − (h/140r)²), or (70r/h)² above h/r = 99 (r out of plane, net section); Ast only when the bars are laterally tied (9.3.2(e)); φ = 0.65, pure compression being compression controlled (Table 9.1.4)
In-plane flexure with axial load	9.3.2 strain compatibility: εmu = 0.0025, 0.80 f'm over a = 0.80c on the net section, Es = 29,000 ksi (4.2.2.1), each bar at Es ε up to fy; Mn at Pn = Pu/φ about the wall mid-length, for every combination; φ by Table 9.1.4 from the net tensile strain: 0.65 at or below εty, 0.90 at εty + 0.003 or more, linear between
Axial limit	9.1.4.4.2: φPn ≤ 0.65 Pbal in the tension-controlled and transition regions, Pbal at εt = εty; a larger Pu is designed on the compression-controlled branch at φ = 0.65

Check	Basis
In-plane shear	9.3.3.1.2 (402-13 Eqs. 9-21 to 9-25): Vn = (Vnm + Vns) γg, Vnm = [4.0 − 1.75 Mu/(Vu dv)] Anv √f'm + 0.25 Pu, Vns = 0.5 (Av/s) fy dv, dv = Lw, Vn ≤ 6 to 4 Anv √f'm γg, γg = 0.70 for partial grout (Eq. 9-15); φ = 0.80 (9.1.4.5)
Shear friction at the base	9.3.5.5 (to confirm against the code text): Vnf = μ (Asp fy + Pu) at Mu/(Vu dv) ≤ 0.5, 0.65 (0.75 Asp fy + Pu) at 1.0 or more, linear between, for every combination; φ = 0.80
Special wall shear for 1.25 Mn	7.3.2.6.1.1: φVn ≥ the lesser of the shear at 1.25 Mn and φ × 2.5 Vu (Vn need not exceed 2.5 Vu), seismic combinations
Maximum flexural reinforcement	9.3.5.6.1, intermediate and special walls; 402-22 deleted it for ordinary walls, which get a note. The rule below is carried from 402-16 9.3.3.2 (to confirm against the code text): at the extreme tension bar strain α εy the masonry and compression bars (tied or not) must balance the tension bars and D + 0.75L + 0.525QE. α = 3 (intermediate) or 4 (special) where Mu/(Vu dv) ≥ 1; where Mu/(Vu dv) < 1, no limit with R ≤ 1.5, otherwise α = 1.5 (402-16 9.3.3.2.4)
Special boundary elements (note)	9.3.5.6.2 (402-13 9.3.6.5): an alternative that waives the maximum reinforcement; the note reports its screens (Pu ≤ 0.10 Ag f'm, and Mu/(Vu dv) ≤ 1 or Vu ≤ 3 Anv √f'm with Mu/(Vu dv) ≤ 3). Boundary elements are not designed
Bar size	6.1.3 (one eighth of the least nominal dimension, taken on the thickness); No. 9 or smaller is the 402-13 strength design limit (9.3.3.1(a))
Vertical bar spacing	7.3.2.3.1 / 7.3.2.5 / 7.3.2.6(a): 120 in. (ordinary), 48 in. (intermediate), the least of Lw/3, h/3 and 48 in. (special)
End bars	7.3.2.3.1: 0.2 in.² or more, within 8 in. of the end of the wall
Horizontal reinforcement	Ordinary and intermediate walls (7.3.2.3.1): 0.2 in.² at 120 in. or less, or two W1.7 joint reinforcement wires at 16 in. or less. Special walls (7.3.2.6): bond-beam bars hooked around the vertical bars, at the least of Lw/3, h/3 and 48 in., ρv and ρh ≥ 0.0007, ρv + ρh ≥ 0.002 (gross section), vertical area ≥ 1/3 of the required shear reinforcement
Wall type and seismic design category	7.4 / ASCE 7-22 Table 12.2-1: SDC D, E and F require special reinforced walls
Mortar in SDC D, E and F	7.4.4.2.2: Type S or M (partially grouted: cement-lime or mortar cement)

The key results list the net area, the bar layout and the horizontal reinforcement. The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the wall elevation with the vertical bars where the calculation places them, the bond-beam bars or joint reinforcement at their spacing, the P and V arrows at the top (values by load type in the sheet notes) and the bar spacing and wall dimensions; a plan section at the wall end (detail) with the grouted cells or the full grout, the end bars and the first distributed bar, the end cover and the thickness; and the φPn–φMn interaction diagram (not to scale) with the (Mu, Pu) point of every combination and the governing one called out. CMU webs, bar splices, foundation dowels, the hooks of the shear bars and the anchorage at the top are not part of the calculation and are not drawn.

Limitations

Not covered:

- Allowable stress design (TMS 402-22 Chapter 8).
- Out-of-plane loading, including the slender wall procedure (9.3.4).

- The special boundary element alternative to the maximum reinforcement (9.3.5.6.2): a note only.
- Unreinforced (plain) walls, clay masonry, openings, flanged or intersecting walls, masonry not laid in running bond.
- Anchorage, development and splices, grout strength (f'g ≥ f'm is assumed), the bar to grout space limits (one third of the least dimension, 4 % of the gross grout space, 6.1.3) and the horizontal reinforcement required at the top of the wall, at floor and roof levels and at openings.
- Joint reinforcement as shear reinforcement (9.3.3.7 allows it with 3/16 in. wires): it is not counted.
- The overstrength factor Ω0: enter the seismic effect you need (Emh = Ω0QE) as E. The ASD exception of 2.4.5 (0.9D for special reinforced walls in combination 10) does not apply: the wall is designed by strength design.

Simplifications:

- One bar per cell.
- Running bond.
- Each grouted cell is taken as 8 in. long, and webs are neglected in a partially grouted wall (as the TMS 402-13 commentary to its 9.3.3.5 does).
- The bar layout is symmetric.

Where this calculation is the earlier rule. Bar size: 6.1.3 has three rules for both design methods. Here only the first, and the 402-13 No. 9 limit.

Checked against TMS 402-13 on 2026-11-01. These were confirmed in TMS 402-13 / 602-13 and, as far as the TMS cross-reference and handouts show, are unchanged in the 2022 edition apart from the section number:

- Es = 29,000 ksi (4.2.2.1);
- εmu = 0.0025 and the 0.80 f'm over 0.80c stress block (9.3.2);
- φ = 0.80 for shear (9.1.4.5);
- the axial strength equations, with tied bars only (9.3.3.1.1, 9.3.2(e));
- the shear equations and the 6 / 4 Anv √f'm limits (9.3.3.1.2);
- the prescriptive reinforcement of 7.3.2.3.1 to 7.3.2.6 and the 1.25 Mn rule (7.3.2.6.1.1);
- the wall types per SDC (7.4);
- the unit strength table (TMS 602-13 Table 2).

The check corrected:

- the 1.25 Mn requirement, now the lesser of the two values;
- f'm above 4,000 psi, now capped instead of rejected;
- the α of squat walls (Mu/(Vu dv) < 1), now 1.5;
- the boundary element screen, now on Ag with all the conditions of 9.3.5.6.2.1.

It also added two checks: the end bar within 8 in. of the end, and the mortar in SDC D to F.

Still to verify in TMS 402-22 / 602-22 (the text is not on hand; marked (to confirm against the code text) in the code and the check references):

- the section numbers, which come from TMS's cross-reference, and the 402-22 equation numbers, confirmed only for Eq. 9-15;
- the α rule, the R ≤ 1.5 exemption and the axial load of the maximum reinforcement at 9.3.5.6.1;
- shear friction at 9.3.5.5: the formulas are from TMS's 2022 handout; the μ conditions, what counts in Asp and φ = 0.80 are not confirmed;
- the 4,000 psi upper limit of f'm;

- that TMS 602-22 Table 2 kept the 2013 values;
- the ASTM C90 face-shell thicknesses, which TMS 402 does not give;
- the ordinary wall height limit of ASCE 7-22 Table 12.2-1;
- the NCMA wall weights.

Verification examples

Verification status: 75 values compared in 20 examples (9 published, 11 hand calculations from the cited equations): 64 within tolerance, 11 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published examples (golden tests in `backend/tests/test_verify_masonry_wall.py`):

- **FEMA P-1051** (NEHRP Recommended Seismic Provisions: Design Examples, 2015 edition, Chapter 13, TMS 402-13), Section 13.2.4.5: Wall D of the five-story building in SDC B. 8 in. CMU partially grouted, Lw = 32 ft 8 in., story height 8 ft 8 in., f'm = 2,000 psi, #4 bars in the end cells and at 8 ft, 0.07 in.²/ft of horizontal steel (entered as #3 at 18.86 in.), Pu = 0.86D. Free from FEMA.
- **The Masonry Society**, webinar handout *Design of Masonry Shear Walls* (R. Bennett, 2018, TMS 402-16), Examples SD-1 to SD-3: a 2 ft pier and a 16 ft × 10 ft partially grouted wall, SDS = 0.5, wall weight 45 psf. Free from TMS.

Example	What it checks	Published	ILX Calcs
FEMA-13.2-W1	Story 1 shear (Table 13.2-3): An, Mu/(Vu dv), Vns, Vn max governs, φVn	1,185 in.², 0.960, 68.3, 164, 131 kips	1,185, 0.960, 68.6, 152.4, 121.9 (γg = 0.70)
FEMA-13.2-W5	Story 5 shear: Mu/(Vu dv), Vnm, (Vnm + Vns) γg governs, Vn max, φVn	0.266, 196, 198, 237, 158 kips	0.266, 196.1, 185.3, 221.0, 148.3 (γg = 0.70)
FEMA-13.2-PM0	Flexure at P = 0 (Case 1): c, Mn, φMn	0.41 ft, 968, 871 kip-ft	0.41, 968.1, 871.3
FEMA-13.2-PM2	Flexure at Pn = 337 kips (Case 2, c = 8 ft)	c = 8.00 ft, Mn = 5,371, φMn = 4,834 kip-ft	8.02, 5,375, 4,838
FEMA-13.2-DUC	Maximum reinforcement, α = 1.5 (Mu/(Vu dv) < 1): c, ΣC − ΣT	14.44 ft, 704 − 23.9 = 680.4 kips	14.43, 679.4
TMS-SD-1	Pier, Mu/(Vu dv) = 2 taken as 1.0, φVnm	2, 16.7 kips (its own expression gives 16.33)	2.0, 16.33
TMS-SD-2	Ordinary wall, #4 at 48 in.: Pu = 0.8D, Anv, Mu/(Vu dv), φVn max, φVn	18.6, 685 in.², 0.625, 91.9, 56.2 kips	18.6, 685, 0.625, 85.8, 52.5 (γg = 0.70)
TMS-SD-3	Special wall, #4 at 32 in., #5 at 24 in.: Anv, φVn (Vn max), required φVn = 2.0 Vu, ρ vertical / horizontal / total, c	767 in.², 102.9, 100 kips, 0.00096 / 0.00169 / 0.00267, 83.8 in.	767, 96.0 (γg = 0.70), 100, 0.00096 / 0.00169 / 0.00265, 83.9
TMS-2017-ORD	Ordinary wall, μ = 1.0: Asp, Vnf at Mu/(Vu dv) ≤ 0.5, φVnf (the handout interpolates to the 402-16 0.42 f'm Anc, ILX to the 402-22 form)	2.17 in.², 148.6, 104.1 kips	2.17, 148.6, 104.2

Documented deviation TMS402-22-GG-WALL: these examples predate 402-22 and use γg = 0.75; ILX uses 0.70, so its partially grouted shear values are 6.7 % lower. FEMA's maximum reinforcement check (FEMA-13.2-DUC) is on an ordinary wall, where 402-22 deleted the limit: it is entered as intermediate, which reaches the same α = 1.5 rule. TMS SD-3 now fails the special wall shear for 1.25 Mn at 1.04, from the same γg.

Documented deviation (`docs/verification/deviations.json`, TMS-SD-EQT): in SD-2 and SD-3 the handout draws the interaction diagram and checks the maximum reinforcement with an equivalent (averaged) thickness of the partially grouted wall. ILX takes the face shells plus the grouted cells, as the FEMA example does. The handout's 0.9 Pn at the maximum-reinforcement strain is 323 / 364 kips (ILX 354 / 393 kips), and SD-2 is stressed to 94 % of its flexural design strength (ILX 89 %).

The hand calculations in closed form (MW-1 to MW-5) check the remaining paths. The base case (MW-1) is:

- 8 in. CMU, f'm = 2,000 psi, Lw = h = 8 ft;
- one #5 bar 4 in. from each end;
- #4 bond beams at 48 in.;
- E: V = 20 kips at the top.

MW-5 evaluates the closed-form maximum reinforcement equation printed in the TMS 402-13 commentary (9.3.3.5, p. C-139).

Example	What it checks	Hand calculation	ILX Calcs
MW-1	Pu = 0. The neutral axis falls inside the first bar, which is elastic in tension: $9.76c^2 + 3.875c - 89.9 = 0$	c = 2.843 in., Mn = 143.0 kip-ft, φMn = 128.7 kip-ft	2.843, 143.0, 128.7
MW-1	Shear at Mu/(Vu dv) = 1.0	Vnm = 73.7 kips, Vns = 12.0 kips, φVn = 68.5 kips	73.7, 12.0, 68.5
MW-1	Axial strength, bars untied	Pn = 846.0 kips	846.0
MW-1	Maximum reinforcement, α = 3	Cm + Cs − T = 257.8 kips	257.8
MW-2	Partially grouted	An = 322 in.², φVn = 24.8 kips (γg = 0.70), Pn = 388.6 kips	322, 24.8, 388.6
MW-3	D = 50 kips, 0.9D + 1.0E: Pn = 50 kips, bar near the compression end neglected	c = 7.03 in., Mn = 326.5 kip-ft	7.03, 326.5
MW-4	M entered, Mu/(Vu dv) = 0.25, Pu = 36 kips, no shear reinforcement	Vn = 125.6 kips	125.6
MW-5	Special wall, α = 4, #5 end bars 8 in. from each end (d = 88 in., d′ = 8 in.); TMS 402-13 commentary equation with an elastic compression bar (44.1 ksi)	c = 20.42 in., axial load at the maximum = 194.3 kips	20.42, 194.3
MW-6	4 ft wall, Pn = 250 kips above Pbal = 216.4 kips: φ = 0.65	Pu = 162.5 kips, c = 27.05 in., fs = 45.4 ksi	27.05
MW-7	Same wall, εt = εty + 0.0015: φ = 0.775 solved with Pn = Pu/φ	c = 18.12 in., Pn = 158.3 kips	0.775, 18.12
MW-8	Same wall: 9.1.4.4.2 and pure axial	0.65 Pbal = 140.7 kips; φPn at φ = 0.65	140.7
MW-9, MW-10, MW-11	Shear friction at Mu/(Vu dv) of 0.25, 1.0 and 0.75 (μ and Asp entered)	51.2, 18.1, 44.6 kips	51.2, 18.1, 44.6

Not yet compared with a published example: the axial strength of 9.3.3.1.1 (402-13 Eqs. 9-19 / 9-20) on its own, and a published TMS 402-22 example. The examples above use earlier editions (402-13 and 402-16); the φ, γg and maximum reinforcement rules of 402-22 are hand calculations from TMS's 2022 handouts.

CHAPTER 140

Masonry column and pilaster

A reinforced concrete masonry (CMU) column or pilaster, fully grouted, designed by strength design to TMS 402-22 Chapter 9 with the column requirements of Chapter 5, the seismic tie rules of Chapter 7 and ASCE 7-22 2.3.1 load combinations. The calculation checks the axial strength with its slenderness reduction, axial load with biaxial bending by strain compatibility (the neutral axis rotated to the direction of the applied moment) with the strength reduction factor set by the strain, shear in each direction, the reinforcement limits and the lateral ties.

The calculation is by strength design only, so the LRFD / ASD switch is not shown. Allowable stress design (TMS 402-22 Chapter 8) is not offered; a calculation saved with ASD is designed by strength design.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers below are 402-22 numbers taken from TMS's own 402-16 to 402-22 cross-reference, and the changes of the 2022 edition come from the TMS webinar handouts on it. The wording of the provisions was read in TMS 402-13 / 602-13 (code and commentary), so an equation number is written as a 402-13 number unless it is marked otherwise. What remains unverified is listed under the limitations.

Inputs of the masonry column

- **Section:**
  - Column or pilaster. A column is an isolated member (TMS 402 2.2), and the column limits of 5.4.1 apply. A pilaster is built into a wall; it is designed as the rectangle alone, with the wall flanges neglected (conservative, as FEMA P-1051 Section 13.1.6 does), and the column limits are not applied.
  - The section from nominal unit sizes (8 to 32 in., the actual size is the nominal less 3/8 in.) or entered as actual dimensions. b is the width along x and t the depth along y; Mx bends about x (the depth t resists it).
  - The clear height between supports (self-weight and the drawing) and the effective height for bending about each axis. The effective height is your input: TMS 402 Commentary Figure CC-5.4-1 (number inferred) takes the clear height for a member braced at both ends and twice the height for a cantilever, and allows the distance between points of inflection only where data show reliable restraint (1.3).
- **Materials:** f'm entered or by the unit strength method (TMS 602 Table 2); at least 1,500 psi, and a value above 4,000 psi is used as 4,000 psi (4.3, Table 4.3.1). The mortar sets the modulus of rupture of the moment magnifier (Table 9.1.9.1, fully grouted), the column of the unit strength table and the SDC D to F mortar rule. Grade 60 or Grade 40 bars.
- **Reinforcement:**
  - The vertical bar size and the bars per face along b and along t (corner bars included): 2 and 2 give 4 bars, 3 and 3 give 8 bars. The face to bar centre distance (3.81 in. is the centre of the cell of an 8 in. unit).
  - The least dimension and the area of the gross grout space (the cell holding a bar), and the bars per cell, for the limits of 6.1.3. Take them from the units you specify.
  - The exposure (masonry cover, 6.1.5.1).
  - The lateral ties: 1/4 in. wire, #3 or #4, their spacing, and whether they are in grout or in the mortar joints. Wire ties are taken at 60 ksi (A1064 wire, limited by 9.1.9.2.1); bar ties at the bar grade.
  - Whether the ties count as shear reinforcement (Vns with the two tie legs): your choice.

    - A pilaster only: whether the compression bars are laterally tied. Untied bars are neglected in compression (9.3.2(e)) and the tie rules are not applied.

- **Loads at the critical section:** one row per load type (D, L, Lr, S, R, W, E): the axial compression P, the largest first-order moments Mx and My along the height, and the shears Vx and Vy. The symmetric section resists either sign; W and E act in both directions. SDS adds Ev = 0.2SDS D to the seismic combinations. The optional self-weight adds the whole column weight to D at the critical section (conservative).
- **Second-order moments** (TMS 402 9.3.3 bases member forces on an analysis and has no column moment magnifier of its own):
    - Moment magnifier: the procedure of the wall provisions, 9.3.4.4.3 (402-13 Eqs. 9-31 to 9-33), applied about each axis: ψ = 1/(1 − Pu/Pe), Pe = π² Em Ieff / h², Ieff = 0.75 In below the cracking moment (fr of Table 9.1.9.1) and Icr above it (Eq. 9-28 with the bars of the tension face, the axial load not credited). ψ multiplies the largest first-order moment, with no Cm reduction (conservative). Pu ≥ Pe is unstable.
    - Entered: the moments come from your second-order analysis (a note says so).
    - TMS strength design has no minimum eccentricity: the 0.80 factor of the axial strength covers it (402-13 Eq. 9-19 and its commentary).

- **Seismic detailing:** the seismic design category, whether the column is part of the seismic-force-resisting system and whether it supports discontinuous stiff elements.

Auto-design (editor toolbar) varies the nominal section (width and depth), vertical bars (size and bars per face) and ties (size and spacing). Objectives: lowest cost index (units, grout, reinforcement); least reinforcement. Every other input is kept; a section entered by its actual dimensions is kept as entered; see Auto-design.

Checks performed

Check	Basis
Axial strength	9.3.3.1.1 (402-13 Eqs. 9-19 / 9-20): Pn = 0.80 [0.80 f'm (An − Ast) + fy Ast] × (1 − (h/140r)²), or (70r/h)² above h/r = 99, with the axis of the larger slenderness (r = side/√12); Ast counted when tied; φ = 0.65, pure axial compression is compression controlled (Table 9.1.4)
Axial load and flexure (P-M, uniaxial or biaxial)	9.3.2 strain compatibility: εmu = 0.0025, 0.80 f'm over a = 0.80c measured perpendicular to the neutral axis, Es = 29,000 ksi, every bar at Es ε up to fy (compression bars when tied). At Pn = Pu/φ the neutral axis is rotated until the resisting moment points along the applied one (no Bresler or load contour approximation); Mu = √(Mux² + Muy²) against φMn; φ by Table 9.1.4 at that point (below)
Strength reduction factor φ	9.1.4.4, Table 9.1.4, from the net tensile strain εt = εmu (dt − c)/c in the bar farthest from the compression face and εty = fy/Es: compression controlled εt ≤ εty, φ = 0.65; tension controlled εt ≥ εty + 0.003, φ = 0.90; transition, φ = 0.65 + 0.25 (εt − εty)/0.003. φ depends on the strain at Pn = Pu/φ, so it is solved with the neutral axis, by an iteration with a cap (a note reports a point that does not converge, which then takes φ = 0.65 and Mn at Pu/0.90). The steps show φ, εt and the classification
Axial limit, tension-controlled and transition regions	9.1.4.4.2: φPn ≤ 0.65 Pbal, Pbal at εt = εty with εmu at the compression face, about the neutral axis direction of the point. A larger Pu is designed on the compression-controlled branch (φ = 0.65, Pn above Pbal), as the TMS handout's chart draws it
Shear along x and along y	9.3.3.1.2: Vnm = [4.0 − 1.75 Mu/(Vu dv)] An √f'm + 0.25 Pu, Vns = 0.5 (Av/s) fy dv with the two tie legs (when counted), Vn ≤ 6 to 4 An √f'm; dv = the side along the shear; φ = 0.80 (9.1.4.5)
Bar size	6.1.3: db ≤ least nominal dimension / 8 and ≤ 1/3 of the least dimension of the gross grout space; bar area in a cell ≤ 4 % of the gross grout space (8 % at laps; splices are not checked here). The bar choices stop at No. 11 (402-13 6.1.2.1, mapped to 402-22 6.1.3.2.1)

Check	Basis
Bar spacing and cover	6.1.4.2: clear distance between vertical bars ≥ 1.5 db and 1.5 in.; 6.1.5.1: masonry cover to the ties 2 in. (bars larger than No. 5 exposed to earth or weather) or 1.5 in.
Column dimensions (columns)	5.4.1.1: h/r ≤ 99 about each axis; least side ≥ 8 in. nominal
Column reinforcement (columns)	5.4.1.3: 0.0025 An ≤ Ast ≤ 0.04 An; at least four bars
Lateral ties (columns, tied pilasters)	5.4.1.4: diameter ≥ 1/4 in.; spacing ≤ 16 db, 48 tie diameters and the least dimension; in a mortar joint, diameter ≤ half the 3/8 in. joint (Commentary 5.4.1.4); bar support by tie corners or crossties (a note: corner and alternate bars, 6 in. clear); first and last tie within half a spacing (a note)
Seismic ties	SDC C and above, participating: anchor bolts at the top inside two No. 4 ties (7.4.3.2.1, a note); supporting discontinuous stiff elements: ties at ≤ 1/4 of the least nominal dimension and ρ ≥ 0.0015 (7.4.3.2.5). SDC D and above, participating: ties ≥ 3/8 in. at ≤ 8 in., in grout (7.4.4.2.1), Type S or M mortar (7.4.4.2.2)

Maximum reinforcement. TMS 402-22 deletes the maximum flexural reinforcement check for columns and pilasters, and the check is gone from this calculation along with its R ≤ 1.5 input (a saved calculation that has it still opens). The strength reduction factor of Table 9.1.4 takes its place: heavily reinforced sections are penalised by φ.

The key results list the section, f'm, the net area, the bars (Ast and ρ), φPn and the φ of the governing point. The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the column section with the bars, the tie, the moment axes, the bar positions and the overall dimensions; one φPn–φMn interaction diagram (not to scale), with φ by Table 9.1.4 and the 9.1.4.4.2 limit, about x (solid) and, for an unsymmetric section, about y (dashed), with the Mux and Muy of every combination and the governing one at its resultant moment; and the column elevation over the clear height with the bars, the ties at their spacing (the first at half a spacing), the axial loads and the effective heights. Crossties, unit joints and webs, splices, dowels, anchor bolts and the bearing at the base are not drawn.

Limitations

Not covered:

- Allowable stress design (TMS 402-22 Chapter 8) and lightly loaded columns (5.4.2).
- Bearing of the column on its support, anchor bolts, splices and development of the bars, grout strength (f'g ≥ f'm), the grout space between bar and unit (6.1.4.5).
- Partially grouted columns (TMS 402 requires columns fully grouted), clay masonry, bundled bars, bars not around the perimeter.
- The flanges of a pilaster (neglected, conservative) and the wall it supports.
- The moment gradient along the height: the moments are the largest along the height, magnified without a Cm reduction.
- The overstrength factor Ω0: enter Emh = Ω0QE as E where ASCE 7-22 12.4.3 requires it.

Simplifications:

- The section is the full rectangle (fully grouted), the bars do not displace masonry in the stress block, as the TMS examples do.
- The bars are at one face to bar centre distance on all four faces, equally spaced along each face.
- The moment magnifier is the wall procedure of 9.3.4.4.3 applied to each axis, with the axial load not credited in Icr and Mcr (both permitted, not required).

How the 2022 edition was checked. The 2022 code text is not on hand. The strength reduction factor, the 9.1.4.4.2 limit, the deleted maximum reinforcement, Eq. 9-28 and the bar size rules come from TMS's own handouts on the 2022 changes. The section numbers come from TMS's cross-reference. The wording of these provisions was read in the 402-13 edition (page numbers of that edition) and is taken as unchanged because neither source lists a change: column limits (402-22 5.4.1; 402-13 5.3.1, pp. C-61 to C-62), bar spacing and cover (6.1.4.2, 6.1.5.1; 402-13 6.1.3.2, 6.1.4.1, pp. C-67 to C-68), seismic ties (7.4.3.2.1, 7.4.3.2.5, 7.4.4.2.1, pp. C-92 to C-94), design assumptions (9.3.2, p. C-134), axial and shear strength (9.3.3.1; 402-13 9.3.4.1, pp. C-140 to C-141) and the moment magnifier (9.3.4.4.3 to 9.3.4.4.5; 402-13 9.3.5.4.3 to 9.3.5.4.5, pp. C-145 to C-146).

Still to verify in TMS 402-22 / 602-22 (marked (to confirm against the code text) in the code and the check references):

- the wording of every provision: the section numbers come from the cross-reference, not from the code, and the 2022 equation numbers other than Eq. 9-28 are not confirmed;
- whether 402-22 added a moment magnifier or a second-order provision for columns;
- that No. 9 is no longer a strength design limit: the handouts say the bar size rules are now the same for both design methods and list the three rules above, and the No. 11 ceiling is carried over from 402-13 6.1.2.1 through the cross-reference;
- the number of Commentary Figure CC-5.4-1 (inferred from 402-13 CC-5.3-1) and the net shear area of Table 4.4.5 (for a fully grouted column it is taken as the net area);
- the 135° or 180° hooks of the ties in SDC D and above (402-16 and later);
- the definition of the transverse reinforcement ratio of 7.4.3.2.5 (taken as two tie legs over the tie spacing times the larger side);
- the 4,000 psi upper limit of f'm.

Verification examples

Verification status: 43 values compared in 11 examples (5 published, 6 hand calculations from the cited equations): 40 within tolerance, 3 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published examples (golden tests in `backend/tests/test_verify_masonry_column.py`):

- **The Masonry Society**, Strength Design of Masonry night school, session 5 *Design of Columns and Pilasters & System Behavior* (J. Hochwalt, March 2021, TMS 402-16), column example: 20 ft, 16 in. × 16 in. CMU column, f'm = 2,000 psi, (4) #6 centered in the cells (d = 11.81 in.). Free from TMS.
- **FEMA P-1051** (NEHRP Recommended Seismic Provisions: Design Examples, 2015 edition, TMS 402-13), Section 13.1.6: the 16 in. square pilaster with (4) #6 bars and #3 ties (d = 12 in.) spanning 28 ft under out-of-plane wind, flanges neglected. Free from FEMA.
- **The Masonry Society**, webinar handout *Design of Masonry Shear Walls* (R. Bennett, 2018, TMS 402-16), Example SD-1: the shear of a 2 ft pier, entered as a 24 in. × 7.625 in. section bending about y. Free from TMS.

Example	What it checks	Published	ILX Calcs
TMS21-COL-PN	r, Pn (h/r = 53.2), φPn	4.51 in., 337.7, 303.9 kips (φ = 0.90)	4.51, 337.7, 219.5 (φ = 0.65, deviation)
TMS21-COL-BAL	Balanced point: Pn, c, Mn, φMn	102.6 kips, c at εy, 82.62, 74.36 kip-ft (φ = 0.90)	102.6, 6.46 in., 82.60, 53.7 (φ = 0.65, deviation)

Example	What it checks	Published	ILX Calcs
TMS21-COL-FLEX	Flexure at P = 0 with both bar layers: Mn, φMn (tension controlled, φ = 0.90 in both)	48.8, 43.9 kip-ft	48.75, 43.87
FEMA-13.1.6	Mu, φMn, In, Mcr, c, Icr	39.89 kip-ft, 520,059 lb-in., 4,967 in.⁴, 103,633 lb-in., 2.64 in., 1,337 in.⁴	39.89, 44.32 kip-ft (deviation), 4,967, 103,633, 2.64, 1,338
TMS-SD-1-COL	Mu/(Vu dv), φVnm (no shear reinforcement)	2.0, 16.7 kips (its own expression gives 16.33)	2.0, 16.33

Documented deviations (`docs/verification/deviations.json`):

- TMS402-22-PHI-COL: the TMS handout is written to 402-16 with a flat φ = 0.90 for axial load and flexure. Its nominal values match; its φ-values at pure axial load and at the balanced point differ because 402-22 Table 9.1.4 gives 0.65 there.
- FEMA-PIL-2L: the FEMA example takes only the two tension bars (φMn = 520,059 lb-in. = 43.3 kip-ft). ILX solves the strain compatibility with all four bars; the second layer is in tension, and φMn = 44.3 kip-ft (2 % higher). The TMS column example makes the same comparison (47.3 kip-ft with two bars, 48.8 kip-ft with both), and ILX matches its both-layer value.

Hand calculations in closed form check the remaining paths:

Example	What it checks	Hand calculation	ILX Calcs
MC-1	45° biaxial bending of the TMS column at Pn = 0: corner triangle stress block (area a², centroid a√2/3 from the faces), four bars at their depth below the compressed corner; c from equilibrium; the far bar is 16.70 in. deep, so the point is in transition	c = 7.18 in., Mn = √2 Mnx = 47.95 kip-ft, εt = 0.00332, φ = 0.754, φMn = 36.14 kip-ft	7.18, 47.95, 0.00332, 36.14
MC-2	Moment magnifier of the FEMA pilaster with D = 20 kips and Mx = 20 kip-ft (1.4D): Icr, Pe, ψ, Mu	1,338 in.⁴, 210.5 kips, 1.153, 32.30 kip-ft	1,338, 210.5, 1.153, 32.30
MC-3	Shear along y with 1/4 in. wire ties at 8 in., 0.9D + 1.0W: Mu/(Vu dv), Vnm, Vns, φVn	0.768, 38.00, 5.74, 34.99 kips	0.768, 38.00, 5.74, 34.99
MC-5	Compression-controlled point of the TMS column, Pu = 100 kips, φ = 0.65: c from the quadratic of equilibrium, εt, Mn, φMn	7.747 in., 0.00131, 82.80, 53.82 kip-ft	the same
MC-6	Transition point of the TMS column, Pu = 40 kips: φ, Pn, c, εt and Mn at the fixed point	φ = 0.806, 49.64 kips, 4.586 in., 0.00394, 66.85 kip-ft, φMn = 53.87 kip-ft	the same
MC-7	The 9.1.4.4.2 limit of the TMS column: cb, Pbal, 0.65 Pbal; Pu = 66 kips is below it and in transition, 67 kips is above it and takes φ = 0.65	6.462 in., 102.58, 66.67 kips	the same

The behaviour tests also compare the stress block of a rotated neutral axis with a fine numerical integration of the section (within 0.3 %). Not yet compared with a published example: a biaxial column (no free published TMS example was found), the seismic tie rules, and a TMS 402-22 example (the TMS / NCMA Masonry Designers' Guide and the TMS *Strength Design of Masonry* book are not free; the free handouts are written to 402-16 and use the old φ).

CHAPTER 141

Masonry lintel and beam

A reinforced concrete masonry (CMU) lintel or beam over an opening, designed by strength design (Sections 5.3 and 9.3, ASCE 7-22 2.3.1) or allowable stress design (Chapter 8, ASCE 7-22 2.4.1) to TMS 402-22. The lintel is a fully grouted simple span between its bearings. The calculation builds the load the lintel carries (its own weight, the wall above, floor and roof loads, point loads), with or without arching action, then checks flexure, shear, the reinforcement limits, the deflection, the bearings and the anchorage of the bottom bars.

Allowable stress design is the LRFD / ASD switch. It takes the cracked transformed section with Fs of 32 ksi (Grade 60) or 20 ksi (Grade 40) and 0.45 f'm in flexure, the shear of Chapter 8, and bearing at 0.25 f'm. The strength design rules for maximum reinforcement and for 1.3 Mcr become notes. **It rests on TMS 402-13 Chapter 8, not on 402-22,** checked against CMHA TEK 17-01D's tables (2011 MSJC, the same equations); section numbers are marked to verify. A calculation saved with ASD before this option was designed by strength design and is now designed by allowable stress.

Edition. The design basis is TMS 402-22 / TMS 602-22, but the code text is not on hand. Section numbers come from TMS's own 2016 to 2022 cross-reference list. The provisions that changed come from TMS's 2022 webinar handouts and the free Masonry Designers' Guide 2022 sample (Example 12.4-2A). The rest was checked against TMS 402-13 / 602-13 on 2026-11-02 and carried over. An equation number appears as a 402-22 number only where TMS confirms it; otherwise it reads "402-13 Eq.". What remains unconfirmed is listed under the limitations.

Arching action is your judgment. Whether the masonry above arches over the opening depends on conditions the calculation cannot see (control joints, openings, the masonry beside the opening). It is a checkbox, off by default, with the usual conditions in its hint and in a note. The calculation checks the two it can: running bond and enough masonry above the lintel.

Inputs of the masonry lintel

- **Lintel:**
  - The nominal width: 6, 8, 10 or 12 in. units (specified widths 5.625, 7.625, 9.625 and 11.625 in.).
  - The number of courses, 1 to 8 (8 in. each). The lintel is designed fully grouted. The actual depth is 8 in. per course less a 3/8 in. joint (15.625 in. for two courses), or you enter it (no more than 8 in. per course).
  - The bond: running bond, or stack bond (not running bond). It sets the modulus of rupture and arching needs running bond.
- **Materials:**
  - f'm, entered or by the unit strength method (TMS 602-22 Table 2). At least 1,500 psi; above 4,000 psi the value used for nominal strengths is 4,000 psi (Table 4.3.1).
  - The mortar type (M or S, N) and the mortar cement (portland cement / lime or mortar cement; masonry cement or air-entrained). Together with the bond they give the modulus of rupture of Table 9.1.9.1 (tension parallel to the bed joints, fully grouted): 267, 200, 160 or 100 psi in running bond, 335 psi for the continuous grout section in stack bond.
  - Grade 60 or 40 bars; coarse or fine grout (1/2 or 1/4 in. of grout between the bars and the units, 6.1.4.5).

- **Reinforcement:**
  - The bottom bar size (#4 to #9), 1 or 2 bars per course, in 1 or 2 courses (the second course 8 in. above the first). Two bars in a course sit against the grout clearance at each side of the grout space.
  - The distance from the soffit to the centre of the lowest bars (bars on chairs in the lintel unit, usually 2 to 4 in.); d follows from it.
  - Optional top bars (for uplift), the same distance below the top.
  - Optional stirrups: single bars with 180° hooks (9.3.3.2.3(a)), #3 to #5, at a spacing of 2 in. or more.
  - The embedment of the bottom bars past each face of the opening (the bearing plus the extension into the grouted course over the jamb), for the development check.
  - Whether the soffit is exposed to earth or weather (cover, 6.1.5.1).
- **Span and bearings:**
  - The clear span (the opening) and the bearing length at each end. The span is the clear span plus half the required bearing length at each end (5.3.1.1, as the Masonry Designers' Guide 2022 states it); the lintel is analysed as a simple span between the bearing centres. The bearing length you enter is taken as the required one, so enter what the lintel needs, not all the masonry beside the opening.
  - The f'm of the supporting masonry (0 = the lintel's) for the bearing check.
  - The spacing of the lateral supports of the top (compression face), 0 when it is braced continuously (5.3.1.3).
- **Loads** (D, L, Lr, S, R, W, E):
  - The lintel self-weight: the weight of the fully grouted masonry per ft² of elevation (NCMA TEK 14-13B, TEK 17-1D Table 1) times the nominal depth.
  - The masonry above the lintel: its height (top of the lintel to the top of the wall) and weight (psf).
  - Arching action develops (checkbox). On: the wall load is the 45° triangle over the effective span, apex at half the span (NCMA TEK 17-1D); loads applied at or above the apex and point loads outside the triangle are not on the lintel (the steps and a note list them). Off: the full height of masonry above is a uniform load and every superimposed load is on the lintel.
  - Uniform loads applied within the triangle (when arching): applied in full, as without arching (TEK 17-1D Figure 4, philosophy 1, the default), or the part within the triangle, w (ℓ − 2y)/ℓ, dispersed at 45° over the span (philosophy 2). TMS 402 does not address it.
  - Uniform loads (plf) with the height above the lintel where they are applied (blank = the top of the wall).
  - Point loads (lb) with the position from the left face of the opening, the height above the lintel (blank = the top of the wall) and the bearing length along the wall. With dispersion on (default), a point load spreads at 2 vertical : 1 horizontal (5.1.1.1) over its bearing plus its height and is applied as a uniform load P / (bearing
    - height), the part beyond a support going to it directly (TEK 17-1D Figure 3). Off: it acts over its bearing, or as a concentrated load.
- **Code options** (the engineer's choices):
  - The shear design section: d/2 from the face of the support, the face (your choice), or the centre of the bearing. 5.3.1.5 allows d/2 only where (a) the support reaction compresses the end regions of the beam and (b) no concentrated load lies between the face and d/2. Otherwise it requires the maximum shear, and the calculation uses the shear at the reaction. Condition (a) is a checkbox, on by default (true for a lintel bearing on the masonry below; turn it off for a beam hung from or framed into its support). Condition (b) is read from the point loads, and a dispersed point load counts by its position. A note states which applied.
  - Mu/(Vu dv) for shear: taken as 1.0 (the TMS 402 commentary permits it for beams; conservative) or computed at the design section (positive, at most 1.0).

- The deflection limit (span / n, 600 prefilled for beams supporting unreinforced masonry, 4.6; 0 = not checked) and whether the 8d exemption of 5.3.1.6.1 applies.

The depths in shear are not an option: TMS 402-22 settles them. The net shear area uses d (Table 4.4.5). dv stays the actual depth of the member, in Mu/(Vu dv), Vns and the stirrup rules, as the Masonry Designers' Guide 2022 example does (a saved value of the earlier shear depth option is ignored).

Auto-design (editor toolbar) varies the number of courses, the bottom bar size, the bars per course, the courses with bars and the stirrups (size and spacing). Objectives: lowest cost index (units, grout, reinforcement); shallowest; least reinforcement. The width, span, bearings, loads, the arching assumption and the code options are kept; see Auto-design.

Checks performed

Check	Basis
Flexure (positive moment)	9.3.2: εmu = 0.0025, 0.80 f'm over a = 0.80c, bars at Es ε up to fy (strain compatibility with two bar courses); one course: Mn = As fy (d − a/2), a = As fy / (0.80 f'm b); φ from the net tensile strain (Table 9.1.4), 0.90 for a tension-controlled section
Flexure under uplift	the same with the top bars; fails when a combination lifts the lintel and there are no top bars
Beam tension controlled (maximum reinforcement)	9.3.3.2.4 and Table 9.1.4: net tensile strain in the extreme tension bars εt ≥ εty + 0.003, εty = fy/Es (0.002 permitted for Grade 60, 9.1.4.4.1). One layer of Grade 60 bars: ρ ≤ 0.00356 f'm (ksi), 0.00711 at 2 ksi. The steps give As,max; P = 0; compression bars neglected. A failure names the reason, and φ drops with the strain (down to 0.65)
Minimum flexural reinforcement	9.3.3.2.2.2: Mn ≥ 1.3 Mcr, Mcr = fr b h²/6; waived where As ≥ 4/3 of the area required (9.3.3.2.2.3)
Shear	9.3.3.1.2: Vn = Vnm + Vns ≤ (6 to 4) Anv √f'm (Eq. 9-15, γg = 1.0), Vnm = [4.0 − 1.75 Mu/(Vu dv)] Anv √f'm, Vns = 0.5 (Av/s) fy dv, Anv = b d (Table 4.4.5), dv the actual depth, Pu = 0; φ = 0.80, at the design section (5.3.1.5, or your choice)
Transverse reinforcement	9.3.3.2.3: required where Vu > φVnm; Av/s ≥ 0.0007 b; spacing ≤ min(dv/2, 48 in.); single bars hooked 180° around the longitudinal bars, the first within dv/4 of the end (a note)
Bar sizes in a beam	9.3.3.2.2.1: top and bottom bars within one bar size
Deflection	4.6 and 5.3.1.6: Ieff = In (Mcr/Ma)³ + Icr [1 − (Mcr/Ma)³] ≤ In (5.3.1.6.2, 402-13 Eq. 5-1, at midspan), Icr of the cracked transformed section, Em = 900 f'm, Ma the largest moment of the ASCE 7-22 2.4.1 gravity combinations; against your limit; a note where the span is 8d or less (5.3.1.6.1)
Lateral support	5.3.1.3: spacing ≤ min(32b, 120b²/d)
Nominal depth	at least 8 in. (beam dimensional limit, section to verify)
Bearing length	5.3.1.4: at least 4 in. at each end
Bearing strength	9.1.8 and 4.4.4: φ 0.80 f'm Abr, φ = 0.60, Abr = A1 (the bearing reaches the edge of the masonry at the opening, so A2 = A1)
Reinforcement size	6.1.3, the same three rules for both design methods: diameter ≤ 1/8 of the least nominal member dimension (width or depth) and ≤ 1/3 of the least dimension of the gross grout space; bars of a course ≤ 4 % of the gross grout space (8 % at laps, not modelled)
Bars fit	6.1.4.1 and 6.1.4.5: grout clearance, stirrups, bars and the clear spacing max(db, 1 in.) fit the clear width

Check	Basis
Cover	6.1.5.1: 2 in. for bars larger than #5 when exposed to earth or weather, 1-1/2 in. otherwise (side and soffit)
Development	6.1.6.3.1 (402-13 Eq. 9-16): ld = 0.13 db² fy γ / (K √f'm) ≥ 12 in., K the least of the cover, the clear spacing and 9db; the embedment past the face of the opening must reach ld (bars developed for fy at the face, as the TMS Strength Design Guide does)
Arching	NCMA TEK 17-1D: masonry above ≥ ℓ/2 + 8 in. and running bond; a note lists the conditions you confirm
Deep beam (not feasible)	5.3.2: effective span (the lesser of centre to centre and 1.15 ℓn) / depth < 2

The key results list the lintel, the effective span, the load model, Mu / φMn, Vu / φVn and the factored reactions. The load steps show the effective span and every part of the load model with its value (and the loads not on the lintel). The drawings (on screen, in the report and on the PDF / DXF sheet) are the lintel loading (the loads on the lintel by type over the effective span, as the calculation applies them), the opening elevation (jambs, bearings, the lintel courses with the bars and their embedment, the stirrups, the wall above and the arching triangle when assumed), the lintel section (bars, stirrup, d and h) and the shear and moment diagrams of the governing combination. Unit shapes, splices, jamb reinforcement and the masonry below the bearings are not drawn.

Limitations

Not covered:

- Deep beams (5.3.2): NOT FEASIBLE with a hint. The commentary allows a shallower lintel that carries the masonry above as load; use fewer courses.
- Continuous or fixed-end beams, a lintel that is part of a continuous bond beam (designed here as a simple span), cantilevers.
- Partially grouted beams, clay masonry, AAC. TMS 402-22 allows partially grouted beams, but a beam needs full grout where shear reinforcement is required; this calculation designs full grout only.
- Allowable stress design (Chapter 8).
- Axial load on the beam (9.3.3.2.1: a note), torsion, the development of the top bars, the grout strength (f'g ≥ f'm is assumed), the masonry supporting the jambs.
- The end limits of the point load dispersion of 5.1.1.1 (half the wall height, a movement joint, an opening, the spacing of the loads): a note asks you to check them.

Simplifications:

- The bearing area is the bearing length times the full width (A2 = A1).
- Two bars in a course are placed against the grout clearance; the stirrup is a single leg at the centre.
- The grout space of a course is the clear width (width less two face shells) times 7.625 in. less a bottom shell taken as the face-shell thickness (lintel unit geometry varies by manufacturer).
- The deflection is the immediate deflection; creep is not added (the commentary notes it).

Unverified against the code text. TMS 402-22 itself is not on hand. The section numbers come from TMS's cross-reference list, the 5.3.1.1 span rule and the depth used in Vns from the Masonry Designers' Guide example, the rest of the changed provisions from TMS's 2022 handouts. The cross-reference leaves these open, marked (to confirm

against the code text) in the check references: the bearing section 4.4.4 (its row is blank, the number is inferred), the subsections of 5.3.1.1 (marked "Deleted?"), the section of the 8 in. nominal depth limit, φ = 0.60 for bearing, and the subsection of 6.1.3 for each size rule. The deep beam definition is carried over from 402-13.

Verification examples

Verification status: 116 values compared in 29 examples (23 published, 6 hand calculations from the cited equations): 106 within tolerance, 10 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published examples (golden tests in `backend/tests/test_verify_masonry_lintel.py`):

- **The Masonry Society**, *Masonry Designers' Guide 2022* (TMS 402/602-22), free sample, Example 12.4-2A: a 16 ft opening, 8 in. CMU lintel 48 in. deep, and its rework with heavier loads. It is the only published example on hand written to the 2022 code.
- **The Masonry Society**, webinar handout *Strength Design of Masonry* (October 2017, 402-16 numbering), beam example: a 10 ft opening, 24 in. deep.
- **The Masonry Society**, webinar handout *Design of Beams* (R. Bennett, February 2021, TMS 402-16), which works the TMS Strength Design of Masonry guide's Examples 6.3.1.8 and 6.3.1.9: a 16 ft opening, 8 in. CMU beam 48 in. deep, f'm = 2,000 psi, Type S PCL mortar, 8 in. bearings, beam weight 81 psf. Free from TMS.
- **NCMA (CMHA) TEK 17-01D**, *ASD of Concrete Masonry Lintels*, design example: 12 in. CMU, 5 ft 4 in. opening, 4 in. bearings, 10 ft of masonry above, 1,000 plf at the top of the wall; Case 1 with arching, Case 2 a 24 in. lintel without. Only the loads, moments and shears are compared (the TEK's capacities are allowable stress design). Free from CMHA.

Example	What it checks	Published	ILX Calcs
MDG-12.4-2A	D = 700, Lr = 300 plf, 2 No. 4 (d = 44 in.): span, Vu, Mu, As required, Mcr, 1.3 Mcr, Mn, ρmax, 32b, 120b²/d, ℓ/d	16.67 ft, 14.25 kips, 713 kip-in., 0.305 in.², 781.8 / 1,016 / 1,038 kip-in. (printing error), 0.00711, 20.3 / 13.2 ft, 4.5	16.67, 14.24, 711.8, 0.305, 781.8 / 1,016 / 1,032, 0.00711, 20.3 / 13.2, 4.55
MDG-12.4-2A-5	The same with 1 No. 5: Mn below 1.3 Mcr	804 kip-in.	804.2 (minimum reinforcement fails)
MDG-12.4-2A-R	D = 2,000, L = 700 plf, 4 No. 4 in two courses, No. 3 at 8 in.: Mu, As required, φMn, ρ, ρmax, Vu at d/2, Anv, Vnm, Av/s min, dv/4, spacing limit, f'm to avoid stirrups	1,630 kip-in., 0.793 in.², 1,640, 0.00262, 0.00711, 24.76 kips, 305 in.², 30.7 kips, 0.0053, 12 in., 24 in., 2,035 psi	1,628, 0.792, 1,643, 0.00262, 0.00711, 24.76, 305, 30.7, 0.0053, 12, 24, 2,033
TMS-2017-BEAM	10 ft opening, 24 in. deep, 2 #6, Type S masonry cement mortar: span, Mu, As required, Mn, φMn, Mcr, 1.3 Mcr, ρ, 32b, 120b²/d, Vu at d/2, φVnm, φVn max, Vns required	10.67 ft, 62.6 kip-ft, 0.77 in.², 78.5, 70.6, 9.76, 12.7 kip-ft, 0.00577, 20.3 / 29.1 ft, 18.33, 12.27, 21.82, 7.58 kips	10.67, 62.55, 0.767, 78.48, 70.63, 9.76, 12.69, 0.00577, 20.3 / 29.07, 18.33, 12.28, 21.82, 7.57
SDG-6.3.1.8	D = 500, L = 450 plf, 2 #4 (d = 44 in.): span, Vu, Mu, As required, Mcr, 1.3 Mcr, Mn, ρ, 32b, 120b²/d	16.67 ft, 14.24 kips, 712 kip-in., 0.305 in.², 782 / 1,016 / 1,032 kip-in., 0.00119, 20.3 / 13.2 ft	16.67, 14.24, 711.8, 0.305, 781.8 / 1,016 / 1,032, 0.00119, 20.3 / 13.2
SDG-6.3.1.8-5	The same with 1 #5: Mn below 1.3 Mcr	804 kip-in.	804.2 (minimum reinforcement fails)

Example	What it checks	Published	ILX Calcs
SDG-6.3.1.9	D = 2,000, L = 700 plf, 4 #4, #3 at 8 in.: Vu, Anv, Vnm, φVn max, Vns required, s required, Av/s min, spacing limit, dv/4	32.6 kips, 305 in.², 30.7, 43.6, 10.1 kips, 15.7 in., 0.0053, 24 in., 12 in.	32.6, 305, 30.7, 43.6, 10.0, 15.8, 0.0053, 24, 12
SDG-6.3.1.9	Deflection: n, k, kd, In, Icr, Ma, Ieff, deflection, ℓ/600	16.11, 0.252, 10.06 in., 70,270 / 14,140 in.⁴, 1,259 kip-in., 27,590 in.⁴, 0.106 in., 0.333 in.	16.11, 0.252, 10.06, 70,272 / 14,141, 1,260, 27,559, 0.106, 0.333
TEK-17-1D-1	Arching: span, masonry needed above, lintel weight, Vlintel	5.67 ft, 3.5 ft, 88 plf, 251 lb	5.67, 3.5, 88, 249
TEK-17-1D-2	No arching, 24 in. lintel: lintel weight, total load, Vmax	264, 1,264 plf, 3,602 lb	264, 1,264, 3,581

Documented deviations (`docs/verification/deviations.json`):

- **MDG-12.4-2A-PRINT**: printing errors in Example 12.4-2A. For 2 No. 4 it prints ρ = 0.00092 (that is 1 No. 5; 2 No. 4 give 0.00119). In the rework it prints 1,640,000 lb-in. as Mn (by hand Mn = 1,826,000 lb-in.; 1,643,000 is φMn). The Mn of 2 No. 4 is printed as 1,038,000 lb-in. (by hand 1,032,000, within the test tolerance), and the page writes 0.007 b for 0.0007 b.
- **TMS-RHOMAX-16**: Example 6.3.1.8 uses the 402-16 limit ρmax = 0.00952; 402-22 requires a tension-controlled beam, 0.00711 at f'm = 2 ksi.
- **TMS-2017-VNS-D**: the 2017 handout takes d = 20 in. in Vns and asks for Av = 0.101 in.²; the 2021 and 2022 TMS examples use the actual depth, which gives 0.084 in.² here.
- **TEK17-1D-L57**: the TEK computes its moments with the span rounded to 5.7 ft (Mlintel 357 lb-ft, Case 2 Mmax 61,601 lb-in.); ILX uses 5.67 ft (353 lb-ft, 60,866 lb-in.).
- **TEK17-1D-TRI**: the TEK text takes the 45° triangle (height half the effective span) but its example multiplies the wall weight by 3.5 ft (half the span plus 8 in.): 221 plf at the apex, Mwall 598, Mmax 955 lb-ft, Vmax 566 lb. ILX follows the text: 179 plf, 478, 831 lb-ft, 502 lb.

The TMS examples of 2021 and 2022 take Anv = b d and dv = the overall depth, the rule TMS now states, so the spacing limit of 24 in. that an earlier version of this calculation recorded as a difference now matches.

The hand calculations in closed form (ML-H1 to ML-H6) check the remaining paths:

Example	What it checks	Hand calculation	ILX Calcs
ML-H1	Default lintel (8 in., 2 courses, 1 #5 at 3 in., 6 ft opening): Vnm = 2.25 b d √f'm, φBn, ld (K = cover 2.69 in.), Mcr (267 psi), Mn	9.69 kips, 58.6 kips, 25.4 in., 6.90 kip-ft, 18.4 kip-ft	9.69, 58.6, 25.4, 6.90, 18.4
ML-H2	Arching, 10 ft opening: span, triangle peak (52 psf), point load of 3,000 lb 2 ft above dispersed over 2.33 ft, its moment, masonry needed	10.67 ft, 277.3 plf, 1,285.7 plf, 6,874 lb-ft, 6.0 ft	10.67, 277.3, 1,285.7, 6,874, 6.0
ML-H3	Stack bond: Mcr with fr = 335 psi	8.66 kip-ft	8.66
ML-H4	Tension controlled, Grade 60: ρmax = 0.64 f'm / fy × εmu / (εmu + 0.002 + 0.003), φ	0.00711, 0.90	0.00711, 0.90
ML-H5	Tension controlled, Grade 40: εty = fy/Es	0.01163	0.01163
ML-H6	2 #9 in one course, f'm = 1,500 psi: εt from the elastic quadratic, φ of Table 9.1.4 (compression controlled)	0.00110, 0.65	0.00110, 0.65

Not on disk and not free (research/DOCUMENTS_NEEDED.md): TMS 402-22 / 602-22, the TMS *Strength Design of Masonry* guide (its lintel and beam examples beyond the webinar slides) and the full *Masonry Designers' Guide* (TMS 402/602-22).

CHAPTER 142

Masonry wall out-of-plane

A reinforced concrete masonry (CMU) wall that spans vertically between lateral supports and carries axial load and out-of-plane wind or seismic pressure, designed by strength design to TMS 402-22 Section 9.3.4 (wall design for out-of-plane loads). The design section is the mid-height of a one-foot strip of wall. The moment includes the P-delta effect by either procedure of 9.3.4.4, and the service deflection is limited to 0.007h (9.3.4.5). Load combinations follow ASCE 7-22: 2.3.1 and 2.3.6 for strength, 2.4.1 and 2.4.5 for the deflection.

The calculation is by strength design only, so the LRFD / ASD switch is not shown. Allowable stress design (TMS 402-22 Chapter 8) is not offered. A calculation saved with ASD is designed by strength design.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers below are 402-22 numbers taken from TMS's own 402-16 to 402-22 cross-reference, and the changes of the 2022 edition come from the TMS webinar handouts on it. The wording of the provisions was read in TMS 402-13 / 602-13 (code and commentary), so an equation number is written as a 402-13 number unless it is Eq. 9-28, the one confirmed for this chapter. What remains unverified is listed under the limitations.

Inputs of the out-of-plane wall

- **Wall:**
  - The nominal thickness: 6, 8, 10 or 12 in. units (specified 5.625, 7.625, 9.625, 11.625 in.; face shells 1.0 in. for 6 in. units and 1.25 in. for the others, the ASTM C90 minimums).
  - The wall height h between the lateral supports. The wall is simply supported at the top support and at the base, the case 9.3.4.4.2 is written for.
  - An optional parapet above the top support. Its weight adds to the axial load at mid-height. Its lateral load is ignored for the mid-height moment unless you tick **Parapet lateral load reduces Mu**: then its cantilever moment at the top support, $w\ hp^2/2$, reduces the mid-height moment by $w\ hp^2/4$. Ignoring it is common practice (FEMA P-1051 13.1.5.2.5) and conservative for the wall; the parapet itself is not designed here.
  - The grout: fully grouted, or grouted only at the reinforced cells.
- **Vertical reinforcement:**
  - The bar size and spacing s. One bar per cell, so s is at least 8 in.; multiples of 8 in. fit the units.
  - The bar position: one bar centred (d = tsp/2), one bar at a depth d from the bearing face, or one bar at each face at a given distance from the face. The bearing face is the face the eccentric axial load leans toward (positive e). Pressure that bends the wall the same way as the eccentric load puts the far face in tension, so the bar depth is d; pressure the other way uses tsp − d. Bars in compression are neglected (not laterally tied, 9.3.2).
  - For a partially grouted wall, the grouted width per grouted cell: the cell with its webs and grout, taken full depth between the face shells. FEMA P-1051 uses 8 in.; the CMHA CMU-TEC-002-23 tables for 8 in. units match 8.2 in.
- **Materials:** f'm entered or by the unit strength method (TMS 602 Table 2); Grade 60 or 40 bars; the mortar type and cement for the modulus of rupture (Table 9.1.9.1). For a partially grouted wall the table says to interpolate

between hollow and grouted masonry. You choose the basis: by net area, as FEMA P-1051 13.1.5.2.5 does, or by the fraction of cells grouted, its simpler alternative.

- **Section properties:** computed, or entered per foot from the NCMA / CMHA tables (An, In, Sn). Computed: the two face shells over the full length plus the grouted width per grouted cell, full depth. FEMA P-1051 enters the TEK 14-1B values.
- **Loads** (one row per load type D, L, Lr, S, R, W, E):
  - P is an axial line load at the top (plf), with its eccentricity e from the wall centre line (positive toward the bearing face). The eccentric moment at mid-height is Σ P e / 2.
  - w is the out-of-plane pressure over the height (psf). W and E act in both directions. For seismic, enter E = 0.4 SDS Ie times the wall weight (ASCE 7-22 12.11.1), at least 0.1 times the wall weight.
  - The wall weight (psf) is dead load. The weight above the mid-height section counts: Pw = w (h/2 + parapet).
  - SDS adds Ev = 0.2SDS D to the seismic combinations (ASCE 7-22 12.4.2.2). It is 0 by default; a note says so while E is present.
- **Design options:**
  - The P-delta procedure: the iterative mid-height deflection of 9.3.4.4.2 or the moment magnifier of 9.3.4.4.3. The same procedure computes the service deflection (9.3.4.5.1 or 9.3.4.5.2).
  - The axial load used in Mcr and Icr: the combination's own axial load (default), the unfactored dead load (FEMA P-1051), or none. The code permits the axial load in both (9.3.4.4.4 commentary, 9.3.4.4.5).
  - The seismic design category, optional: it adds the prescriptive vertical bar spacing of a nonparticipating wall (7.4.3.1 / 7.4.4.1) as an advisory.

All values are per foot of wall, and the steps give the per-bar values where useful.

Auto-design (editor toolbar) varies the nominal thickness, the grout (at the bars or full) and the vertical bars (size #4 to #8 and spacing 8 to 120 in.). Objectives: lowest cost index (units, grout, reinforcement); least reinforcement. The bar position, f'm, the height, the loads (including the wall weight you entered) and the procedure are kept; update the wall weight for a new thickness or grout. See Auto-design.

Checks performed

Check	Basis
Axial stress limit of the slender wall procedure	9.3.4.4.2 (402-13 Eq. 9-26): Pu/Ag ≤ 0.20 f'm at mid-height (Ag = 12 in. × tsp), and ≤ 0.05 f'm where h/t (nominal thickness) exceeds 30. A strength check with the iterative procedure; an advisory with the moment magnifier, since the limit is written in 9.3.4.4.2
Axial strength	9.3.4.2 → 9.3.3.1.1 (402-13 Eqs. 9-19 / 9-20): Pn = 0.80 (0.80 f'm An) (1 − (h/140r)²), or (70r/h)² above h/r = 99, r = √(In/An); no bars in compression; φ = 0.65, pure axial compression is compression controlled (Table 9.1.4)
Out-of-plane flexure with axial load and P-delta	9.3.4.2 / 9.3.2: the stress block 0.80 f'm over a = 0.80c on the face shell, or on the face shell plus the grouted cells when a passes the face shell (T-section); the bar stress from the strain (yields in practice); Mn = T (d − ȳ) + Pu (tsp/2 − ȳ), which is As fy (d − a/2) + Pu (tsp − a)/2 in the face shell (402-13 commentary). Effective compressive width per bar min(s, 6t, 72 in.) (5.1.2). Mu at mid-height by 9.3.4.4.2 (402-13 Eqs. 9-27 to 9-30, iterated until the deflection changes by less than 0.1 %) or 9.3.4.4.3 (402-13 Eqs. 9-31 to 9-33: Ieff = 0.75 In below Mcr, Icr above; Pu/Pe below 1). Mcr = Sn (fr + P/An) (9.3.4.4.4); Icr by Eq. 9-28, Icr = n As (d − c)² + n (P/fy)(tsp/2 − c)² + b c³/3, with the neutral axis in the face shell, else the same steel terms with the transformed section of the grouted cells (9.3.4.4.5). For a centred bar this equals the older form; for an off-centre bar the axial load now acts at the centre line. Every combination, both directions; φ by Table 9.1.4 (below)

Check	Basis
Strength reduction factor φ	9.1.4.4, Table 9.1.4, from the net tensile strain εt = εmu (d − c)/c at the design point Pn = Pu/φ, εty = fy/Es: compression controlled εt ≤ εty, φ = 0.65; tension controlled εt ≥ εty + 0.003, φ = 0.90; transition, φ = 0.65 + 0.25 (εt − εty)/0.003. φ is solved with c, by an iteration with a cap (a note reports a point that does not converge, which then takes φ = 0.65). The steps show φ, εt and the classification. Mn: compression controlled, at Pn = Pu/φ; otherwise the commentary form with Pu, which is conservative against Mn at Pu/φ below the balanced point (within about 8 % at the top of the tension-controlled range, and it leaves a narrow conservative step at the balanced point)
Axial limit, tension-controlled and transition regions	9.1.4.4.2: φPn ≤ 0.65 Pbal, Pbal at εt = εty with εmu at the compression face. A larger Pu is designed on the compression-controlled branch (φ = 0.65), as the TMS handout's chart draws it. The steps also give the largest Pu of a tension-controlled section (Commentary Table CC-9.1-1)
Out-of-plane shear	9.3.4.3 → 9.3.3.1.2: at the top support, Vu = wu h/2 + Puf e/h, Mu/(Vu dv) with Mu = Puf e and dv = tsp (between 0 and 1); Vnm = [4.0 − 1.75 Mu/(Vu dv)] An √f'm + 0.25 Pu, Vns = 0, Vn ≤ 6 to 4 An √f'm; γg = 1.0; φ = 0.80
Mid-height deflection (service)	9.3.4.5 (402-13 Eq. 9-36): δs ≤ 0.007h under the ASCE 7-22 2.4.1 / 2.4.5 combinations, P-delta by 9.3.4.5.1 (402-13 Eqs. 9-29 / 9-30 iterated with Mser) or 9.3.4.5.2 (first-order deflection times 1/(1 − P/Pe))
Bar size	6.1.3: diameter ≤ least nominal dimension / 8 and ≤ 1/3 of the least dimension of the gross grout space; bar area ≤ 4 % of the gross grout space (8 % at laps; splices are not checked here), the space taken as (tsp − 2 face shells) × 6.31 in., 1 in. webs. The bar choices stop at No. 11 (402-13 6.1.2.1, mapped to 402-22 6.1.3.2.1)
Bar placement	6.1.4.5: 1/4 in. of grout between bar and unit (fine grout); 6.1.4.1: clear distance between the two bars of the each-face layout ≥ db and 1 in.
Prescriptive vertical reinforcement (advisory)	7.4.3.1(b) / 7.4.4.1(b), when the seismic design category is given: a nonparticipating wall needs at least a No. 4 bar at 120 in. (SDC C) or 48 in. (SDC D to F)

Maximum reinforcement. TMS 402-22 deletes the maximum flexural reinforcement check for walls loaded out of plane, and the check is gone from this calculation. The strength reduction factor of Table 9.1.4 takes its place: heavily reinforced walls are penalised by φ.

The key results list the procedure, Mu with P-delta, φ at the design point, φMn, the factored deflection and the service deflection against 0.007h. The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are:

- the wall section: the wall from the base to the top of the parapet, the bars where the calculation puts them, the lateral supports, the eccentric axial load at the top with its eccentricity, the pressure band (values by load type, both ways) and the deflected shape of the governing combination (exaggerated) with δu;
- the moment diagram along the height for the governing combination (P-delta included; the deflected shape taken as a half sine for the diagram, which equals the code's value at mid-height), at its own vertical scale;
- a plan section of a length of wall: face shells, the grouted cells (or full grout), the bars at their spacing and the thickness.

The roof and floor framing, the anchorage of the wall, the foundation, splices and dowels, horizontal reinforcement and the parapet design are not part of the calculation and are not drawn.

Limitations

Not covered:

- Allowable stress design (Chapter 8).
- Support conditions other than simple supports top and bottom: a fixed base, a cantilever wall, a wall spanning two ways or horizontally between pilasters. 9.3.4.4.2 sends those to mechanics; use the engineer's own moments elsewhere.
- Concentrated loads and their distribution (5.1.1), openings, jambs and pilasters.
- In-plane loads (see the masonry shear wall calculation), unreinforced walls, clay masonry.
- The parapet as a cantilever, the support reactions and the wall anchorage (ASCE 7-22 12.11.2).
- Horizontal reinforcement, splices and development.

Simplifications:

- The mid-height section is the design section, as 9.3.4.4.2 states. With a large eccentricity and a small lateral load the largest moment may be elsewhere (402-13 commentary); the moment diagram shows it.
- Lateral loads uniform over the height; axial loads at the top support (floor loads at mid-height are entered as top loads, which is conservative for the eccentric moment).
- The out-of-plane shear uses γg = 1.0 and dv = tsp.
- Running bond.

Still to verify in TMS 402-22 (the code text is not on hand; marked (to confirm against the code text) in the steps where it bears on a step):

- the wording of every provision: the section numbers come from the cross-reference, not from the code, and the 2022 equation numbers other than Eq. 9-28 are not confirmed;
- that No. 9 is no longer a strength design limit: the handouts say the bar size rules are now the same for both design methods and list the three rules above, and the No. 11 ceiling is carried over from 402-13 6.1.2.1 through the cross-reference;
- whether the 402-22 commentary still writes Mn with Pu (402-13 Commentary 9.3.5.2) now that the design point is at Pu/φ;
- the axial stress limit (402-13 Eq. 9-26) and whether it covers the moment magnifier as well;
- the moment magnifier's Ieff rule (0.75 In / Icr);
- the modulus of rupture table, the net shear area out of plane (Table 4.4.5, taken as An here), the γg of out-of-plane shear and dv;
- the effective compressive width of 5.1.2 and the face-shell thicknesses (ASTM C90-22).

Verification examples

Verification status: 71 values compared in 29 examples (19 published, 10 hand calculations from the cited equations): 69 within tolerance, 2 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Golden tests in `backend/tests/test_verify_masonry_slender_wall.py`:

- **FEMA P-1051** (NEHRP Recommended Seismic Provisions: Design Examples, 2015 edition, Chapter 13, TMS 402-13), Section 13.1.5.2: the 28 ft side walls of the one-story warehouse. 8 in. CMU grouted at 24 in., #7 at 24 in. centred, f'm = 2,000 psi, 2 ft parapet, roof 200 plf at e = 7.32 in., wall 65 psf, seismic out-of-plane 37 psf, SDS

= 1.43 (0.614D and 1.486D). FEMA enters In = 355 in.⁴/ft, Sn = 93.2 in.³/ft and An = 51.3 in.²/ft from NCMA TEK 14-1B and uses 1.0D in Mcr and Icr; the test does the same. Free from FEMA.

- **CMHA CMU-TEC-002-23** (the successor of NCMA TEK 14-1B), Table 2A: net section properties per foot of 8 in. walls, face-shell bedded. Free from CMHA.
- **The Masonry Society**, webinar handout *Major Structural Design Changes in TMS 402/602-22* (R. Bennett, October 2022), slide 19: the largest axial load of a tension-controlled wall, f'm = 2 ksi, Grade 60. The one source here written to 402-22. Free from TMS.

Example	What it checks	Published	ILX Calcs
FEMA-13.1.5.2-IT	fr interpolated by net area	112 psi	111
FEMA-13.1.5.2-IT	Mcr = Sn (fr + P/An)	12,700 in.-lb/ft	12,632
FEMA-13.1.5.2-IT	Icr with P = 1.0D	42.1 in.⁴/ft	41.7
FEMA-13.1.5.2-IT	Mu by iteration, 0.614D / 1.486D	48,289 / 57,670 in.-lb/ft	48,404 / 58,167
FEMA-13.1.5.2-IT	Table 13.1-2: a, Mn for 0.614D / 1.486D	0.978 / 1.03 in., 62.3 / 65.35 in.-kip/ft	0.977 / 1.033, 62.36 / 65.40
FEMA-13.1.5.2-IT	φMn for 0.614D / 1.486D	56.1 / 58.8 in.-kip/ft (φ = 0.90)	56.13 (φ = 0.90) / 57.30 (φ = 0.876, deviation)
MAJOR-S19	Largest Pu of a tension-controlled wall, 10 rows of the slide: 8 in. #4 at 48 / #5 at 48 / #4 at 24 / #5 at 24 / #6 at 24 / #4 at 16 / #6 at 40 / #4 at 8, 12 in. #4 at 48 / #5 at 24	14.5, 13.2, 12.0, 9.0, 5.5, 9.3, 10.3, 1.2, 16.4, 15.0 kip/ft	14.52, 13.22, 12.01, 9.04, 5.53, 9.31, 10.28, 1.21, 16.34, 14.88
FEMA-13.1.5.2-MM	Moment magnifier, 1.486D: Pe, Mu	6.62 k/ft, 61.8 in.-kip/ft	6.56, 62.0
FEMA-13.1.5.2-DEF	Service deflection by the magnifier, limit 0.007h	3.85 in., 2.35 in.	3.87, 2.35
CMHA-2A	An, In, Sn: solid, grouted at 16, 24, 48, 96 in. (grouted width 8.2 in.)	e.g. 51.0 in.², 354.6 in.⁴, 93.0 in.³ per ft at 24 in.	within 0.2 % for all 15 values

FEMA iterates until the deflection changes by less than 5 % (1.2 % and 1.7 % at its last step); ILX iterates to 0.1 %, which raises Mu by 0.2 % (0.614D) and 0.9 % (1.486D). FEMA's Icr arithmetic gives 42.1 in.⁴/ft; Eq. 9-34 with its own numbers gives 41.7. The wall's bars are centred, so 402-22 Eq. 9-28 gives the same Icr as FEMA's equation and Mcr, Mu, Pe and the deflections do not change; only the φ of the 1.486D combination does. The ductility values of the example are no longer compared, because 402-22 deletes that check. Both FEMA and ILX find the 1.486D case governing. By the moment magnifier FEMA is about 5 % over its design strength and ILX about 8 % over.

Documented deviations (`docs/verification/deviations.json`):

- TMS402-22-PHI-OOP: FEMA uses the flat φ = 0.90 of 402-13. At Pn = Pu/φ the 1.486D case has εt = 0.00478, in the transition region of Table 9.1.4 (0.90 starts at 0.00507), so φ = 0.876 and φMn = 57.3 in.-kip/ft against 58.8. With the iterative Mu of 58.2 in.-kip/ft the wall that passes in FEMA is now 1.5 % over; with the moment magnifier it is over in both.

- ASCE7-EV-ASD: FEMA's service deflection uses D + 0.7E. ASCE 7-22 2.4.5 adds 0.7Ev = 0.14 SDS D, so with SDS = 1.43 the axial load is 1.2D and the deflection 4.09 in. against 3.85 in. The deflection example is also run with SDS = 0, which reproduces FEMA (3.87 in.).

Hand calculations in closed form (written out in the test file):

Example	What it checks	Hand calculation	ILX Calcs
HC-1	8 in. fully grouted, #5 at 16 in., h = 16 ft, D = 500 plf, W = 60 psf, 0.9D + 1.0W: Mcr with fr = 163 psi, Icr (Eq. 9-28, centred), converged δu and Mu (fixed point of Eq. 9-30), Mn	1,688 lb-ft/ft, 36.1 in.⁴/ft, 0.279 in., 1,944 lb-ft/ft, 4,271 lb-ft/ft	the same
HC-1M	HC-1 by the moment magnifier (Ieff = Icr)	Mu = 2,040 lb-ft/ft	2,040
HC-2	Partially grouted #7 at 16 in., block past the face shell: a = tfs + (C − C1)/(0.80 f'm bw), Mn = T (d − ȳ) + Pu (tsp/2 − ȳ)	1.624 in., 7,086 lb-ft/ft	the same
HC-3	12 in. wall, one #6 at 24 in. at d = 5.5 in.: Mn = As fy (d − a/2) + Pu (tsp − a)/2	6,237 lb-ft/ft	6,237
HC-4	HC-1 at 5 psf (uncracked): magnifier with 0.75 In, Pn (402-13 Eq. 9-19), Vn at Mu/(Vu dv) = 0	161 lb-ft/ft, 71,655 plf, 16,481 plf	the same
HC-5	HC-3 (bar off centre): Icr by Eq. 9-28 (402-13 Eq. 9-34 would give 84.8)	85.36 in.⁴/ft	85.36
HC-6	8 in. fully grouted, #5 at 16 in., h = 8 ft, 1.4D = 17.25 k/ft: compression controlled: c from the quadratic of equilibrium, εt, Mn at Pu/0.65, φMn; φPn pure axial	2.385 in., 0.00150, 8,726 and 5,672 lb-ft/ft, 68,740 plf	the same
HC-7	Same wall, 1.4D = 6.05 k/ft: transition: φ, Mn with Pu, φMn	0.848, 5,486 and 4,651 lb-ft/ft	the same
HC-8, HC-8A	8 in. grouted at 48 in., #4: 0.65 Pbal = 14,521 plf. Pu = 14.4 k/ft: tension controlled, φ = 0.90, φMn = 4,384 lb-ft/ft. Pu = 14.6 k/ft: φ = 0.65, Mn = 6,648, φMn = 4,321 lb-ft/ft. The chart on slide 17 of the TMS handout has this notch at about 14.5 kip/ft and 4.3 kip-ft/ft	the same	the same

Not yet compared with a published example: a complete TMS 402-22 out-of-plane example (the TMS / NCMA Masonry Designers' Guide is not on hand; the free handouts with full wall examples are written to 402-16 and use the old φ), a partially grouted wall whose stress block passes the face shell, and the iterative service deflection.

CHAPTER 143

Masonry pilaster with wall

A reinforced concrete masonry (CMU) pilaster built into a wall, by strength design to TMS 402-22 Chapter 9 with 5.5 and the flange rules of 5.1.1. The calculation follows the load from the wall to the pilaster: the wall pressure per foot of height, the moment and shear along the height, the wall as a flange where the code allows it, axial load and flexure in each direction of bending, second-order moments, shear, the transfer between flange and pilaster, deflection and the detailing of the bars.

The masonry column calculation also has a pilaster: there the moments are entered and the flanges neglected. Use that one for a load already worked out, and this one for the wall load path and the flange.

The calculation is by strength design only, so the LRFD / ASD switch is not shown. Allowable stress design is not offered.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers are 402-22 numbers from TMS's own cross-reference, the 2022 changes come from the TMS webinar handouts, and the wording was read in TMS 402-13 (5.1.1.2 p. C-47, 5.4 p. C-63). What remains unverified is listed under the limitations.

Inputs of the masonry pilaster

- **Pilaster and wall:** nominal width along the wall and overall depth across it (wall included); the wall thickness. The pilaster is flush with one face of the wall unless you enter a projection beyond the other face. Height, end conditions (pinned; fixed base, pinned top; cantilever) and effective height, which is your input: the height for pinned ends, twice it for a cantilever, lower only where data show reliable restraint.
- **Wall as a flange:** 5.5 allows flanges only in running bond (5.1.1.2.1) with the intersection connected by interlocking units, steel connectors or reinforced bond beams (5.1.1.2.5). You state both; otherwise the flanges are neglected. The wall beside the pilaster is the actual flange: to a movement joint, an opening, or half way to the next pilaster. The calculation limits it to 6 times the nominal wall thickness on each side (5.1.1.2.3). A hollow or partially grouted wall counts as its two face shells only.
- **Loads:** the tributary width X and the portion K give wp = K w X (NCMA TEK 14-03A), acting both ways. K is your input: 1.0 when the wall spans horizontally only, otherwise from TEK 14-03A Figure 1 or your analysis. Pressures W and E are at strength level. Axial loads act at the top, with an eccentricity from the pilaster centre, positive toward the projecting face. The wall's weight on the flanges is yours to add to the axial rows.
- **Reinforcement:** bars in a layer on the wall side and a layer on the projecting side, at one face to bar centre distance. Compression bars count only when tied (9.3.2(e)). The grout space inputs are for the limits of 6.1.3.
- **Deflection:** the service factors on W and E and the limit as height / n. 143 is 0.007 h, the wall limit, applied here by your choice.

Auto-design (editor toolbar) varies nominal section (width and depth), vertical bars (size and bars per layer) and ties (size and spacing). Objectives: lowest cost index (units, grout, reinforcement); least reinforcement. Every other input is kept.

Checks performed

Check	Basis
Axial strength	9.3.3.1.1 (402-13 Eqs. 9-19 / 9-20) on the net area of the pilaster and effective flange, r from that section, out of plane; bars counted when tied; φ = 0.65
Axial and flexure, each direction	9.3.2 strain compatibility: εmu = 0.0025, 0.80 f'm over a = 0.80c, Es = 29,000 ksi; the hollow cells of the flange are gaps; moments about the pilaster centre, at the levels of the largest moment and the base, for each combination with the wall pressure each way. φ by Table 9.1.4 at Pn = Pu/φ
Axial limit, tension-controlled and transition regions	9.1.4.4.2: φPn ≤ 0.65 Pbal
Second-order moments	9.3.4.4.3 (402-13 Eqs. 9-31 to 9-33) on the effective height, the wall procedure applied to the pilaster, as the TMS 2021 handout advises; Ieff = 0.75 In below Mcr and Icr above it, Icr with the flange in the compression zone; or not applied by your choice
Shear out of plane	9.3.3.1.2 on the pilaster rectangle alone; the largest moment of the combination with its largest shear (conservative); ties when counted; φ = 0.80
Shear between the flange and the pilaster	5.1.1.2.4: the force in the overhang (the flange block at Mn × Mu/Mn, on the wider side) over two face shells or the wall thickness, along the distance from the largest moment to zero moment; Vn = 2.25 Anv √f'm, the shear equation at Mu/(Vu dv) = 1.0 with no axial load or reinforcement. The form of this check is this calculation's choice
Deflection under the wall load	9.3.4.5 (402-13 Eqs. 9-29 / 9-30) for pinned ends; the elastic beam formula with In or, when cracked, Icr over the whole height for the other end conditions. Lateral load only
Bar size, spacing, cover	6.1.3, 6.1.4.2 (1.5 db and 1.5 in.), 6.1.5.1
Lateral ties (where bars are counted in compression)	5.4.1.4(a), (b), and the mortar joint rule

The key results list the wall load per foot, the section, the flanges counted, φPn and the largest factored reactions at the top and the base, to carry to the roof and footing design. The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the plan section with the effective flange, the interaction diagram for both directions of bending (not to scale) and the elevation with the wall load.

Limitations

Not covered:

- Allowable stress design (Chapter 8).
- The wall panel between pilasters and its horizontal reinforcement (masonry wall calculation); the K factor itself.
- Bending along the wall, in-plane forces, the seismic rules of Chapter 7.
- Bars in the flanges and a flange in flexural tension: 5.1.1.2.3(c) permits 0.75 times the height, but no bars in it are counted and the masonry takes no tension.
- Connections at the top and base, anchor bolts, splices and development, bearing (masonry bearing calculation), clay masonry.

Simplifications:

- Uncracked properties (In, Mcr) are those of the pilaster rectangle, flanges neglected: a lower cracking moment, conservative.
- Axial loads act at the top. The self-weight is the pilaster rectangle's and acts at its centre.
- The moment gradient is not used to reduce the magnifier. Deflection takes the lateral load only, not the eccentric axial load or P-delta.

Still to verify in TMS 402-22 / 602-22 (marked (to confirm against the code text) in the check references):

- the wording of 5.5 and 5.1.1.2: the 402-22 flange numbers are carried from 402-13 through the cross-reference;
- what 5.1.1.2.4 requires for the interface, and whether the shear equation at Mu/(Vu dv) = 1.0 is the intended form;
- the equation numbers other than Eq. 9-28, which is used here with the axial load not credited.

Verification examples

Verification status: 36 values compared in 9 examples (4 published, 5 hand calculations from the cited equations): 35 within tolerance, 1 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

All published examples are older than the 2022 code. None has a flanged masonry pilaster: **no free published flanged example was found**, so the flanged results rest on hand calculations, solved in the test by a separate equilibrium loop.

Example	Values compared	Published	ILX
TMS webinar Oct. 2017, pilaster, pp. 14-15 (402-16)	wp, Pu at the top, top moment, level of the largest moment, Mu, Pu at that level	0.416 kip/ft; 0.54 kips; 3.1 kip-in; 143.7 in.; 361 kip-in; 2.7 kips	Agree within 1 %. The example's required As = 0.57 in.² is not compared: this calculation checks given bars
NCMA TEK 14-03A pp. 2-3	wp, moment, shear, fixed base and pinned top	328 lb/ft; 5,900 lb-ft; 2,460 lb	Agree within 1 %. K = 0.91 read from the TEK's chart and entered
FEMA P-1051 13.1.6, pp. 13-22 to 13-24 (402-13)	wu, Mu, In, Mcr, c, Icr, service moment, deflection parts, total	0.407 kip/ft; 39,886 lb-ft; 4,967; 103,633 lb-in; 2.64; 1,337.1; 335,042 lb-in; 0.136 + 1.131 = 1.267 in.	Agree within 1 %
FEMA P-1051 13.1.6	φMn	520,059 lb-in	About 2 % higher: a recorded deviation. The example uses the two tension bars, and ILX solves both layers, the second also in tension
TMS night school 2021, column, pp. 10-11 (402-16)	r, Pn, Mn at Pn = 102.6 kips	4.51 in.; 337.7 kips; 82.62 kip-ft	Agree within 1 %. Nominal values: the published φ = 0.90 is 402-16; 402-22 Table 9.1.4 gives 0.65 there
Hand calculations	Flange in compression with the block in the face shell and past it; projecting face in compression; flange transfer; net area, r, Pn		Agree

A note on FEMA P-1051 13.1.6. The example takes 20 ft of wall for a pilaster with pilasters at 40 ft on center. By TEK 14-03A wp = K w X with X the center-to-center span, which would be 40 ft. The tributary width is your input here, and the 20 ft is entered as printed to compare the example. Enter the width your framing carries.

Not verified: any flanged pilaster against a published example, and any value written to the 2022 code. Masonry Designers' Guide 2022 examples 12.4-7 and 11.4-6 are paid.

CHAPTER 144

Masonry bearing

Bearing of a beam or other concentrated load on the top of a concrete masonry wall, by strength design to TMS 402-22: the bearing area under the loaded area (4.4.4), the 2:1 spread along the wall (5.1.1), bearing on the masonry under a bond beam and the nominal bearing strength Bn = 0.8 f'm Abr with φ = 0.60 (9.1.8). It also gives the length of wall that carries the load at half the height and the load per foot, to copy into the wall calculation.

The calculation is by strength design only, so the LRFD / ASD switch is not shown.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers come from TMS's cross-reference and the wording was read in TMS 402-13 (4.3.4 p. C-43, 5.1.3 pp. C-51 to C-52, 9.1.8 p. C-127).

Inputs of the masonry bearing

- **Wall:** thickness; the masonry under the load (a bond beam, a fully grouted wall, or hollow units on their face shells); for a bond beam, its courses and the wall under it (hollow, partially grouted with an equivalent solid thickness you enter, or fully grouted); running bond or not; f'm.
- **Loaded area:** length along the wall and width across it, offset from the wall centre line. The load is taken as concentric on this area: for a beam end that rotates or sits off centre, enter the part of the area concentric with the load. That is your judgment; the code gives no rule.
- **Along the wall:** the wall each side of the loaded area, to the nearest end, opening or movement joint; the height below the load; the spacing of concentrated loads.
- **Loads:** by load type, P the concentrated load and w a line load on the wall at the same level. w is added at the section under a bond beam, over the spread length; it does not bear on the loaded area.

Auto-design (editor toolbar) varies loaded length, loaded width and bond beam courses. Objective: the smallest loaded area, with a bond beam course counted as 40 in.². Every other input is kept.

Checks performed

Check	Basis
Bearing under the loaded area	9.1.8 Bn = 0.8 f'm Abr, φ = 0.60; 4.4.4 Abr = min(A1 √(A2/A1), 2 A1). A2 similar to and concentric with A1: its offset stops at the nearer wall face, the nearer end, opening, joint or half way to the next load, and at 45 degrees at the depth of the support. Not in running bond A2 = A1, since A2 stops at head joints whose position is not an input. Hollow units: Abr is the face shell under the area
Bearing on the wall under the bond beam	5.1.1 and Commentary Figure CC-5.1-5(a): L = lb + v/2 each side, each limited; Abr = net thickness × L; load = Pu + wu L; Bn as above
Length of wall at half the height (a note, not a capacity)	5.1.1: the spread ends at half the height. Not in running bond: no spreading across head joints

The wall's axial and flexural strength is not checked here. It depends on the wall's moments and slenderness, which are not inputs, and a bare axial check at this level would suggest a precision it does not have. The key results give the load per foot of wall: check the wall with it in the masonry wall calculation.

The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the wall elevation with the load, the bond beam courses and the 2:1 lines, and the plan on the wall with A1 and A2.

Limitations

Not covered: allowable stress design; clay masonry; the bearing plate or beam; anchorage and uplift; the wall's strength; the bond beam as a flexural member; a load pocketed into the side of a wall; grout strength ($f'_g \geq f'_m$).

Simplifications: the bond beam course is 8 in. nominal less the 3/8 in. joint. A2 in a wall not in running bond is taken as A1 (conservative). In partially grouted masonry the equivalent thickness averages grouted and hollow cells: your judgment if grouted cells sit directly under the load.

Still to verify in TMS 402-22 / 602-22 (marked (to confirm against the code text) in the check references): the section number 4.4.4 (the cross-reference row is blank and the number is inferred), the numbering of the bearing φ in 9.1.4, the number of Commentary Figure CC-5.1-5, and that the provisions are unchanged from 402-13.

Verification examples

Verification status: 25 values compared in 9 examples (0 published, 9 hand calculations from the cited equations): 25 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

No published numeric example of masonry bearing is available free. The Masonry Designers' Guide 2022 examples 7.5-1 to 7.5-3 are paid. Every value below is a hand calculation from the equations above, not a published result; they will be replaced or joined by those examples when the guide is on hand.

Hand calculation	Values
Plate narrower than the wall	k = 1.271, A_2 = 77.5 in.², A_{br} = 61.0 in.², φB_n = 58.6 kips
Loaded area across the full wall	A_{br} = A_1 = 61.0 in.², φB_n = 58.6 kips
Under one bond beam course on hollow masonry	L = 15.63 in., A_{br} = 39.1 in.², load 14.1 kips (1.2D + 1.0S), φB_n = 37.5 kips
Spread cut at a wall end	L = 13.81 in., k = 1.5 along the wall
Half the height	L = 80 in., 1.98 kip/ft
Hollow units	A_{br} = 40 in.², φB_n = 38.4 kips
Small area on a grouted wall	A_{br} = 2 A_1 = 16 in.²
Next load nearby	L = 12 in.
Not in running bond	A_{br} = A_1 = 48 in.², L stays at 15.63 in. below the bond beam

CHAPTER 145

Masonry anchor bolts

Cast-in headed and bent-bar anchor bolts in grouted concrete masonry, one bolt or a rectangular group, in the side of a wall or the top of a wall or bond beam, by strength design to TMS 402-22. It checks tension, shear and the two combined, and the placement limits. The drawings show the bolt layout with the projected areas, and a section through a bolt.

The calculation is by strength design only, so the LRFD / ASD switch is not shown.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. The equations were read in TMS 402-13 (6.2, 9.1.6), masonry crushing with 1,750 and the 5/3 interaction in the TMS 402-16 handout of March 2018, and steel from the specified tensile strength in the TMS 402-22 handout of October 2022. φ for steel from fu is not printed in anything on hand: 0.75 in tension is read back from Example 9.3-6a of the Masonry Designers' Guide 2022, and 0.65 in shear is an inference. Both are marked (to confirm against the code text) in the report.

Inputs of the masonry anchor bolts

- **Bolts:** side or top of a wall; headed or bent bar; diameter 1/2 to 1 in.; steel by ASTM grade or entered fu and fy; embedment from the masonry surface to the bearing surface of the head or of the bent end; bolts along the wall and rows, up to 6 × 6.
- **Masonry:** wall thickness, f'm, grout, the length and height of masonry to the free edges, openings or movement joints that stop the cones, and the distance of the first bolt from the left end and the near edge. In the top of a wall the near edge is the near face and the wall thickness closes the other side.
- **Open cells:** the part of each bolt's cone over open cells, cores or head joints is deducted (6.3.2, 6.3.3). The module has no cell layout: measure this area and enter it. Zero means the masonry is grouted over the whole cone.
- **Loads:** tension and shear on the group by load type, shared equally by the bolts. The direction of the shear sets the edge distance for shear breakout.
- **Offset shear:** a ledger's load acts away from the wall. Enter the offset and a moment arm and the tension from that moment is added. The arm is your judgment; TEK 12-03C takes 5/6 of the distance from the bolt to the edge of the ledger. An arm of 0 adds nothing.
- **Seismic anchorage:** on by default. Combinations with E take the factor (2.0) on each force whose strength is not governed by steel, as ASCE 7 requires for anchorage of seismic-force-resisting masonry to diaphragms and chords (to confirm against the code text: ASCE 7-22 14.4.5). IBC 1613 excludes ASCE 7 Chapter 14, and anchorage of nonstructural components takes the overstrength factor of 13.4.2.2 instead, so the choice is yours.

Auto-design (editor toolbar) varies bolt diameter, embedment and bolts along the wall. Objective: the least bolt steel. Spacing, edge distances, f'm and loads are kept; if more bolts do not fit the masonry, that candidate is dropped.

Checks performed

Check	Basis
Tension per bolt	9.1.6.3.1: the least of masonry breakout 4 Apt √f'm (φ = 0.50), steel Ab fu (Eq. 9-2, φ = 0.75 (to confirm against the code text)), and for a bent bar pullout 1.5 f'm eb db + 300 π (lb + eb + db) db (φ = 0.65)
Apt	6.3.2: a circle of radius lb, cut by the masonry edges; overlapping circles share their area so no masonry counts twice; a group of overlapping bolts shares the group's area equally (Commentary), the least group governs
Shear per bolt	9.1.6.3.2: the least of masonry breakout 4 Apv √f'm, crushing 1,750 (f'm Ab)^(1/4), pryout 8 Apt √f'm (all φ = 0.50), steel 0.6 Ab fu (Eq. 9-7, φ = 0.65 (to confirm against the code text))
Apv	6.3.3, 6.3.7: half a circle of radius lbe on the free edge the shear acts toward, each row of bolts with its own lbe, cut by the masonry at that edge; the row with the least area governs
Combined	9.1.6.3.3: (baf/φBan)^(5/3) + (bvf/φBvn)^(5/3) ≤ 1.0 (to confirm against the code text: 402-16 exponent, kept in 402-22)
Placement	6.3.1 clear distance at least the bolt diameter and 1 in.; 6.3.6 effective embedment at least 4 db and 2 in.
Grout around the bolt	Top of a wall: face shell + grout + half the head to each face. Side of a wall: embedment at most wall − face shell − grout − head (TMS handout, March 2018, from 6.3.1)

The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the layout on the masonry surface with the dashed projected areas, and a section at a bolt. Edge distances too long to fit in the view are in the sheet notes.

Limitations

- Strength design only; allowable stress design is planned with the other masonry ASD versions.
- Post-installed anchors and anchors found by test (9.1.6.2) are not covered.
- The forces are shared equally. Moment on the group, other than the offset shear, is not distributed: work out the tension on the worst bolt and enter it as an equivalent group force.
- Apv for several rows takes each row alone; the code gives no rule for rows behind each other.
- The ledger or plate and the wall under the bolt forces are not checked.

Verification examples

Verification status: 36 values compared in 12 examples (8 published, 4 hand calculations from the cited equations): 32 within tolerance, 4 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published examples: Masonry Designers' Guide 2022 free sample, Examples 9.3-6 and 9.3-6a (written to 402-22): Apt, breakout and the steel design strength. TMS webinar Anchor Bolt Design, March 2018 (402-16), pp. 22, 24-25, 29, 41-42, 51-54: bent bar, headed, near an edge, shear, combined with E. CMHA TEK 12-03C pp. 4-5 (2013 code), strength part.

Two differences are deliberate and recorded on the Verification page. The 2018 handout's steel values use a 36 ksi yield; 402-22 uses the specified tensile strength. TEK 12-03C's crushing uses 1,050, the 402-13 coefficient; 402-16 raised it to 1,750. TEK's page prints 90.00 in.² in its equations where its text gives 90.99 in.²; its results follow 90.99.

Hand calculations from the equations: shear breakout with overlapping half circles, the limit on fu, the seismic factor, a corner bolt. Anything that depends on a (to confirm against the code text) above (φ of the steel, the exponent, the crushing coefficient) waits for the 2022 code text.

CHAPTER 146

Masonry wall at openings

The strip of a reinforced concrete masonry wall beside one or two openings, spanning vertically out of plane and carrying the load of the openings, by strength design to TMS 402-22: flexure with axial load, the moment magnifier, axial strength, shear and service deflection. The wall away from the openings is the masonry wall out-of-plane calculation; the lintel over the opening is the masonry lintel.

The calculation is by strength design only, so the LRFD / ASD switch is not shown.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers come from TMS's cross-reference, the changes from the TMS handouts of 2022, and the wording was read in TMS 402-13. Only Eq. 9-28 is a confirmed 402-22 equation number here.

Inputs of the masonry wall at openings

- **Wall and openings:** the height between supports, simply supported top and bottom; a parapet; the strip width; the opening on each side, half of which loads the strip; sill and head.
- **Wind on the openings:** on by default. A door or window passes the wind to the masonry beside it, so wind loads the whole tributary width over the whole height. Seismic comes from the masonry's own mass, so beside the openings it loads the strip only (FEMA P-1051 13.1.6.2). Whether a given door or window passes the wind on is your judgment.
- **Parapet reduces the moment:** off by default. A parapet's load is added to the top reaction, but its cantilever moment is not taken off the moment in the span unless you say so.
- **Section:** thickness, face shell, grouting, f'm, bars per layer, one layer at the centre or one at each face. With a layer at each face, the layer near the compression face is not counted unless you ask for it, then only while it is in tension (bars not tied, 9.3.2(e)). The published examples leave it out.
- **Loads:** by load type, w the axial load per foot of wall over the whole tributary width at the top, e its distance from the wall centre line, p the out-of-plane pressure. The wall's weight is per wall area; openings weigh nothing.

Auto-design (editor toolbar) varies bar size, bars per layer and thickness. Objective: the least reinforcement, then the thinnest wall. Strip and openings are kept.

Checks performed

Check	Basis
Moments and reactions	Statics on the load pieces, simply supported. The moment from the eccentric load at the top is added in the direction of the lateral load
Flexure with axial load	9.3.2 strain compatibility about the wall centre line; φ of Table 9.1.4 at Pn = Pu/φ; checked at the largest moment and at the door head of every combination
Moment magnifier	9.3.4.4.3: ψ = 1/(1 − Pu/Pe), Ieff = 0.75 In below Mcr, otherwise Icr by Eq. 9-28 when the neutral axis is in the face shell or the wall is grouted

Check	Basis
Axial strength	9.3.3.1.1 with h/r; bars not counted, not tied
Axial limit	9.1.4.4.2
Shear	9.3.4.3, 9.3.3.1.2 with Mu/(Vu dv) = 1.0, axial load not counted, on the larger reaction
Service deflection	9.3.4.5: 0.007 h, ASD combinations; the curvature with the uncracked section to Mcr and the cracked beyond it, integrated along the height (for a uniform load this is the code's two-part equation), times ψ
Compressed width	5.1.2: per bar at most the spacing, 6 t and 72 in.
Bars	6.1.3, 6.1.4

The drawings are the wall elevation to the middle of each opening, and the plan section of the strip.

Limitations

- Strength design only.
- The strip's section is used over the whole height. Above and below an opening the wall is wider, which this ignores: conservative, by a margin the FEMA example shows at the largest moment (95.6 kip-ft on the bay "does not control").
- Openings at the same height on each side, one sill and one head. Different openings: use the worse, or enter the wider one on both sides and judge the result.
- Simply supported top and bottom. No fixed base.
- Lintel and its bearing, bond beams, splices and in-plane forces are separate calculations. Prescriptive seismic bars at openings are not checked (to confirm against the code text: TMS 402-22 7.4.3.1).

Verification examples

Verification status: 22 values compared in 5 examples (3 published, 2 hand calculations from the cited equations): 15 within tolerance, 7 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published examples, both of the same building: TMS webinar "SD Walls Axial and Out-of-Plane Loads", Feb. 2021, Guide Example 6.3.3.12, pp. 23-26 (402-16): reactions, first-order moment, Pu, ψ, Mu, Mn with one layer and with two, φMn, the 1.2D + Ev + Eh combination. FEMA P-1051 13.1.6.3, pp. 13-28 to 13-35 (402-13): axial load, Mn and φMn at the door head.

Differences recorded on the Verification page: Icr and so Pe (402-22 Eq. 9-28 against the older form); the deflection, where both published checks use the cracked section throughout and the handout puts the upper load over the whole height, as "quick checks"; a rounded load in FEMA (1,178 printed as 1,200 plf); FEMA's Mn at the door head adds Pu where the handout and ILX use Pu/φ, 1.5 % apart. FEMA's Mu at the door head comes from a P-delta frame run with the wider wall above, and is not compared with the magnifier as a verification value. FEMA's C = 184.1 kips is not its own T + P = 186.1 kips; its Mn is within tolerance of either.

Hand calculations: no opening equals w h²/8; an unloaded opening. All published examples are older than 402-22.

CHAPTER 147

Anchored veneer

Veneer ties and the checks around them for anchored veneer: the permitted design method, tie spacing or the force in the tie, the cavity and tie components, backing and height limits, and the out-of-plane stability of the backing. **TMS 402-22 itself is not on hand: this follows BIA Technical Note 3C (July 2026), written to the 2022 code.** Section numbers are marked to verify. BIA's note is for brick; concrete masonry veneer follows the same chapter, but its unit limits have not been compared.

Inputs of the anchored veneer

- **Design method.** Prescriptive Basic, Prescriptive Enhanced or Tributary Area. **The Modeling Analysis method is not offered:** a wall that needs it says so.
- **Load that governs, out-of-plane load, seismic design category.** The load is at strength level: wind from components and cladding, seismic from Fp.
- **Veneer.** Specified thickness, unit height and weight. Over 30 ft (38 ft at a gable) on wood or cold-formed steel, **detailed for differential movement** is your statement.
- **Backing, height hb, deflection.** The backing is designed elsewhere; you enter its deflection, under 0.42 of the strength wind load or the strength seismic load.
- **Veneer tie.** Type, wire size or pintle legs, spacing both ways. In the Tributary Area method, the stiffness and strength come from TN 3C Table 6 or from your test values. BIA asks for professional judgment on in-house data.
- **Cavity.** Drainage space, continuous insulation, sheathing. The sheathing counts unless it bears 100 psi or the ties reach the framing.

Auto-design varies the tie spacing both ways, fewest ties first. Tie type, method, cavity and backing stay yours.

Checks performed

Check	Basis (BIA TN 3C, reading TMS 402-22 Chapter 13)
Method permitted for the load and category, 75 % of the loads for corrugated ties	Table 1
Tributary area and spacing, Prescriptive	Table 2: 2.67 ft² and 24 in. Basic, 1.78 ft² and 16 in. Enhanced
Force in the tie, Tributary Area: 2, 2.5, 3 or 4 p At by stiffness	Tables 6, 7
Cavity width by tie and component; drainage at least 1 in.; BIA's 2 in. as a note	Tables 3, 4, p. 5
Component at most half the joint	p. 6
Tie type allowed for the backing	Tables 3, 4

Check	Basis (BIA TN 3C, reading TMS 402-22 Chapter 13)
Corrugated ties on wood at most 30 ft, 38 ft at a gable; movement statement otherwise	Table 5, p. 8
Backing deflection for stability, no interpolation	Table 10
Unit thickness, height and weight	p. 2, p. 8

Fasteners, edges, running bond and seismic detailing come out as notes: BIA states them and there is nothing to calculate.

Limitations of the anchored veneer

- The Modeling Analysis method is not built. It needs a model of veneer, ties and backing with cracked hinges; MDG Example 16.2-7 covers it.
- Stability analysis for a slender backing (hb / t over 167) is not built.
- The backing, its fasteners' strength, shelf angles and lintels (L/600), movement joints, water penetration and corbels are not part of it.
- The allowable stress level of the Tributary Area method applies 0.6 to wind and 0.7 to seismic to the strength-level load, ILX's reading.
- A tie's fasteners to the backing are listed, not calculated.
- BIA's examples say corrugated ties are excluded once engineered, though its Tables 1 and 6 give values for them. ILX flags that and leaves it to you.

Verification examples

Verification status: 28 values compared in 15 examples (14 published, 1 hand calculations from the cited equations): 28 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

BIA TN 3C (July 2026): the five design examples of Appendix A, pp. 18-26, wherever they compute or decide something, and seven values of Table 8. Example 3 is the case ILX refuses: it needs the Modeling Analysis method. The allowable stress level of the Tributary Area method has no worked value: by hand. **The code was not read:** when TMS 402-22 and the Designers' Guide arrive, check section numbers, the corrugated question, and add MDG 16.2-x.

CHAPTER 148

Adhered veneer

Thin masonry units bonded to their backing, by the 2022 chapter: the unit and assembly limits and fastener spacing along the framing on the prescriptive path, stresses and fastener loads on the engineered path, and the backing's deflection. **The code text was not read.** This type rests on one set of slides, TMS's Night School handout of November 2022, and has no worked example behind it. Treat it as a checklist that saves looking up the table, and read section 13.3 when you can.

Inputs of the adhered veneer

- **Design path.** Prescriptive, or engineered.
- **Load that governs, seismic design category.** Category D, E or F takes 80 percent of the table's spacing. qh is on the prescriptive path, since the tables stop at 60 psf; the out-of-plane load p, at strength level, on the engineered path.
- **Veneer.** Unit thickness, face, weight, assembly weight as installed, thickness to the back of the scratch coat, height above grade plane. Units over 360 in.² need an approved installation procedure: your statement.
- **Backing and fasteners.** Wood at 16 in. or 16 ga. steel at 16 in. take the tables; concrete and masonry have no fasteners here. On a framed backing, the fastener, the cavity from the stud to the back of the assembly and the spacing along the framing. On the engineered path the capacities are yours, from the fastener's design standard, at your method's level, and the vertical deflection under strength dead and seismic load is your value.
- **Backing height and deflection.** Under 0.42 of the strength wind load or under the strength seismic load.

Auto-design varies the fastener spacing, widest first. The fastener, cavity and assembly stay yours.

Checks performed

Check	Basis, all from the handout (to confirm against the code text)
Backing deflection, h/360 or h/150	13.3.1.2, p. 24
Unit: at most 2-5/8 in., 720 in.², under 30 psf	13.3.2.2, p. 26
Assembly: at most 50 psf, 4-5/8 in., 60 ft; sheathing on frames	13.3.2.5, p. 28
Fastener spacing, no interpolation, 80 % in D, E, F	Tables 13.3.2.5, 13.3.2.6, pp. 28-29
Engineered: tension 100 psi (60 for allowable stress), shear 50 psi (30)	13.3.3, p. 30
Engineered: fastener withdrawal and lateral load on its tributary area	Principles of mechanics, p. 30
Engineered: vertical deflection at most 1/8 in.	13.3.3, p. 30

Limitations of the adhered veneer

- **The steel table on the handout stops part way:** after the #8 screw at a cavity to 2 in. A fastener and cavity not on the page are refused, not guessed.
- The handout numbers its tables 12.3.2.5 and 12.3.2.6, as drafted; ILX reads them as 13.3.2.5 and 13.3.2.6.
- Framing at other spacings and qh over 60 psf go to the engineered path.
- On the engineered path, the tension and shear in the assembly almost never govern: they are there because the code gives them. How to apply them is ILX's plain reading, one square foot of bond face.
- Not part of it: the backing's strength, lath fastening, mortars, shear bond tests, movement joints, water penetration.

Verification examples

Verification status: 25 values compared in 17 examples (15 published, 2 hand calculations from the cited equations): 25 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No free worked example exists. Fourteen table cells are read back from the handout's pp. 28-29, including the next-band rule and the 80 percent factor; everything on the engineered path is by hand. MDG Example 16.3-1 and section 13.3 itself are in the documents to buy: when they arrive, check every limit here, complete the steel table and add that example.

CHAPTER 149

Masonry lap splice

The lap or development length of one bar size in grouted concrete masonry, to TMS 402-22 6.1.6 and 6.1.7. The equation is the same for strength and allowable stress design, so the LRFD / ASD switch is not shown.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers come from TMS's cross-reference, the hook rule from TMS's 2022 handout, and the wording was read in TMS 402-13 and in a 402-16 summary table. The 402-22 equation numbers are not confirmed: the steps give the 402-13 ones.

Inputs of the masonry lap splice

- **Length to check:** a lap, a straight bar's development, or a bar with a standard hook.
- **Bar:** No. 3 to 11, Grade 60 or 40, epoxy-coated or not. The length provided is checked against the length required.
- **Masonry:** f'm, wall thickness, the position of the bar: centre of the wall (cover worked out) or near a face (cover entered). **Limit the f'm used** is on by default at 4,000 psi; turn it off to use f'm as entered.
- **Splice spacing:** centre to centre of adjacent splices. K uses the clear distance between them; the bars spliced together do not count.
- **Lap:** contact or noncontact, with the distance between the bars.
- **Transverse bars in the lap:** off by default. On, with the conditions as inputs: a bar within 8 in. of each end, at most 1.5 in. clear of the lapped bars, fully developed. If any is not met, the lap is not reduced and the calculation says so.

Auto-design (editor toolbar) varies the length provided, in whole inches. Objective: the shortest length provided.

Checks performed

Check	Basis
Length required against provided	ld = 0.13 db² fy γ / (K √f'm), at least 12 in. (402-13 Eq. 9-16); K the least of the cover, the clear spacing and 9 db; γ 1.0, 1.3, 1.5; epoxy 150 percent
Transverse bars	ξ = 1.0 − 2.3 Asc / db^2.5, Asc at most 0.35 in.²; not below 36 db, and never longer than the lap without them (402-13 Eq. 9-18)
Standard hook	ldh = ld − γh db, γh 9.0 for No. 3 to 8, 8.0 for No. 9 to 11 (TMS 2022 handout)
Noncontact lap	Bars at most one-fifth of ld and 8 in. apart
Bar size	6.1.3: db at most 1/8 of the wall and 1/3 of the grout space; bars at most 8 % of the grout space at a contact lap, 4 % elsewhere

The drawings are the bars in elevation with the lengths, and a plan of the wall at the bars. Hook geometry is schematic.

Limitations

- Concrete masonry. The clay masonry rules were not read for the 2022 code.
- The order of the epoxy factor and the reduction by transverse bars is this calculation's reading: epoxy first. The code's least length for a hook is not on hand, so none is applied beyond ld's 12 in. (to confirm against the code text).
- Welded and mechanical splices, joint reinforcement and wires are not sized. Seismic rules for where splices may sit are not checked.

Verification examples

Verification status: 84 values compared in 79 examples (73 published, 6 hand calculations from the cited equations): 83 within tolerance, 1 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published: Masonry Designers' Guide Example 9.2-1 and Table 9.2.5, both as printed in TMS's ASD Night School 6 handout (March 2022, written to 402-16), slides 21 and 22: the cover, K and the 19.5 in. lap, and all 72 table values that have a number, at three f'm, three bar positions and nine bars. 71 agree. One is a recorded difference, on the Verification page: the table prints 12 in. for 12.27 in., where its other values round up. The dashes in the table are bars the size limit refuses here too.

Hand calculations, since no free example has them: epoxy, spacing governing K, a hook, transverse bars and the 36 db limit, the noncontact limit. The hook rule is from the 2022 handout. Nothing here is checked against the 402-22 text.

CHAPTER 150

Masonry retaining wall

A cantilever concrete masonry wall on a reinforced concrete footing, per foot of wall. The stem is by strength design or allowable stress design to TMS 402-22, stability by service loads, the footing by ACI 318-19 strength design either way. A concrete stem is the cantilever retaining wall calculation.

The LRFD / ASD switch sets the stem only. **Allowable stress** takes the service moment on a cracked section with the axial load left out, conservative for a wall with little of it, axial stress with its slenderness factor and shear, Chapter 8; there is no second-order term. It rests on TMS 402-13 Chapter 8, not 402-22; section numbers are marked to verify, and a calculation saved with ASD before this option is now designed by allowable stress. Stability is by service loads whichever way the stem is designed, which is why the allowable stress example below still verifies it.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers come from TMS's cross-reference, the changes from the TMS handouts of 2022, the second-order equation from the free sample of the Masonry Designers' Guide 2022, and the wording was read in TMS 402-13. Only Eq. 9-28 is a confirmed 402-22 equation number here.

Inputs of the masonry retaining wall

- **Wall and footing:** stem height, height above the backfill, footing thickness, toe, heel, key.
- **Masonry stem:** thickness, grouting, wall weight (an input: it counts in stability and in the axial load), f'm, mortar, one layer of bars with its distance from the earth face, the lap on the dowels. The dowels are taken as the size of the stem bars. TEK 15-07B's tables use d of 5, 7 and 9 in., which is 2.625 in. from the face.
- **Earth pressure:** Rankine, Coulomb, at rest or an equivalent fluid density entered from the geotechnical report. Water table, surcharge as live or dead. A live surcharge over the heel does not resist sliding or overturning.
- **Lateral resistance:** friction coefficient and lateral bearing per foot of depth (IBC Table 1806.2, up to 15 times the value at depth), top depth neglected. The factors of safety are inputs, 1.5 prefilled (IBC 1807.2.3).
- **Bearing pressure as soil depth over the key:** off by default. TEK 15-07B turns the average bearing pressure under the footing into an equivalent soil depth at the top of the key, which raises its resistance a good deal. Off, the key sees only its depth below grade. Whether the extra is reliable is your judgment.
- **Deflection limit:** 0.007 of the retained height prefilled. TMS 402 9.3.4.5 is written for walls between supports; a limit for a cantilever is your input.
- **Load factor on H:** 1.6 prefilled, ASCE 7-22 2.3.

Auto-design (editor toolbar) varies wall thickness, heel, toe, footing thickness, stem bars, lap and footing bars. Objective: the lowest cost index. The height, key, soil, wall weight and loads are kept, so a thicker wall gets no credit for its extra weight until you enter it.

Checks performed

Check	Basis
Sliding	Friction plus lateral bearing on the footing edge and the key, against the factor of safety entered; IBC 1806.3, 1807.2.3
Overturning	About the toe, against the factor of safety entered
Bearing pressure, middle third	Trapezoidal or triangular; allowable entered; the middle third is your criterion
Stem flexure with axial load	9.3.2 strain compatibility about the wall centre line, per foot, compressed width per bar of 5.1.2; φ of Table 9.1.4 at Pn = Pu/φ
Second-order moment	9.3.4.4, the cantilever form of MDG Example 17.2-5: the tip moves M h^2 / (κ E I), half the wall weight and the top load act over it, uncracked up to Mcr (with 0.9D) and Icr of Eq. 9-28 beyond. κ is 5 for a triangle, 4 for a uniform load, weighted when both act
Axial strength, axial limit	9.3.3.1.1 with an effective height of twice the stem; 9.1.4.4.2
Stem shear	9.3.4.3, 9.3.3.1.2, axial load not counted
Service deflection	The same tip equation at service, second order, against your limit
Bars	6.1.3 (including 8 % at laps), 6.1.4
Lap on the dowels	6.1.7.1.2, K the least of cover, clear spacing between splices and 9 db
Heel, toe, footing steel	ACI 318-19 22.2, 22.5, 7.6.1, 7.7.2 (with Table 24.3.2 crack control, an option on by default), 24.4.3, 25.4, as the concrete retaining wall
Dowel hook	ACI 318-19 25.4.3

The drawings are the wall section with bars and lateral pressure, and the service bearing pressure under it.

Limitations

- Allowable stress of the stem by hand and by a reading of TEK 15-07B Table 1 (No. 5 at 16 in. carries 45 pcf at 8 ft in a 12 in. wall, No. 4 does not): no free worked stem in full. Axial load does not reduce the moment on this path.
- Seismic earth pressure, compaction pressure, global stability and the key's own strength are not checked. Enter their effects separately where they apply.
- One thickness and one layer of bars up the stem; bars are not cut off. Horizontal bars, bond beams, drains and waterproofing are not designed.
- Spacing above six times the thickness leaves masonry spanning between bars; the compressed width is limited but that span is not checked (TEK 15-07B Table 1 note c).
- Concrete masonry only. Prescriptive seismic reinforcement (7.3, 7.4) is not checked (to confirm against the code text).
- 6.1.7.1.2 and the modulus of rupture are 402-13 wording under the 402-22 numbers (to confirm against the code text).

Verification examples

Verification status: 49 values compared in 8 examples (3 published, 5 hand calculations from the cited equations): 39 within tolerance, 10 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published: CMHA TEK 15-07B, pp. 2-3 (allowable stress era, IBC 2003): earth pressure, overturning moment, weights, friction and lateral bearing, sliding factor of safety, average pressure; the earth and footing moments agree. Its masonry lever arm does not: the note puts the masonry at the footing centre (2.67 ft) where its own drawing gives 2.15 ft, so its resisting moment, resultant and toe and heel pressures differ. It also uses 14,670 for its own sum of 16,194. Its key leaves out the key's own weight. All of these are recorded on the Verification page, not fitted.

Masonry Designers' Guide 2022, Example 17.2-5 (sample; 402-22; GFRP bars): pressure, forces, moments, section properties, Mcr. Only the equation for the second-order moment transfers to steel bars. Its printed 22,710 lb-in./ft drops the 5 from the denominator of its own equation; the equation as printed gives 21,464. Recorded on the Verification page.

A read of TEK 15-07B Table 2 (a sanity check, not a verification): at 8.7 ft and 45 pcf, No. 5 at 16 in. passes and No. 4 does not, as the table says. Its 60 pcf column gives the same No. 5 at 16 in., which falls short here; the table's basis isn't given, so this is an open question.

Hand calculations: stem strength and shear with no axial load, stability of the default wall, surcharge moments, lap. No free published example of a steel-reinforced masonry retaining stem written to 402-22 exists; the Guide's other examples are paid.

CHAPTER 151

Unreinforced masonry wall

A one-foot strip of a concrete masonry wall or partition without reinforcement, spanning out of plane, loadbearing or not. The design keeps it uncracked: TMS 402-22 8.2 by allowable stress or 9.2 by strength, with the switch in the toolbar. Reinforced walls are the masonry wall out-of-plane calculation.

Edition. The design basis is TMS 402-22 / TMS 602-22. The text of the 2022 code is not on hand. Section numbers come from TMS's cross-reference and the free sample of the Masonry Designers' Guide 2022, which also confirms 8.2.6.2 and a value of Table 8.2.4.2; the wording was read in TMS 402-13. No 402-22 equation number is confirmed here: they are written as 402-13 numbers. The bed joint shear rows of the 2022 tables are the 402-13 values (to confirm against the code text).

Inputs of the unreinforced wall

- **Span:** vertically (supported top and bottom, or a cantilever from the base) or horizontally between supports. A horizontal span uses tension parallel to the bed joints, gets no credit from axial load, and needs running bond.
- **Section:** thickness; hollow units on face shell bedding, grouted at a spacing or fully; or solid units. The net section is at a bed joint: face shells only where hollow. Partial grouting interpolates the table by the share of grouted cells. Webs per unit and web thickness set the area for shear on the webs.
- **Wall weight:** an input, counted as resisting tension. Enter what the wall weighs with its grout.
- **Check low dead load with a live lateral load:** on by default. ASCE 7 has no combination for a live lateral load with dead load resisting; the MDG-2022 example takes 0.6D. This adds 0.6D + L in allowable stress and 0.9D + 1.6L in strength. Off uses the ASCE 7 combinations only, which credits the full dead load. Your judgment.
- **Use the lowest bed joint shear stress:** off by default. The Guide's example takes the table's lowest value for out-of-plane sliding; off uses the row for the bond and grouting.
- **Loads:** w the axial load per foot at the top, e from the wall centre line, p the out-of-plane pressure. Interior partitions: at least 5 psf (ASCE 7-22 4.3.4).
- **Deflection limit:** span divided by the number entered; IBC Table 1604.3 by finish.

Auto-design (editor toolbar) varies thickness, grouting and grouted cell spacing. Objective: the thinnest wall, then the least grout. The wall weight is kept, so a heavier wall gets no credit until you enter it.

Checks performed

Check	Allowable stress	Strength
Flexural tension	M/Sn − P/An against Table 8.2.4.2, 8.2.4.2	ψ Mu/Sn − Pu/An against φ fr of Table 9.1.9.1, 9.2.4.1, φ = 0.60
Compression	fa/Fa + fb/Fb ≤ 1, 8.2.4.1 (402-13 Eq. 8-14)	Pu/An + ψ Mu/Sn ≤ φ 0.80 f'm, 9.2.4.1
Axial	P ≤ Pe/4 with the load's actual eccentricity (Eqs. 8-15, 8-19)	φ Pn, 9.2.4.2 (Eqs. 9-11, 9-12)

Check	Allowable stress	Strength
Moment magnifier	None in the code	ψ of 9.2.4.3, 1.0 at h/r up to 45
Axial tension	Not permitted, 8.2.5	Not permitted, 9.2.5
Shear on the webs	3/2 V/A ≤ 1.5 √f'm and 120 psi, 8.2.6	3.8 A √f'm and 300 A, 9.2.6
Shear on the bed joint	3/2 V/An ≤ 37 or 60 psi + 0.45 Nv/An, or 15 psi, Table 8.2.6.2	56 or 90 An + 0.45 Nu, or 23 An, Table 9.2.6.1
Deflection	Uncracked, against your limit	Same

Stresses are checked at 41 stations along the height in every combination, so the wall's weight changing with height is captured. A cantilever takes an effective height of twice its height.

The drawings are the wall along its span and the wall at a bed joint.

Limitations

- Out of plane only. In-plane shear wall action is not covered here.
- Chapter 15 of 402-22, prescriptive partition walls, is new and only its contents are on hand: its height and length limits and anchorage rules are not checked. This calculation is the engineered route of the MDG example.
- Concrete masonry. Clay masonry has a different modulus; AAC has its own chapter.
- Masonry not laid in running bond spanning horizontally needs a continuous grout section, which is not modelled, so it is refused.
- Parapets add weight; their own bending is not checked.
- Stresses from restraint of movement are controlled by joints (8.2.2 commentary), not calculated.
- φ of 0.60 and the numbers of the shear tables are 402-13 values (to confirm against the code text).

Verification examples

Verification status: 28 values compared in 6 examples (1 published, 5 hand calculations from the cited equations): 28 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Published: Masonry Designers' Guide 2022, Example 11.3-1 (sample pp. 4-5; written to 402-22): section properties, stresses, allowable stresses and both shear limit states of a 12 ft hollow partition. The Guide checks midheight and gets 9.6 psi net tension. The wall's weight falls with height, so the largest net tension is a little higher up, 9.9 psi at 6.9 ft; ILX reports that, within the tolerance on the printed value. The margin against 12 psi is about the same.

Hand calculations: the same wall by strength design, bed joint strength, a grouted cantilever, a loadbearing wall's Fa, Fb, Pe and Pn, a horizontal span. **There is no free published strength example** of an unreinforced wall: MDG 12.3-1 to 12.3-3 are in the paid Guide.

CHAPTER 152

Masonry infill

One concrete or clay masonry panel in a bounding frame, participating as an equivalent diagonal strut, or isolated and carrying only out-of-plane load. The basis is the infill chapter of the TMS code, read here as TMS 402-13 Appendix B and CMHA TEK 14-23. TMS 402-22 moved it to Chapter 12, which is not on disk for this calculation: a TMS summary says virtually no changes, and it is marked (to confirm against the code text).

Inputs of the masonry infill

- **Infill:** participating, or non-participating. A participating infill must fill the frame with no openings. A partial infill, or one with openings, cannot be part of the lateral system (B.1.5) and is entered as non-participating; its effect on the frame stays yours to consider.
- **Height, length, nominal thickness, fully grouted:** hollow units take the two face shells as net thickness.
- **f'm, Em:** Em is an input, 900 f'm for concrete masonry and 700 f'm for clay.
- **Bond and grouting:** sets the line of the bed joint shear strength in 9.2.6. **Net bedded area per foot:** 30 in.² for 8 in. units on face shell bedding.
- **Bounding frame:** E and I of a bounding column and beam, for bending in the plane of the infill. Average different columns; on a single story use the beam above.
- **At the top of the infill:** tight, gap under 3/8 in. or not sized for the displacement (strength and stiffness are halved, B.3.1.2.1), or sized to pass no force (refused: non-participating). A **gap at the sides** removes the horizontal arch; a top gap the vertical one.
- **Strut thickness:** the code text at B.3.4.1 reads specified thickness, and that is prefilled. TEK 14-23's worked example uses net thickness, which gives about a third of the stiffness on hollow units. The option is yours to decide, and the Eq. B-2a stiffness parameter uses net thickness in both.
- **Loads:** factored shear on this panel, from your frame analysis; bed joint compression; factored out-of-plane pressure. This is strength design only, as the code says.
- **Connectors and joints:** spacing, at most 4 ft; on a non-participating infill, the joint width and the frame's design displacement.

Auto-design varies the nominal thickness, thinnest first, then full grouting. Height, length, frame, loads and f'm are kept.

Checks performed

Check	Basis
Height to thickness at most 30	B.3.1.1
In-plane shear: least of corner crushing, the strut force at 1 in. of racking, bed joint shear over 1.5, φ = 0.60	B.3.4.3, Eqs. B-3, B-4; 9.2.6
Out-of-plane arching, φ = 0.60	B.3.6, Eqs. B-7a, B-8, B-9, thickness at most a height over 8

Check	Basis
Connector spacing	B.2.2.4, B.3.3.3
Isolation joint: at least 3/8 in. and the frame's displacement	B.2.1.2
Forces for the bounding frame (a note)	B.3.5

Limitations of the masonry infill

- The bounding frame is not analysed or designed. Enter the shear on this panel from your own analysis of a frame with the strut in it. The note lists the strut force, the column and beam shear to add and, as a pin-jointed frame, the axial forces.
- A non-participating panel's own bending between connectors is not here: use Unreinforced masonry wall or Masonry wall. Connector design is yours; the force is listed.
- AAC infill has its own equations and is not offered.
- R and Cd for this hybrid system are not in ASCE 7's table. The Commentary recommends the smaller of the frame's and the masonry's, which is yours to decide.
- Code numbers are 402-13's. Confirm them against 402-22 Chapter 12.

Verification examples

Verification status: 19 values compared in 4 examples (4 published, 0 hand calculations from the cited equations): 19 within tolerance, 0 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Source	What
CMHA TEK 14-23, Example 1, pp. 3-6	λ 0.0392, stiffness 56,030 lb/in., the bed joint lines, Vn 22,500 lb, φVn 13,500 lb (with the net thickness option, as the example)
CMHA TEK 14-23, Example 2, pp. 6-7	αarch 21, βarch 10, qn 41.4 psf, φqn 24.8 psf
C. J. Tucker, "So, You Need to Design a Masonry Infill", STRUCTURE magazine	32 ft by 14 ft panel: Eq. B-3 30,000 lb, strut at 1 in. 70,340 lb, 9.2.6 lines, φVn 18,000 lb, φqn 37.6 psf

No clay masonry example, no fully grouted example and no example with gaps was on disk: those paths are by hand in the behaviour tests. TMS's Masonry Designers' Guide Examples 15.2-1 and -2 are to be added when it is bought.

CHAPTER 153

Prestressed masonry wall

An unreinforced, post-tensioned concrete masonry wall per foot of wall, spanning vertically and pinned at the top and bottom, under out-of-plane lateral load and axial load. The basis is Chapter 10 of the TMS code, read as TMS 402-13: the 402-22 chapter is not on disk, so it is marked (to confirm against the code text). Service stresses by Chapter 8, moment strength by the chapter's own equations, as the code requires both.

Inputs of the prestressed masonry wall

- **Wall:** height, specified thickness, face shell, and the net section per foot (An, In, Sn, r) from CMHA CMU-TEC-002 for your bedding. 8 in. CMU with full mortar bedding is prefilled.
- **Masonry:** f'm, and f'mi, the strength when stressed. Em. The allowable flexural tension from Table 8.2.4.2 for your mortar: 33 psi prefilled, Type M or S portland cement lime, hollow units, normal to bed joints. The 402-22 table may read lower: read it when it arrives.
- **Tendons:** bonded or unbonded; laterally restrained or unrestrained; spacing, area, fpy, fpu.
- **Forces per tendon:** jacking, after transfer, and effective after all losses. These are entered because the code leaves the loss calculation to you (10.3.4). The Commentary's ranges for concrete masonry are 5 to 10 % at transfer and 30 to 35 % in total. The check says if you fall far outside them.
- **Anchorage bearing area.**
- **Loads:** axial dead and live at the top, an eccentricity, and the lateral load at the strength level (ASCE 7-22 W or E). It is used at 0.6 in service and 1.0 at strength.

Auto-design varies the tendon spacing, widest first. The tendon, its forces and the wall are kept.

Checks performed

Check	Basis
Tendon stress at jacking, after transfer, at anchorages	10.3.1, 10.3.2, 10.3.3
Axial and flexure after losses, and axial at transfer on f'mi with the 20 % increase	8.2.4 Eq. 8-14; 10.4.1.2
No net tension under prestress and dead load; net tension under lateral load at most Table 8.2.4.2	10.4.1.3; 8.2.4.2
Buckling, P at most Pe/4, prestress counted only with unrestrained tendons	10.4.2; Eqs. 8-15, 8-19
Moment strength: φ = 0.80, a from 0.80 f'm, Eq. 10-3 for unbonded tendons, d limited for unrestrained	10.4.3
a/d at most 0.38; compression block within the face shell on hollow walls	10.4.3.6; Commentary
Out-of-plane shear with the effective prestress in the axial force	10.6.1, 9.2.6
Anchorage bearing at jacking, 0.50 f'mi	10.8.4

Limitations of the prestressed masonry wall

- Walls pinned top and bottom, tendons concentric, one lateral load case with the load combinations stated. No cantilevers, no in-plane shear wall design, no mild reinforcement, no clay masonry losses.
- First-order moments. The chapter says the factored moment shall include the moment from relative lateral displacement (10.4.3.1): on a tall wall add it yourself. Deflection, camber and time effects (10.7) are not computed.
- Anchorage zone bursting (10.8.3), bond beams, corrosion protection (10.9) are stated, not designed.
- Code numbers are 402-13's. The 402-22 chapter and the Masonry Designers' Guide Examples 13-4 are to be read when bought.

Verification examples

Verification status: 12 values compared in 1 example (1 published, 0 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

Source	What
CMHA TEK 14-20A, Design Example, pp. 4-5	Wall weight at midheight 234 lb/ft, M 270 lb-ft/ft, the tendon stress limits, Ppi 2,850 and Ppf 1,800 lb/ft, fb 37, fa 49, fai 74 psi, fb - fa = -12 psi

That example is to 402-02, whose moment strength rule has since changed, so its strength figures are not compared. **No published example of the moment strength or the unity check at the current rules was available:** those are by hand from the equations. Treat this as a checked implementation of the chapter's equations, not a type verified end to end.

CHAPTER 154

AAC masonry wall

A one-foot strip of an autoclaved aerated concrete (AAC) masonry wall, spanning vertically between supports at the top and bottom, loaded out of plane with an axial load at the top: reinforced (TMS 402-22 11.3) or unreinforced (11.2), by strength design, the only method Chapter 11 offers. US and metric units.

Edition. TMS 402-22 is not on hand. Chapter 11 and Table 4.2.2 were read in TMS 402-16 (pp. C-32, C-161 to C-176). TMS's overview of the 2022 changes says the strength reduction factors moved into Table 11.1.5 with no other significant change; references give the 2016 section numbers, and steps that rest on them say "to confirm against the code text". The AAC strength classes and the strain factor α of the maximum reinforcement are from knowledge.

No free worked example of an AAC wall exists. This type was deferred in wave 7 for that reason and is now built on hand calculations only; see the verification section.

Inputs of the AAC wall

- **Wall:** reinforced or unreinforced; specified thickness (AAC units are solid); height between the supports; a parapet above the top support (its weight only); wall weight.
- **AAC:** strength class AAC-2, AAC-4 or AAC-6 (f′AAC 290, 580 or 870 psi), or entered (at least 290 psi). Class 2 uses a maximum usable strain of 0.0012, class 4 and higher 0.003.
- **Bed joint at the critical section:** thin-bed mortar caps the modulus of rupture at 80 psi; a Type M or S leveling bed at 50 psi.
- **Reinforcement:** bar and spacing, centred in grouted cores; grade (design fy at most 60 ksi); grout strength (2,000 to 5,000 psi); core diameter (for the bar size and area limits); the strain factor α of the maximum reinforcement, 1.5 prefilled.
- **Loads:** w the axial load per foot at the top, e its eccentricity from the wall centre line, p the out-of-plane pressure.
- **Limit shear to sliding friction μ Pu:** on by default, Eq. 11-8 as written, with μ = 1.0 at a thin-bed or leveling bed. Dowels across the base joint (shear friction) are outside this calculation; turning the limit off is your decision.

Auto-design (editor toolbar) varies the thickness, bar size and spacing. Objective: the thinnest wall, then the least steel. The wall weight is kept, so a heavier wall gets no credit until you enter it.

Checks performed

Check	Reinforced	Unreinforced
Flexure	Mu = wu h²/8 + Puf eu/2 + Pu δu by iteration (Eqs. 11-16, 11-18, 11-19), Mu ≤ φ Mn (Eqs. 11-27, 11-28), φ = 0.90	Net tension ψ Mu/Sn − Pu/An ≤ φ fr, φ = 0.60; ψ by 11.3.5.5.3 with 0.75 In
Compression	Through Mn	Pu/An + Mu/Sn ≤ φ 0.85 f′AAC
Axial	φ Pn, Eqs. 11-5, 11-6; Pu/Ag ≤ 0.20 f′AAC (0.05 above h/t 30), Eq. 11-15	φ Pn, Eqs. 11-3, 11-4

Check	Reinforced	Unreinforced
Shear	VnAAC = 0.8 √f'AAC b d (Eq. 11-14), φ = 0.80; sliding μ Pu (Eq. 11-8)	Same
Reinforcement	Bar ≤ 1/4 of the core, area ≤ 4.5 percent of the core (11.3.3.1), maximum reinforcement (11.3.3.2 with 9.3.3.2), grout 2,000 to 5,000 psi	—
Deflection	δs ≤ 0.007h with P-delta (Eq. 11-29)	Same, uncracked

Material values: EAAC = 6500 f'AAC^0.6 (Table 4.2.2); fr = 2 × 2.4 √f'AAC capped by the bed (11.1.8.2, 11.1.8.3). The drawings are the wall section along its height and a plan section with the grouted cores.

Limitations

- Out of plane, simply supported top and bottom (the 11.3.5.5.2 procedure). Cantilevers, partially supported walls and in-plane shear walls (11.3.6) are not covered.
- The bars are smeared over the foot of wall; the effective width of widely spaced bars (5.1.2) is not applied.
- Shear d is taken as t/2 for the unreinforced wall too, conservatively (to confirm against the code text).
- Bearing, anchor bolts in AAC, laps, bond beams and seismic detailing are not checked.

Verification examples

Verification status: 13 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

Hand calculations only. No free published AAC wall example exists: the Masonry Designers' Guide examples of Chapter 14 are paid. The hand calculations check the material values, a reinforced wall's moment, strength, stress block and shear, an unreinforced wall's tension, axial strength and slenderness, and the maximum reinforcement. This type should be read as hand verified until the Guide's examples are added.

CHAPTER 155

Glass unit masonry panel

A non-loadbearing glass block panel in an exterior or interior wall, designed by the empirical rules of TMS 402-22 Chapter 14: the panel's area and dimensions for its units and wind pressure, its support, expansion joints, mortar and joint reinforcement, and the line load its head and jamb supports must carry. US and metric units.

Edition. TMS 402-22 is not on hand. The chapter was read in TMS 402-16, where it is Chapter 13 (pp. C-191 to C-195). TMS's overview of the 2022 changes says the chapter became 14 and its limits were put in tables, with no other change listed; references give the 2022 number with the 2016 one in brackets. Figure 14.2-1 (13.2-1) was read off the scanned page at its grid lines.

Inputs of the glass unit panel

- **Panel:** exterior or interior; standard units (at least 3-7/8 in.) or thin units (3-1/8 in. hollow, 3 in. solid); the unit's face size (for panels one unit wide or high); width and height between structural supports; curved or not.
- **Strength level wind pressure:** the components and cladding pressure at strength level (ASCE 7-22). Interior panels above 16 psf follow the exterior rules.
- **Support:** panel anchors (spacing, embedment in the joint, fasteners per anchor) or channel-type restraints (the panel's recess in them); the deflection ratio of the supporting members; whether the panel stands on wood construction, and its installed weight.
- **Joints and mortar:** expansion joint thickness at the top and sides, mortar type, joint reinforcement spacing.

There is no auto-design: the panel's size is the architect's and the rules are limits.

Checks performed

Check	Basis
Area: exterior standard units by Figure 14.2-1; thin units 100 ft²; interior 250 / 150 ft² at 16 psf or less	14.2.1 to 14.2.4 (402-16 13.2.1 to 13.2.4)
Width and height between supports: 25 by 20 ft, thin units 15 by 10 ft	same
Thin units outside: wind pressure at most 32 psf	14.2.2 (13.2.2)
Unit thickness	14.1.3 (13.1.3)
Supporting members deflect at most l/600	14.3.2.1 (13.3.2.1)
On wood: 40 psf and 12 ft at most, vertical expansion joint	14.3.2.2, 14.3.2.3 (13.3.2.2, 13.3.2.3)
Panel anchors 16 in. apart at most, 12 in. into the joint, two fasteners; or 1 in. in channels; no anchors for a single unit	14.3.3 (13.3.3.1 to 13.3.3.4)
Line load for the head and jamb supports: the greater of 0.6W and 200 lb/ft	14.3.3 (13.3.3)

Check	Basis
Expansion joints at least 3/8 in.; Type S mortar outside; joint reinforcement 16 in. apart at most	14.4, 14.6, 14.7 (13.4, 13.6, 13.7)

The support line load takes each support as carrying half of the panel spanning to it (the head with the panel spanning vertically, the jambs with it spanning horizontally): a conservative split of a two-way panel. The drawing is the panel elevation with its supports.

Limitations

- The chapter is empirical: no stresses in the glass units are calculated.
- Figure 14.2-1 is digitized: areas are within about 2 percent of the printed curve. Pressures above the curve's start (about 102 psf) are not covered; below its end (about 15 psf) the area is held at the end of the curve, about 264 ft².
- The supporting frame, its members and anchorage, the base emulsion and fire ratings (listed assemblies) are not designed.

Verification examples

Verification status: 15 values compared in 10 examples (8 published, 2 hand calculations from the cited equations): 15 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published: the area and dimension limits of TMS 402-16 13.2.2 to 13.2.4 and Figure 13.2-1 read at its 16, 32, 48, 64 and 80 psf grid lines (compared within 5 percent, the reading accuracy of a scanned chart). **Hand calculations:** the support line load. No worked example of a glass block panel is on hand.

CHAPTER 156

Freestanding masonry wall

A garden wall or fence: a concrete masonry stem cantilevered from a continuous concrete footing, under wind or a seismic coefficient. The stem is designed at the top of the footing to TMS 402-22 (reinforced or unreinforced, strength or allowable stress, with the switch in the toolbar); the footing is checked for overturning, sliding and soil bearing. US and metric units.

Edition. TMS 402-22 is not on hand: the stem uses the shared masonry library (the 402-13 text with the 402-22 numbering where confirmed, as the other masonry types). The wind on a freestanding wall follows ASCE 7-22 29.3; whether 7-22 writes Kd inside Eq. 29.3-1 is to be confirmed. CMHA TEK 14-16B, Concrete Masonry Fence Design, is on hand and its tables are compared below.

Inputs of the freestanding wall

- **Wall:** exposed height above grade; the stem below grade to the top of the footing (TEK 14-16B uses 18 in. for 4 and 6 ft fences, 24 in. for 8 ft; check frost depth); thickness; reinforced or unreinforced; grouting (fully, or only the cells with bars); f'm; for unreinforced stems the mortar; for reinforced stems the bar, its spacing and grade (bars at the centre of the wall); the wall weight with its grout.
- **Wind:** a strength level pressure entered, or computed from the basic wind speed, exposure, Kzt, Ke, Kd (0.85 for solid freestanding walls) and the force coefficient Cf of ASCE 7-22 Figure 29.3-1, which you read for the wall's proportions: 1.3 for a long wall with clearance ratio s/h = 1 (case A), up to 1.8 for short walls, and higher near free ends (case C). Kz is taken at the top of the wall, at least 15 ft.
- **Seismic:** a horizontal coefficient on the stem's weight at strength level, from ASCE 7-22 Chapter 15 for the structure; 0 for none.
- **Footing and soil:** width, thickness and the stem's offset from its centre (both wind directions are checked); concrete and soil unit weights; the allowable bearing pressure (geotechnical report or IBC Table 1806.2); the friction coefficient; whether to count the lateral bearing pressure of IBC 1806.2 down to the footing bottom; the factors of safety for overturning and sliding. IBC 1807.2.3 asks 1.5 for retaining walls; for a fence it is your choice, 1.5 prefilled.

Auto-design (editor toolbar) varies the wall thickness, the bar and spacing and the footing width. Objective: the least masonry, steel and footing concrete. The wall weight is an input and is kept.

Checks performed

Check	Basis
Stem flexure, reinforced	Strength: 0.80 f'm stress block (face shell, or face shell and grouted cells), φ from the net tensile strain (TMS 402-22 9.3.2, Table 9.1.4). Allowable stress: cracked transformed section (8.3)
Stem flexure, unreinforced	Net tension M/S − P/A at most φ fr (Table 9.1.9.1, φ = 0.60) or the allowable tension of Table 8.2.4.2
Stem shear	9.3.3.1.2 or 8.3.5 (reinforced, γg = 0.75 when partially grouted); 9.2.6 or 8.2.6 on the webs (unreinforced)

Check	Basis
Overturning	Unfactored dead load (stem, footing, soil on the footing) about the toe against the lateral load at 0.6W or 0.7E
Sliding	Friction μ N plus the optional lateral bearing, against the lateral load at 0.6W or 0.7E
Soil bearing	Every ASD combination of ASCE 7-22 2.4, no tension between footing and soil

The stem is checked at the top of the footing, where the moment is largest. The drawing is the wall section with the stem bar, the footing and grade.

Limitations

- One foot of a straight cantilevered wall. Pier-and-panel fences, pilasters, serpentine walls (TEK 14-16B Figure 2) and walls between returns are not covered.
- Free ends without a return carry more wind: TEK 14-16B adds 5 psf over a length equal to the height; Figure 29.3-1 case C gives higher coefficients. Enter the higher pressure for that length.
- The footing's own reinforcement, dowels and laps, the coping and control joints are not designed.
- Second-order effects in the stem are neglected (short cantilevers); the footing's own seismic inertia is not added.

Verification examples

Verification status: 8 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published calculation of a fence to the current codes is on hand: hand calculations of the wind pressure, the footing stability and bearing, and the stem strength. CMHA TEK 14-16B's Tables 2 and 4 are prescriptive selections (2003 / 2006 IBC, ASCE 7-05, f′m 1,500 psi) and are read backward in the behaviour tests: an 8 ft fence of 8 in. CMU with No. 4 bars at 24 in. under the table's 20 psf (32 psf at strength level) uses about 88 percent of its flexural strength here and fails at twice the spacing; the table's 12 by 42 in. footing passes overturning and bearing on 2,000 psf soil.

CHAPTER 157

Masonry fire resistance

The calculated fire-resistance rating of a concrete masonry or clay masonry wall from its equivalent thickness, with finishes, plaster and a second wythe, by IBC 2021 Section 722 (722.3 concrete masonry, 722.4 clay brick and tile). Section 722 is the building code's copy of the calculation method of TMS 216 / ACI 216.1. US and metric units.

Edition. IBC 2021 Section 722 was read (pp. 7-65 to 7-75). TMS 216 / ACI 216.1 itself is not on hand; any difference in its current edition is to be confirmed. The matching of the aggregate types of Table 722.3.2 to the columns of the finish factor table, Table 722.2.1.4(1), was made from the column headings and is marked to confirm.

Inputs of the fire resistance calculation

- **Masonry:** concrete masonry units, solid clay brick (at least 75 percent solid), or hollow clay brick or tile.
- **Aggregate** (concrete masonry): the four rows of Table 722.3.2. Units made with a blend of aggregates (note d) need the supplier's value; enter the row that is conservative.
- **Actual thickness and percent solid:** the equivalent thickness is the percent solid times the actual thickness (net volume by ASTM C140 or C67, from the supplier).
- **Cells:** hollow or partially grouted (counts as hollow, 722.3.1.2), solidly grouted, or every cell filled with an approved loose fill: these two count at the actual thickness.
- **Required rating:** from the IBC for the wall's use (Tables 601 and 602, Chapter 7).
- **Finishes** (concrete masonry): gypsum wallboard, Type X wallboard, Portland cement-sand plaster or gypsum sand plaster on each side. Away from the fire a finish adds its thickness times the factor of Table 722.2.1.4(1); on the fire side it adds the time of Table 722.2.1.4(2). Portland cement-sand plaster 5/8 in. or less applied directly counts in the equivalent thickness.
- **Fire from:** either side (the wall is rated from both and takes the lower), or side 1 only, which 722.3.2.3 allows for exterior walls more than 5 ft from the lot line. Side 1 is then the inside.
- **Sanded gypsum plaster** (clay masonry): 1/2, 5/8 or 3/4 in. on one or two sides, Eq. 7-9 with Table 722.4.1(4).
- **Second wythe:** concrete masonry of the same aggregate, or solid brick, by its equivalent thickness, and whether a continuous airspace of 1/2 to 3-1/2 in. separates the wythes.
- **Combustible members framed into the wall:** the solid material behind them must be at least 93 percent of the tabulated thickness.

Auto-design (editor toolbar) varies the unit thickness and the filling of the cells. Objective: the thinnest wall, then no fill. The aggregate, percent solid, finishes and the required rating are kept.

Checks performed

Check	Basis
Rating at least the required rating	IBC 2021 722.3.2 (Table 722.3.2) or 722.4.1 (Table 722.4.1(1)), interpolated

Check	Basis
Finishes away from and on the fire side, both directions	722.3.2.1 to 722.3.2.3, Tables 722.2.1.4(1) and (2)
Masonry alone at least half the required rating where finishes contribute	722.3.2.4
Plaster on clay masonry	722.4.1.2, Eq. 7-9
Two wythes	722.3.3 Eq. 7-7, 722.4.1.3 Eq. 7-10, 722.4.2 Eq. 7-11
Solid material behind framed combustible members	Table 722.3.2 note b, Table 722.4.1(1) note c

The drawing is a plan section through the wall with its finishes or second wythe.

Limitations

- Walls only: lintels and columns (Tables 722.3.4, 722.3.5, 722.4.1(5), (6)) and steel-framed brick veneer (Table 722.4.1(2)) are not covered.
- A wall thinner than the first row of its table rates as zero; above the last row it rates at the last row's hours (4 hours).
- With two wythes, finishes are not added.
- Joints, penetrations and the attachment of finishes (722.3.2.5) are not checked. A tested and listed assembly may replace this calculation.

Verification examples

Verification status: 17 values compared in 12 examples (8 published, 4 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published: rows of IBC 2021 Table 722.3.2 (all four aggregates) and Table 722.4.1(1) (all three clay rows), each a solidly grouted or solid wall of the tabulated thickness giving the tabulated rating. **Hand calculations:** interpolation, finishes from both sides, plaster on brick by Eq. 7-9, and two wythes by Eq. 7-7. No published worked example of the calculation is on hand.

CHAPTER 158

Masonry strength f′m

The specified compressive strength of masonry, f′m, verified by the unit strength method of TMS 602-22 Article 1.4 B.2: from the strength of the units and the mortar type, for concrete masonry (Table 2) and clay masonry (Table 1). The calculation also checks the bed joints, the grout and the range of f′m the Code allows, and the spacing of control or expansion joints against criteria you set. US and metric units.

Edition. TMS 602-22 is not on hand. Tables 1 and 2 were read in TMS 602-16 (pp. S-16 and S-18); TMS's overview of the 2022 changes lists no change to the unit strength method. The f′m limits were read in TMS 402-16 9.1.9.1; TMS says the 2022 edition moves them to 4.3 so they hold for every design method. Steps that rest on the 2016 text say "to confirm against the code text".

Inputs of the f′m calculation

- **Masonry:** concrete masonry units (ASTM C90) or clay units (ASTM C62, C216, C652).
- **Unit strength:** the net area compressive strength of the units, from their tests or the supplier.
- **Mortar:** Type M or S, or Type N (ASTM C270).
- **Units less than 4 in. nominal height:** concrete masonry only; Table 2's footnote takes 85 percent of the tabulated f′m.
- **Bed joint thickness:** the method needs 5/8 in. or less.
- **Interpolate between table rows:** off by default. The Specification does not say whether its tables may be interpolated. Off reads the row the units meet, which is conservative; on interpolates linearly between rows. Your decision.
- **Specified f′m** and, for grouted masonry, the **grout strength**.
- **Movement joints:** wall height, joint spacing, and your criteria: a maximum spacing and a maximum ratio of panel length to height. The Code does not set them. For concrete masonry CMHA's crack control guidance (TEK 10-2C, CMU-TEC-009-23) uses 1.5 and 25 ft with horizontal reinforcement; for clay masonry BIA Technical Note 18A spaces expansion joints at about 25 ft in walls without openings. Those values are prefilled; change them to your own.

The results also list the unit strength each mortar type needs for the specified f′m, the way the Specification's commentary reads the tables backward.

Auto-design (editor toolbar) varies the unit strength over the values of the two tables (and the mortar, if you unlock it). Objective: the weakest units. f′m, grout and the joint criteria are kept.

Checks performed

Check	Basis
f′m from the units and mortar at least the specified f′m	TMS 602 1.4 B.2, Tables 1 and 2
Bed joints at most 5/8 in.	TMS 602 1.4 B.2.a(2), b(2)

Check	Basis
f'm from 1,500 to 4,000 psi (concrete) or 6,000 psi (clay)	TMS 402-22 4.3 (402-16 9.1.9.1.1)
Grout at least f'm and 2,000 psi; at most 5,000 psi for concrete masonry	TMS 602 2.2; TMS 402-22 4.3 (402-16 9.1.9.1.2)
Joint spacing and panel ratio	Your criteria

The drawing is the wall panel between joints with the strengths noted.

Limitations

- The unit strength method only. The prism test method (1.4 B.3) and AAC masonry (whose f'AAC is the unit's strength class) are not covered.
- Unit, mortar and grout testing are the project's quality assurance, not part of the calculation.
- Joint spacing is checked against your criteria; shrinkage, thermal and moisture movements are not calculated.
- Table values are those of TMS 602-16 (to confirm against the code text: against 602-22).

Verification examples

Verification status: 13 values compared in 7 examples (5 published, 2 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published: the rows of TMS 602-16 Table 2 (concrete) and Table 1 (clay) read forward and backward: f'm for a unit strength and mortar, and the unit strength needed for an f'm. **Hand calculations:** the 85 percent of short units and an interpolated value. There is no published worked example of the method beyond the tables themselves.

CHAPTER 159

Empirical masonry design

A masonry wall proportioned by the empirical rules of TMS 402-16 Appendix A rather than by analysis: the limits on where the method applies (seismic design category, risk category, wind speed and building height), lateral support (l/t or h/t), the allowable compressive stress on the gross area, minimum thicknesses, foundation walls, shear wall length and the ties of non-composite walls. US and metric units.

Edition: TMS 402-22 has no empirical design. The TMS cross reference list (2016 to 2022, on hand) marks Appendix A "Deleted"; the 402-22 contents have Appendices C and D only. This calculation follows TMS 402-16 Appendix A (pp. C-203 to C-219, on hand), the edition IBC 2021 adopts. Where your governing code adopts TMS 402-22 (IBC 2024, to be confirmed against its text) a wall cannot be designed this way: choose that edition in the input and the calculation says NOT FEASIBLE; engineer the wall with the masonry wall or unreinforced masonry wall types.

Inputs of the empirical wall

- **Building and limits:** the code edition adopted; the seismic design category; the risk category; the basic wind speed and building height (Table A.1.1); whether the wall is exterior, interior load-bearing, or a shear wall of the lateral-force-resisting system; whether the building is enclosed (interior walls); for SDC C, whether the prescriptive reinforcement of 7.4.3.1 is provided; whether the masonry sits on wood; whether it resists soil, fluid or other horizontal loads.
- **Wall:** the construction (the rows of Table A.4.2: solid or grouted clay and concrete masonry, hollow clay or concrete units, non-composite multiwythe walls, stone); for concrete units, whether they meet the normalized web area of A.1.1.3 or are grouted solid; the unit strength on the gross area (net strength × percent solid); the mortar; the nominal thickness (the sum of the wythes for a non-composite wall, with the loaded wythe entered separately); load-bearing or not; the number of stories; a wall supported top and bottom, a cantilever wall or a foundation wall; the height between lateral supports, the length between cross walls or pilasters, a parapet, and one opening with its strip WS.
- **Gravity loads:** dead and live load on the wall per foot, the wall weight, and the eccentricity of the resultant.
- **Foundation walls:** unbalanced backfill, the length between perpendicular walls, and the site conditions of A.6.3.1 (b), (c), (d), (f) as one confirmation.
- **Ties and shear walls:** tie spacing for non-composite walls; for shear walls the long dimension of the building, the cumulative length in the weakest plane, the number of planes, the diaphragm and its length-to-width ratio.

Auto-design varies the nominal thickness and the unit strength. Objective: the thinnest wall, then the weakest units. Everything else is kept.

Checks performed

Check	Basis
Edition adopted, SDC, risk category, wood support, other horizontal loads, wind and height	A.1.2.2 to A.1.2.6, Table A.1.1; Table CC-A.1.1; TMS 402-22 deleted the appendix
SDC C reinforcement, CMU web area	Table CC-A.1.1 (7.4.3.1), A.1.1.3
Building height 35 ft (shear walls)	A.2
Compressive stress on the gross area	A.4.1, Table A.4.2 interpolated (note 1); the loaded wythe of a non-composite wall (note 2)
Resultant in the middle third	A.1.2.1
l/t or h/t, with openings	A.5.1, Table A.5.1, √(WT/WS), WS ≥ 3t, opening span
Cantilever walls	A.5.2
Minimum thickness, parapets	A.6.2
Foundation walls	A.6.3.1, Table A.6.3.1
Shear walls	A.3.1.1, A.3.1.2, Table A.3.1
Wythe ties	A.7.3.1

Twelve-inch hollow concrete masonry falls under both "greater than 8 and up to 12 in." and "12 in. and greater" in Table A.4.2; the lower values are used (to confirm against the code text). Rubble stone has one printed value, used for both mortars. The drawing is the wall elevation with its supports, parapet and opening.

Limitations

- Anchorage (A.8), solid units under joist bearing (A.8.3.6), roofs without thrust (A.3.2), headers (A.7.2), stone bonding (A.7.4), change in thickness (A.6.2.7), chases and lintels (A.9), foundation piers (A.6.4) and running bond (4.5) are listed as notes to follow on the drawings, not checked.
- Partition walls are not designed by Appendix A (A.1.1.1): use the masonry partition type.
- One opening per wall strip; for several openings enter the virtual opening that encompasses them (A.5.1).

Verification examples

Verification status: 12 values compared in 11 examples (8 published, 3 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values: rows of TMS 402-16 Tables A.4.2, A.5.1 and A.6.3.1 and the shear wall length of Commentary Figure CC-A.3-1. The interpolation of Table A.4.2, the reduction for an opening and the parapet thickness are **hand calculations** from the same provisions. There is no TMS 402-22 example: the 2022 edition has no empirical design.

CHAPTER 160

Corbelled masonry

A corbel of masonry courses stepped out from a wall: as a detail, by the prescriptive limits of TMS 402-22 5.6.2, or as a bearing ledge that carries a line load, designed by allowable stress for unreinforced masonry (5.6.1, Chapter 8). The corbel itself and the concrete masonry wall below it are checked. US and metric units.

Edition. TMS 402-22 is not on hand. The corbel provisions were read in TMS 402-16 5.5 (pp. C-59 to C-61, Figures CC-5.5-1 and CC-5.5-2); the TMS cross reference list renumbers them 5.6 in 402-22 with no change noted. The Chapter 8 values come from the shared masonry library (402-13 text with 402-22 numbering where confirmed); the allowable tension parallel to the bed joints was added to it from knowledge and is to be confirmed against the code text.

Inputs of the corbel

- **Corbel:** load-bearing or not; the number of corbelled courses; the projection of each course; the nominal unit height and thickness; whether the units are solid or filled with mortar or grout; the wall kind (single wythe, hollow, cavity or veneer: half the wythe; multiwythe bonded: half the wall) and the nominal thickness the limit uses; whether the back surface stays within 1 in. of plane.
- **Loads (load-bearing):** dead and live load on the corbel per foot, its position from the wall face (the centre of the bearing), the bearing width across the corbel, and the dead and live load on the wall above, which helps against tension in the wall below.
- **Wall below (load-bearing):** concrete masonry thickness, grouted solid or hollow, f′m, mortar, and the height between its supports.

The method switch is hidden: allowable stress design only, load combinations D and D + L (ASCE 7-22 2.4.1).

Auto-design varies the number of courses, the wall thickness and its grouting. Objective: the fewest courses, then the lightest wall. The projection per course, the units, the loads, f′m and the mortar are kept.

Checks performed

Check	Basis
Total projection, projection of one unit (height and thickness), solid or filled units, back surface	5.6.2 (a) to (d): code checks for a non-load-bearing corbel, guidance for a load-bearing one
Bearing on the corbel	0.25 f′m (8.1.5 in the library, to confirm)
Corbel flexural tension parallel to the bed joints	The corbel as a cantilever from the wall face, its courses acting together; Table 8.2.4.2, running bond (engineer's model)
Corbel shear at the wall face	1.5 V / A against min(1.5 √f′m, 120 psi, 37 psi), no precompression on the vertical plane (8.2.6.2)
Wall below: axial and flexural compression	$f_a/F_a + f_b/F_b \le 1$, the load at e + t/2 from the wall centre (8.2.4.1)
Wall below: buckling	$P \le P_e/4$, $P_e = \pi^2 E_m I / h^2 (1 - 0.577\, e/r)^3$

Check	Basis
Wall below: net tension	Table 8.2.4.2 normal to the bed joints, under the dead-load combination

The drawing is a section through the wall and the corbel with each course's projection and the load position.

Limitations

- Unreinforced, allowable stress only; reinforced or strength-designed corbels are not covered.
- The wall below is concrete masonry; brick or stone walls below a corbel are not covered.
- The ledger or angle on the corbel and its anchorage are not designed (the masonry ledger and masonry anchor types).
- The wall's own wind or seismic load is not added: check the wall with the masonry wall or unreinforced masonry type.
- Pe goes to zero when the eccentricity reaches 1.73 r, as the equation is written: heavy ledges far from the wall fail the buckling check by design.

Verification examples

Verification status: 10 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published corbel example is on hand: hand calculations of the 5.6.2 limits, the bearing, the corbel's tension and shear, and the wall below (axial and flexural compression, buckling load, net tension), from the cited equations.

CHAPTER 161

Masonry parapet

A concrete masonry parapet cantilevered above the roof line: components-and-cladding wind on parapets or the seismic force on a cantilevered parapet, the parapet designed at the roof line (reinforced or unreinforced, strength or allowable stress with the switch in the toolbar), and the anchorage of the wall to the roof at that line. US and metric units.

Edition. TMS 402-22 is not on hand: the parapet uses the shared masonry library (the 402-13 text with the 402-22 numbering where confirmed), as the freestanding wall does. ASCE 7-22 is on hand and read for 30.6 (parapets), 13.3 and Table 13.5-1, 12.11.2.1 and 1.4.4.

Inputs of the parapet

- **Parapet:** height above the roof line; wall thickness; reinforced or unreinforced; grouting; f′m; the mortar (unreinforced); the bar, spacing and grade (bars at the centre); the wall weight with its grout; the wall below from the floor to the roof (for the anchorage reaction and the tributary weight); the mean roof height.
- **Wind:** from the basic wind speed, exposure, Kzt, Ke and Kd (0.85), the wall zone (4 or 5, Figure 30.3-1) and the roof zone behind the parapet (2 or 3, Figure 30.3-2A), and GCpi for the wall below (0.18 enclosed); or the net parapet pressure and the wall pressure entered at strength level. The effective wind area of the parapet is its height times a third of it.
- **Seismic:** SDS, Ip, and the building's R, Ω0 and Ie (for Rμ); the flexible diaphragm span Lf for ka (0 for a rigid diaphragm).
- **Anchorage:** the anchor spacing along the wall and the design strength of one anchor at strength level (φRn: the masonry anchor type, or the connector's listing).

Auto-design varies the wall thickness, the bar and spacing and the anchor spacing. Objective: the least masonry and steel, then the fewest anchors. The height, loads, f′m, the anchor's strength and the building data are kept.

Checks performed

Check	Basis
Wind on the parapet	ASCE 7-22 30.6, Eq. 30.6-1: case A (wall positive, roof edge or corner negative) and case B (wall positive and negative); zone 3 takes zone 2 with a parapet of 3 ft or more (Figure 30.3-2A note 5); Figure 30.4-1 above 60 ft
Seismic on the parapet	ASCE 7-22 13.3.1: Fp = 0.4 SDS Ip Wp (Hf/Rμ)(CAR/Rpo), CAR = 2.2, Rpo = 1.5, Hf = 3.5 at the roof, between 0.3 and 1.6 SDS Ip Wp; Ev = 0.2 SDS D
Parapet flexure, reinforced	Strength: stress block, φ by Table 9.1.4 (9.3.2). Allowable stress: cracked transformed section (8.3)
Parapet flexure, unreinforced	M/S − P/A at most φ fr (Table 9.1.9.1) or Table 8.2.4.2

Check	Basis
Parapet shear	9.3.3.1.2 / 8.3.5 (reinforced), 9.2.6 / 8.2.6 (unreinforced)
Anchorage at the roof line	Wind reactions inward and outward of the wall below and the parapet; 12.11.2.1 Fp = 0.4 SDS ka Ie Wp (at least 0.2 ka Ie Wp and 5 psf); 1.4.4 (0.2 Wp, at least 5 psf); times the spacing against the anchor's design strength

The anchorage is at strength level whatever the method of the parapet. A solid masonry parapet carries the difference of the pressures on its two faces, so the GCpi of Eq. 30.6-1 cancels across it; GCpi is used on the wall below. The drawing is a section through the parapet and the top of the wall at the roof.

Limitations

- Low-slope roofs only (Figure 30.3-2A); sloped roofs, return corners and braced parapets are not covered.
- The roof diaphragm, its continuous ties (12.11.2.2), the 1.4 factor on steel anchorage elements (12.11.2.2.2), the anchor hardware, coping and flashing are not designed.
- The wall below is not designed in its own span (masonry wall or slender wall types).
- Second-order effects in the parapet are neglected (short cantilevers).

Verification examples

Verification status: 13 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published masonry parapet example to ASCE 7-22 is on hand: hand calculations of the parapet wind pressures (cases A and B), the seismic force, the moment and strength at the roof line, and the anchorage reactions, from the cited equations.

CHAPTER 162

Ledger to masonry wall

A wood or steel ledger bolted to the face of a grouted concrete masonry wall, carrying a floor or roof: the cast-in headed anchor bolts by TMS 402-22 strength design (gravity shear, the tension from the ledger's eccentricity, and the wall anchorage when it is routed through the ledger), the ledger itself, the prohibition of cross-grain bending of wood ledgers, and the wall anchorage forces of ASCE 7-22 12.11.2.1 and 1.4.4. US and metric units.

Edition. TMS 402-22 is not on hand: the bolts use the shared anchor library, as the masonry anchor type does (402-13 / 402-16 text with the 2022 changes from the TMS handouts, marked to confirm). The wood ledger uses NDS 2024 (on hand), ASCE 7-22 12.11.2 and 1.4.4 are read in the library copy, and CMHA TEK 12-03C (on hand) gives the lever arm. The method switch is hidden: strength design (LRFD) only.

Inputs of the ledger

- **Ledger:** sawn lumber (species, grade, size) or a steel angle (legs, thickness, Fy, Fu). For wood, the dowel bearing strength of the grouted masonry Fem: no published value exists; 3 f'm ≤ 7,500 psi by analogy with concrete (AWC TR12 Table A1) is prefilled, your call.
- **Bolts:** diameter, steel, embedment to the bearing surface of the head, spacing along the ledger, the bolt's distance below the top of the ledger, and the lever arm factor for the eccentricity (5/6 of the bolt-to-bottom-of-ledger distance, TEK 12-03C).
- **Wall:** concrete masonry thickness (grouted at the bolts), f'm, grout, and the distances from the bolt row to the masonry edges below (the shear acts toward it) and above.
- **Loads:** dead, live, roof live or snow per foot on the ledger, the load line from the wall face (the hangers' bearing line), and the NDS live load λ.
- **Wall anchorage:** through separate ties (the ledger carries gravity only) or through the ledger bolts; the seismic design category, SDS, Ie, the flexible diaphragm span, the wall weight and height tributary to the ledger line, the wind on the wall; with the anchorage through the bolts, the factor of ASCE 7-22 14.4.5 on seismic forces not governed by steel (on, 2.0, as in the masonry anchor type).

Auto-design varies the bolt spacing and diameter and the ledger size. Objective: the fewest and smallest bolts, then the smallest ledger.

Checks performed

Check	Basis
No cross-grain bending of a wood ledger for wall anchorage, SDC C to F	ASCE 7-22 12.11.2.2.3
Bolt tension, shear, interaction	TMS 402-22 9.1.6.3: breakout on Apt (the row's cones shared, cut by the edges), steel from fu, crushing, pryout, breakout toward the edge below; exponent 5/3
Embedment, grout around the head, clear distance	6.3.1, 6.3.6

Check	Basis
Bolt bearing in the wood ledger	NDS 2024 Table 12.3.1A yield limits, Fe⊥ of the ledger, Fem of the masonry; Z′ = Z KF φz λ
Ledger bending and shear between bolts	NDS 2024 3.3, 3.4 (simple span between bolts)
Edge distances in the ledger	NDS 2024 Table 12.5.1C: 4D to the loaded edge, 1.5D to the unloaded edge
Steel angle: bearing at the hole, the outstanding leg	AISC 360-22 J3.10, F11.1
Wall anchorage	ASCE 7-22 12.11.2.1, 1.4.4; reported for separate ties, added to the bolts when routed through the ledger

The drawing is a section through the wall, the ledger and one bolt.

Limitations

- One row of cast-in headed bolts in grouted concrete masonry. Post-installed anchors, bent bars, hollow masonry and brick are not covered.
- Joist hangers, continuous ties and subdiaphragms (12.11.2.2.1), the wall out of plane and ledger splices are not designed.
- The ledger spans simply between bolts (conservative); its own end distances at splices are not checked.
- Strength design only.

Verification examples

Verification status: 11 values compared in 4 examples (1 published, 3 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not yet recorded for this module.

Published: the tension from the eccentric load in CMHA TEK 12-03C's strength design example (2,688 lb for two bolts). TEK 12-03C's anchor strengths follow TMS 402-13 and differ from the current equations, so they are not compared. The rest are **hand calculations** from NDS 2024, TMS 402 and ASCE 7-22.

CHAPTER 163

Masonry arch

A segmental, semicircular or jack (flat) arch of unreinforced masonry over an opening under a uniform load: the line of thrust, the compression in the ring, the skewback joint, and the masonry pier beside the opening or a steel tie that takes the horizontal thrust. US and metric units.

Edition and method. The BIA Technical Notes on arches (TN 31 series) are not on hand: the thrust-line method is written from knowledge and marked "to confirm against the code text"; the stresses use TMS 402 Chapter 8 (unreinforced, allowable stress) through the shared masonry library (402-13 text with 402-22 numbering where confirmed). The method switch is hidden: allowable stress only, load combinations D and D + L.

Inputs of the arch

- **Arch:** shape; span between springings; the rise of the axis (segmental; a semicircle rises half the span, a jack arch is flat); the ring's radial depth; the arch width (the wall thickness); for a jack arch the skewback angle from vertical; f′m; mortar (for the abutment's allowable tension).
- **Loads:** dead and live load per foot, uniform over the span. Whether the arch carries the full height of wall above or only the arching triangle is your judgment: enter the load that results, with the arch's own weight.
- **Abutment:** a steel tie (diameter, Fy) or the masonry beside the opening: its length to the wall end, the next opening or a joint (the end pier governs), the springing's height above the pier's base, the masonry above the springing over it, and the wall weight.

Auto-design varies the ring depth and the tie diameter. Objective: the shallowest ring, then the smallest tie. The span, rise and shape are the architect's and are kept.

Checks performed

Check	Basis
Thrust line within the middle third	The least-thrust parabola through points within the middle third at the crown and the springings (a 21 × 21 grid), tested at 41 sections; BIA TN 31 series (to confirm)
Ring compression	fa/Fa + fb/Fb ≤ 1, Fa = 0.25 f′m, Fb = 0.45 f′m, the resultant along the thrust line at its offset (TMS 402 8.2.4)
Skewback joint shear	1.5 V / A ≤ min(1.5 √f′m, 120 psi, 37 psi + 0.45 Nv/An) (8.2.6.2)
Steel tie	AISC 360-22 D2, yielding of the gross area
Abutment pier	Net tension (Table 8.2.4.2, solid units), compression, and sliding at the springing bed joint

$H = w S^2 / (8 f)$ with f the rise of the thrust line; for a jack arch $f = d/3$ and $H = 3 w S^2 / (8 d)$. The drawing is the arch elevation with the line of thrust.

Limitations

- Uniform load only: concentrated loads and live load on half the span (which moves the thrust line) are not covered.
- The thrust line's offset is measured vertically and converted by the axis slope; near the springings of a semicircular arch, where the axis is steep, this approximation is coarse.
- Unreinforced arches only; the wall above acting as a deep beam, out-of-plane loads, rings acting separately, Gothic and elliptical shapes, and settlement of the abutments are not covered. The tie's end anchorage is not designed.

Verification examples

Verification status: 10 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published arch example is on hand: hand calculations of the jack arch thrust ($3\,w\,S^2 / (8\,d)$), a shallow segmental arch's least thrust ($f + d/6 + d/(6\cos\theta s)$), the ring and skewback stresses, the abutment pier, and the tie.

CHAPTER 164

Bond beam chord and collector

A reinforced concrete masonry bond beam at a roof or floor diaphragm: as the **chord** it carries the diaphragm's bending moment as a tension and compression couple, as the **collector** (drag strut) it delivers the diaphragm reaction of a wall line to a shear wall shorter than the line. Wind and seismic, strength or allowable stress design (the switch in the toolbar), with the overstrength factor Ω0 on collectors where ASCE 7-22 12.10.2.1 requires it. US and metric units.

Edition. TMS 402-22 is not on hand: the masonry uses the shared library (the 402-16 text read in the library scan, with the 402-22 numbering of the TMS cross-reference list; to confirm against the code text). ASCE 7-22 12.10 was read in the library copy.

Inputs of the bond beam

- **Bond beam:** what it acts as (chord, collector or both); the wall thickness; 1 or 2 courses, each with 1 or 2 bars of one size; Grade 60 or 40; f′m; coarse or fine grout (the clearance around the bars, TMS 402 6.1.4); epoxy coating (150 % development length); the lap length provided; corner bars and their leg length.
- **Diaphragm:** span L along the chord and depth b between chords; the wind and seismic loads per foot of span at strength level. The seismic load is the diaphragm force Fpx of ASCE 7-22 12.10.1.1 (Eq. 12.10-1, not less than 0.2 SDS Ie wpx).
- **Collector:** the reaction of the wall line, from the same diaphragm (R = w L / 2 on the end wall line, whose length is then the diaphragm depth) or entered with the line length; the shear wall length and whether it sits at one end of the line or in the middle. The diaphragm shear is taken uniform along the line.
- **Seismic:** the seismic design category (Ω0 applies in SDC C to F), Ω0 of ASCE 7-22 Table 12.2-1 for the system (2.5 prefilled, reinforced masonry shear walls), and SDS, Ie and the weight wpx when you want the minimum of Eq. 12.10-2 checked (wpx = 0 skips it).

Auto-design (editor toolbar) varies the courses, bars per course, bar size, lap and corner bar legs. Objective: the least bond beam steel, then one course before two. The diaphragm, loads, wall line, f′m, wall thickness and seismic factors are kept.

Checks performed

Check	Basis
Chord tension and compression	T = C = w L² / (8 b) for each load type, ASCE 7-22 2.3.1 / 2.3.6 or 2.4.1 / 2.4.5 (1.0W, 1.0E or 0.6W, 0.7E); no Ω0 on chords (12.10.1)
Collector tension and compression	F = R (Lc − Lw) / Lc (wall at one end) or half of it (wall centred); seismic × Ω0 in SDC C to F (12.10.2.1)
Tension on the bars	Strength: φ As fy, φ = 0.90 (TMS 402 Table 9.1.4); allowable stress: As Fs (8.3.3.1)

Check	Basis
Compression on the bond beam	Strength: φ 0.80 (0.80 f′m An), φ = 0.65 (9.3.3.1.1, Table 9.1.4); allowable stress: 0.25 f′m An (8.3.4.2.1). Braced continuously by the wall and diaphragm (no slenderness reduction); the bars are not tied and are not counted (9.3.2)
Minimum diaphragm force	Seismic load × L ≥ 0.2 SDS Ie wpx (Eq. 12.10-2), when wpx is entered
Bar size and area	Diameter ≤ t/8 and ≤ 1/3 of the grout space; area at a lap ≤ 8 % of the grout space (6.1.3)
Lap splices and corner bars	ld = 0.13 db² fy γ / (K √f′m) ≥ 12 in. (Eq. 6-1; 402-22 6.1.6.3.1, 6.1.7.1.2), K the least of cover, clear spacing and 9 db; without corner bars an advisory says the force cannot pass the corner

The drawings are the diaphragm plan (chords, the wall line with its shear wall and collector, dimensions) and the bond beam section with its bars.

Limitations

- A simply supported diaphragm under a uniform load; continuous, cantilevered or flexible-rigid diaphragm analyses are yours (enter the reaction of the line instead).
- The bond beam under the chord or collector force only: its action as a lintel, or with the out-of-plane wall bending at the same time, is not combined.
- The diaphragm-to-bond-beam connection (ledger and anchor bolts: use the masonry anchor calculation), the shear wall, transfer forces of irregularities (12.10.1.1) and the 25 % increase of 12.3.3.4 for connections of irregular structures in SDC D to F are not designed.
- The overstrength combinations are applied to the force you enter; the three cases (a) to (c) of 12.10.2.1 are covered when that force is the largest Fpx-based force.

Verification examples

Verification status: 8 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published example is on hand: hand calculations of the chord and collector forces, the bar and masonry strengths and the development length of the default bond beam. TMS 402-22 itself is to be read for the final numbering; the equations are from the 402-16 text (Eq. 6-1, 8.3, 9.3).

CHAPTER 165

Movement joints

Expansion joints of clay brick masonry and control joints of concrete masonry: the movement of the wall over the joint spacing, the strain it puts on the sealant in extension and compression, the joint width needed and the largest spacing a joint width allows, with the spacing limits of the industry guides. US and metric units; a serviceability calculation, so there is no strength or allowable stress switch.

Edition and sources. The movement coefficients are those of TMS 402 4.2.3 to 4.2.5 (read in the 402-16 text; the numbering is unchanged in 402-22). **BIA Technical Notes 18 and 18A and CMHA TEK 10-2C and 10-3 are not on hand:** the closing formula of brick expansion joints, the freezing expansion coefficient, the 25 ft brick spacing and the TEK 10-2C empirical criteria are written from knowledge and marked "to confirm against the code text" in the steps. Every one of them is an input you can change.

Inputs of the movement joints

- **Wall and joints:** clay brick (expansion joints) or concrete masonry (control joints); the wall length (for the number of joints) and height; the joint spacing and width; the minimum joint width (3/8 in. prefilled); the sealant's ASTM C920 class (±25, ±35, ±50 or +100/−50 percent).
- **Temperatures:** the rise of the wall temperature above its installation temperature and the drop below it (of the wall, not the air; dark walls in the sun run hotter).
- **Clay:** the thermal coefficient (4×10^{-6} /°F, TMS 402 Table 4.2.3), the moisture expansion (300×10^{-6}, 4.2.4), the freezing expansion (200×10^{-6} where brick freezes wet, BIA TN 18; 0 in mild climates) and your spacing limit (25 ft, BIA TN 18A).
- **Concrete masonry:** the thermal coefficient (4.5×10^{-6} /°F), the drying shrinkage εs of the units (0.065 %, the ASTM C90 maximum; km = 0.5 εs, 4.2.5.1), the panel length-to-height limit and the spacing limit (1.5 and 25 ft, TEK 10-2C) and the horizontal reinforcement per foot of height.

Auto-design (editor toolbar) varies the joint spacing and width. Objective: the fewest joints, then the narrowest. The wall, temperatures, coefficients, sealant class and spacing limits are kept.

Checks performed

Check	Basis
Sealant compression	Clay: $\Delta c = (ke + kf + kt\,\Delta T)\,S$ with ΔT the full temperature range (BIA TN 18A form); concrete masonry: $\Delta c = kt\,\Delta T_{rise}\,S$. Δc / w at most the compression capability
Sealant extension	Clay: $\Delta o = kt\,\Delta T_{drop}\,S$ (moisture expansion is irreversible); concrete masonry: $\Delta o = (km + kt\,\Delta T_{drop})\,S$. Δo / w at most the extension capability
Joint width	At least the minimum entered
Joint spacing	Clay: at most your limit; concrete masonry: at most min(ratio × H, limit) (TEK 10-2C empirical criteria)

Check	Basis
Horizontal reinforcement (concrete masonry)	Advisory: the empirical spacing assumes at least 0.025 in.²/ft of height

Results: the required joint width, the largest spacing for the width entered and the number of joints in the wall. The drawing is the wall elevation with the joints.

Limitations

- Joints are located at regular spacing; the joints at openings, corners, offsets, changes of height and at the ends of shelf angles (BIA TN 18A, TEK 10-2C) are yours.
- The sealant depth, backer rod, bond breaks, shelf angles, ties and the shear transfer across control joints are not designed.
- Restraint (the wall's friction on its base or flashing) is not credited: the free movement is used.
- The engineered method of TEK 10-2C (spacing from the amount of horizontal reinforcement) is not built in.

Verification examples

Verification status: 11 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published example is on hand: hand calculations of the closing, opening, sealant strains, required width and largest spacing for a brick wall and a concrete masonry wall. BIA TN 18 / 18A and CMHA TEK 10-2C / 10-3 are listed as documents needed; when they arrive, the formula, kf and the spacing limits are to be compared.

CHAPTER 166

Masonry partition wall

A non-loadbearing interior concrete masonry partition spanning from the floor to a braced head, or cantilevered from the floor, under its out-of-plane seismic force as a nonstructural component (ASCE 7-22 13.3 and 13.5) or the 5 psf minimum of ASCE 7-22 4.3.4. Reinforced or unreinforced, TMS 402-22 strength or allowable stress design (the switch in the toolbar), with the slip (deflection) head that lets the structure above deflect. US and metric units.

Edition. ASCE 7-22 4.3.4, 13.1.4, 13.3.1, 13.5.8 and Table 13.5-1 were read in the library copy. TMS 402-22 is not on hand: the wall uses the shared masonry library (402-16 text with the 402-22 numbering where confirmed). MDG-2022 Example 11.3-1 is on hand and verifies the unreinforced allowable stress case.

Inputs of the partition

- **Wall:** height; braced at the head (simple span) or cantilevered; thickness; reinforced or unreinforced; grouting (fully, at the bars only, or none); f′m; mortar (it sets the modulus of rupture and the allowable tension); bar, spacing and grade (bars at the centre); the wall weight with its grout.
- **Lateral loads:** the minimum horizontal load of 4.3.4 (5 psf, applied to walls over 6 ft) as a live load; the dead load that acts with it, which ASCE 7 does not state: 0.6D in allowable stress and 0.9D in strength design as MDG-2022 Example 11.3-1 reasons (prefilled), the combination factor, or none.
- **Seismic:** the seismic design category (exempt in SDC A, and in SDC B with Ip = 1.0, 13.1.4); SDS; Ip; supported above grade plane or at or below it; the height z of the upper attachment and the roof height h (the upper attachment is the conservative reading of z for a wall attached at two levels); Hf by Eq. 13.3-5 or 13.3-4 with Ta; Rμ = 1.3 when the system is not specified, or from R, Ω0 and Ie (Eq. 13.3-6). CAR and Rpo come from Table 13.5-1 by the wall: reinforced masonry partition 1.0 / 1.5; all other partitions 2.2 (at or below grade) or 2.8 (above) / 1.5; cantilever interior walls 1.8 or 2.2 / 1.5. The vertical effect 0.2 SDS acts on the wall weight.
- **Head (slip joint):** the gap; the deflection of the structure above (your analysis); the angle leg length, the minimum engagement (1-1/2 in. prefilled, your choice), the clip spacing, length and thickness and Fy.

Auto-design (editor toolbar) varies the wall thickness, bar size and spacing and the head clip thickness and spacing. Objective: the thinnest wall, then the least steel and clips. Height, loads, seismic data, f′m, wall weight and the head gap are kept.

Checks performed

Check	Basis
Seismic force	Fp = 0.4 SDS Ip Wp (Hf/Rμ)(CAR/Rpo), 0.3 to 1.6 SDS Ip Wp (Eqs. 13.3-1 to 13.3-3), uniform on the wall
Bracing of partitions over 6 ft	A braced head is required (13.5.8.1); a masonry partition cannot meet the exception (10 lb per ft of height)
Flexure, reinforced	Strength: stress block, φ of Table 9.1.4, moment magnifier ψ = 1/(1 − Pu/Pe) with 0.75 In uncracked or Icr cracked (9.3.5.4.3); allowable stress: cracked transformed section (8.3)

Check	Basis
Flexure, unreinforced	Net tension M/Sn − P/An at a bed joint, at most φ fr (Table 9.1.9.2, φ = 0.60) or the allowable tension of Table 8.2.4.2
Shear	Reinforced: 9.3.3.1.2 or 8.3.5 with Mu/(Vu dv) = 1; unreinforced: on the webs (three 3/4 in. webs per unit) and, in allowable stress, sliding on the bed joint against 15 psi (MDG-2022 Example 11.3-1)
Head gap and engagement	Gap ≥ the deflection above; angle overlap at the full gap ≥ the minimum engagement
Clip angle	Reaction w H/2 per clip on a lever arm of the gap plus half the overlap; Mn = Fy Z of the leg (AISC 360-22 F11.1), φ = 0.90 or Ω = 1.67

The drawing is the wall section with the floor, the structure above, the head angles, the gap and the bar.

Limitations

- One foot of a solid wall; openings, intersecting walls and wall ends are not covered.
- The angle's attachment to the structure, the base of the wall (dowels or friction), the wall's in-plane forces and its in-plane drift accommodation (13.3.2) are not designed.
- The allowable stress method does not magnify the moment for P-delta (the wall carries only its own weight).
- The sliding stress limit of 15 psi is the MDG's conservative choice; the full Table 8.2.6.2 value is to be confirmed against the code text.

Verification examples

Verification status: 16 values compared in 2 examples (1 published, 1 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not yet recorded for this module.

Published: MDG-2022 Example 11.3-1 (section properties, flexural and axial stresses, net tension against the 12 psi allowable, shear on the webs and the bed joint). **Hand calculations:** the seismic force of the default wall and its slip head.

CHAPTER 167

Masonry chimney

A rectangular masonry chimney above the roof, cantilevered from the roof line or braced at one level above it, under wind (ASCE 7-22 29.4) and its seismic force as a nonstructural component (ASCE 7-22 13.3). The section is checked at the roof line, at the brace and in the braced span by TMS 402-22, reinforced or unreinforced, strength or allowable stress design (the switch in the toolbar). The prescriptive limits of IBC 2021 2113 (the same as IRC R1003 for dwellings) are checked as reference. US and metric units.

Edition. IBC 2021 2113 and ASCE 7-22 29.4, Figure 29.4-1, 13.3.1 and Table 13.5-1 were read in the library copies. IRC 2018 R1003 is in the library only as an image scan and was not read: its rules are taken as the IBC's (to confirm against the code text). TMS 402-22 is used through the shared masonry library.

Inputs of the chimney

- **Chimney:** outside plan width Bx and depth By; the wall thickness (at least 4 in. nominal, 2113.10); hollow units grouted solid or solid units (sets the modulus of rupture); the masonry unit weight and the weight of the liner and cap per foot; f′m; mortar; reinforced or not; the vertical bar, the number of bars (four at the corners, the rest along the faces Bx) and grade; the tie spacing.
- **Above the roof:** the height above the roof; the roof height at the chimney (it sets qz and z/h); the highest part of the building within 10 ft of the chimney, above the roof penetration (termination, 2113.9); bracing at one level and its height. A brace above the centre of mass makes the chimney "braced above its centre of mass" in Table 13.5-1 (CAR 1.0 instead of 2.2).
- **Wind:** a strength level pressure, or the basic wind speed, exposure, Kzt, Ke, Kd (0.90 for square chimneys) and Cf of Figure 29.4-1 for a square section with the wind normal to a face or along the diagonal (h/D with h the height above the roof and D the least plan dimension), or Cf entered. qz is taken at the centroid of the part above the roof.
- **Seismic:** the seismic design category (exempt in SDC A, and B with Ip = 1.0); SDS; Ip; Hf by Eq. 13.3-5 or 13.3-4 at z/h = 1 (attached at the roof); Rμ = 1.3 or from R, Ω0 and Ie.

Auto-design (editor toolbar) varies the number and size of the bars and the wall thickness. Objective: the least steel, then the thinnest walls. Plan size, height, brace, loads and f′m are kept.

Checks performed

Check	Basis
Wind	p = qz Kd G Cf (Eq. 29.4-1), G = 0.85, on the projected width, both axes
Seismic	Fp = 0.4 SDS Ip Wp (Hf/Rμ)(CAR/Rpo) within 0.3 and 1.6 SDS Ip Wp, uniform with the mass; vertical effect 0.2 SDS on the weight
Statics	Cantilever M = w $H^2/2$; braced at a: propped cantilever with overhang c, M at the roof = w $a^2/8$ − w $c^2/4$, brace reaction (w $H^2/2$ − M)/a, the largest moment between

Check	Basis
Flexure, reinforced	Strength: strain compatibility of the hollow section at Pn = Pu/φ (0.9D or 1.2D with Ev), φ of Table 9.1.4, bars not tied; allowable stress: cracked elastic section with the weight, fb ≤ 0.45 f′m, fs ≤ Fs (8.3)
Flexure, unreinforced	M/S − P/A at most φ fr (Table 9.1.9.2) or the allowable flexural tension (Table 8.2.4.2)
Shear	On the two walls parallel to the load (9.3.3.1.2 or 8.3.5; 9.2.6 or 8.2.6 unreinforced)
IBC 2113	Reinforcement in SDC C to F (2113.3); four No. 4 bars up to 40 in. wide plus two per further 40 in. (2113.3.1); ties at most 18 in. (2113.3.2); walls at least 4 in. nominal (2113.10); at least 3 ft above the roof and 2 ft above the building within 10 ft (2113.9); the roof-line straps of 2113.4 stated

The drawings are the elevation above the roof (with the brace when there is one) and the plan section with the bars.

Limitations

- Rectangular chimneys with one flue; Figure 29.4-1 is for square sections, so for a rectangle the least dimension is used for h/D (enter Cf for a different reading).
- The brace, its connections and the roof-line anchorage (2113.4 straps or a designed connection) are not designed; the brace reaction and the roof-line moment are reported.
- The chimney below the roof, its footing (2113.2), the liner, cap, corbels and offsets are not covered.
- One brace level; the brace is taken as rigid.

Verification examples

Verification status: 9 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published example is on hand: hand calculations of the wind pressure and moment, the weight and seismic force and the statics of the braced chimney. A published worked chimney design (TMS, BIA or CMHA) is listed as a document needed.

CHAPTER 168

Masonry beam and deep beam

A fully grouted reinforced concrete masonry beam on two bearings, or an interior span of a continuous beam, to TMS 402-22 strength design or allowable stress design (the switch in the toolbar). Where the effective span is short for the depth, the beam is designed as a **deep beam** (TMS 402-22 5.3.2) with the internal lever arm of Table 5.3.2.2 and distributed flexural bars. US and metric units. For a lintel with arching action and point loads in the wall above, use the masonry lintel calculation.

Edition. TMS 402-22 is not on hand. The beam and deep beam provisions were read in the 402-16 text (5.2, 5.2.2, Table 5.2.2.2, 9.3.4.2) and carry the 402-22 numbers of the TMS cross-reference list (5.3, 5.3.2, 9.3.3.2); the MDG-2022 sample example 12.4-2A, written to 402-22, is the published check. Items still to confirm against the 2022 text are marked in the steps.

Inputs of the masonry beam

- **Beam:** nominal width; depth from the courses (8n − 3/8 in.) or entered; f′m; the mortar (it sets the modulus of rupture of Table 9.1.9.2 for Mcr); Grade 60 or 40; coarse or fine grout.
- **Reinforcement:** bottom bar size, bars per course and courses with bars, the soffit to the lowest bars; for a deep beam the spacing of the distributed bars over the lower half (bond-beam courses or joint reinforcement, at most dv/5 and 16 in.); top bars for a continuous span; single-leg stirrups and their spacing; the bar embedment past the face of the support.
- **Span and loads:** simple span or interior span of a continuous beam; the clear span and the bearing lengths (span = clear span plus half the bearing at each end, 5.3.1.1); for a continuous span the moment coefficients and the shear factor are yours (1/16, 1/11 and 1.15 prefilled, ACI 318-19 6.5 style); the lateral support spacing of the compression face (0 = braced continuously); the self-weight (81 psf for a fully grouted 8 in. medium-weight wall) and uniform dead, live, roof live and snow loads.
- **Code options:** the shear design section (d/2 from the face, 5.3.1.5, or the face); Mu/(Vu dv) taken as 1.0 or computed; whether the beam supports unreinforced masonry (the l/600 limit of 4.6), the limit and the 8d exemption.

Auto-design (editor toolbar) varies the courses, the courses with bars, bars per course, bar size and the stirrups. Objective: the shallowest beam, then the least steel. Span, loads, width, f′m and code options are kept.

Checks performed

Check	Basis
Deep beam classification	leff = min(centre to centre, 1.15 ln); deep when leff/dv < 2 (simple) or < 3 (continuous) (5.3.2.1, 2.2)
Flexure (beam)	Strain compatibility, 0.80 f′m block, φ of Table 9.1.4 (9.3.2); cracked transformed section with Fs and 0.45 f′m (8.3)
Tension controlled	c at most the depth for εt = εty + 0.003 (9.3.3.2.4), εty = 0.002 for Grade 60

Check	Basis
Minimum flexural strength	Mn ≥ 1.3 Mcr, Mcr = fr b h²/6 with fr parallel to the bed joints, unless As ≥ 4/3 of the required area (9.3.3.2.2.2, .3)
Flexure (deep beam)	Mn = As fy z (Ma = As Fs z), z of Table 5.3.2.2; distributed bars over the tension half at most dv/5 and 16 in. apart (5.3.2.3)
Negative moment (continuous)	Top bars, by the same methods
Shear	Vn = Vnm + Vns, Anv = b d, Vns with dv = h, upper limits of 9.3.3.1.2; 8.3.5 in allowable stress
Stirrups	Required above φVnm; Av/s ≥ 0.0007 b; s ≤ dv/2 and 48 in. (9.3.3.2.3), or dv/5 and 16 in. in a deep beam with horizontal shear steel at least half the vertical (5.3.2.4)
Total reinforcement (deep beam)	ρh + ρv ≥ 0.001 (5.3.2.5)
Lateral support, bearing	Compression face braced at most min(32b, 120b²/d) (5.3.1.3); bearing at least 4 in. (5.3.1.4)
Development	Bottom bars past the face of the support at least ld of Eq. 6-1 (6.1.6.3.1)
Bar limits	db ≤ b/8 and 1/3 of the grout space, area per course ≤ 4 % of the grout space (6.1.3)
Deflection	l/600 under service dead plus live with Ieff of Eq. 5-1 (5.3.1.6, 4.6), where the beam supports unreinforced masonry and the span exceeds 8d

The drawings are the beam elevation with its loads and bars and the beam section.

Limitations

- Uniform loads only; no point loads, no axial load (9.3.3.2.1 limits it to 0.05 An f′m), no torsion.
- Fully grouted beams in running bond; partially grouted beams, stack bond and clay masonry are not covered.
- The bearing stress on the supporting masonry is not checked (masonry bearing calculation); arching action is not taken (masonry lintel calculation).
- Deflection is calculated for simple spans only; for continuous spans the averaged Ieff is yours.
- Deep beams: the minimum vertical shear reinforcement and the total reinforcement are read as ratios (Av/(b s) and ρh + ρv); the 402-16 text prints 0.0007 b d and 0.001 b dv (to confirm against the 2022 text).

Verification examples

Verification status: 18 values compared in 4 examples (2 published, 2 hand calculations from the cited equations): 18 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: not yet recorded for this module.

Published: MDG-2022 Example 12.4-2A (on hand as a sample chapter): factored load, moment, required steel, cracking moment, nominal strength, maximum reinforcement ratio, lateral support limit, and in the rework the moment, design strength (printed as "Mn", the value is φMn), shear at d/2 and Vnm. **Hand calculations:** the allowable moment of the same beam and a deep beam (lever arm, Mn, shear).

CHAPTER 169

Cold-formed steel stud

A lipped C-section (SSMA stud or joist) as a simply supported member: a wall stud in a gravity wall under axial load and out-of-plane wind or seismic load, alone or in a pack, or a floor or roof joist. AISI S240-20 gives the wall framing rules. AISI S100-16 (with Supplement S2-20), Effective Width Method, with ASCE 7-22 load combinations, LRFD or ASD. Every provision was checked against the AISI S100-16 text (with its Commentary) on 1 November 2026.

Inputs of the cold-formed steel stud

- **Section:** an SSMA designation (for example 600S162-54: 6 in. web, 1-5/8 in. flange, 54 mil) or custom out-to-out dimensions D, B and lip d, the design thickness t (minimum base metal thickness / 0.95) and the inside bend radius R. The designations use the SSMA design thicknesses and radii (54 mil: t = 0.0566 in., R = 0.0849 in.) and lips of 1/2 in. (162 flanges) or 5/8 in. (200 flanges and wider). **Fy** is entered (33 ksi for 33 and 43 mil, 50 ksi for 54 mil and thicker are the usual ASTM A1003 grades). **Web holes:** unpunched, or punched with holes centred in the web (shape, depth, length, spacing, nearest distance to a bearing, and how they enter the distortional stress).
- **Studs at each location:** one stud; two back to back, which meet S100 I1.2 (connector spacing a; you confirm the end connections or the S240 B1.3.1 track exception, and the connector strength), or do not, and are then the sum of the two (S240 B1.3.2); or up to six side by side, the sum of the studs.
- **Design basis:** all steel (discrete bridging), or sheathing braced by S240 B1.2.2: identical sheathing on both flanges (or the weaker taken on both), studs at most 24 in. on center. Then Ly = Lt = twice the fastener spacing, K = 1 (B3.2.1(b)), the compression flange is braced (B3.2.2(b)), gypsum limits the stud load (Table B3.2-1, screws at most 12 in. on center), or you enter a limit for other sheathing. The stud is also checked without the sheathing, on the bridging you enter, under 1.2D + (0.5L or 0.2S) + 0.2W with LRFD factors in either method (Eq. B1.2.2-1).
- **Member and bracing:** the length L (simple span) and the spacing, which is the tributary width of the area loads. Kx (Kx Lx = Kx L). The bridging spacing sets Ky Ly = Ky × spacing and Kt Lt = Kt × spacing for the axial check (longest segment) and the unbraced segments for lateral-torsional buckling; 0 means no bridging. Tick *Compression flange braced by sheathing* only when the compression flange is attached to sheathing for every load direction (lateral-torsional buckling then does not apply). Cb is computed per segment by Eq. F2.1.1-2 from each combination's moment diagram, or entered. **Distortional buckling** is computed as prefilled (E4, F4). You may instead determine that it is restrained or not critical: the checks are then left out, as this calculation did before. Without restraint L = Lcrd. *Braces restrict rotation of both flanges* makes Lm the brace spacing (S240 B3.2.1(a)): strap bridging alone does not do that as a rule, and the choice is yours. kφ comes from sheathing (S240 Appendix 1, 0 is permitted; gypsum values there are for serviceability only).
- **Loads (per member):** concentric axial compression per stud by gravity load type (D, L, Lr, S, R); area loads perpendicular to the member by load type (psf, multiplied by the spacing); point loads perpendicular to the member. See Loads and combinations. Axial loads are per stud location: a pack shares them.
- **Floor-to-floor load takedown:** rows of level, load type, pressure and tributary length, each multiplied by the stud spacing and added to the axial loads. A wall weight is its psf and height. Live load reduction (ASCE 7-22 4.7) is an option with KLL. The tributary area is the floor assigned to this stud location (S240 B1.1.1), so below 400 ft² of KLL AT it does not reduce, which is the usual case.

- **Bearing:** end bearing on a support (one-flange end loading, web crippling checked with the bearing length N and whether the flange is fastened to the support) or *Not checked* when the reaction is taken through screws or clips. Point loads are checked for web crippling as one-flange interior loading (end loading when the bearing edge is within 1.5h of an end) when *Web crippling at point loads* is on; N is at least 3/4 in. (G5).
- **Deflection limits:** span / n for live load or wind and for total load. IBC 2021 Table 1604.3 limits for exterior walls are L/360 with plaster or stucco, L/240 with other brittle finishes and L/120 with flexible finishes; floors L/360 live and L/240 total. The wind load for deflection is W × the entered factor (0.42 × the component and cladding load, IBC Table 1604.3 note). 0 turns a check off.

Auto-design (editor toolbar) varies the SSMA C-section (lightest first) and spacing; the yield stress when unlocked. Objectives: least steel per foot of wall or floor; lightest member, then widest spacing; lowest cost index. Kept unless you select them: yield stress Fy. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Section properties	Linear (centerline) method with rounded corners for A, Ix, Sf, Iy; square-corner centerline for xo and Cw (AISI Cold-Formed Steel Design Manual practice)
Axial compression	E2: flexural (x, y), torsional and flexural-torsional buckling (Eq. E2.2-1, β = 1 − (xo/ro)²), Fn by Eqs. E2-2 / E2-3; E3.1: Pnℓ = Ae Fn with the Appendix 1 effective widths at Fn (web k = 4, edge-stiffened flanges by 1.3 with Table 1.3-1, lips k = 0.43); φc = 0.85, Ωc = 1.80
Flexure	F2: Mne = Sf Fn ≤ My, Fcre = Cb ro A √(σey σt)/Sf per bridging segment; F3.1: Mnℓ = Se Fn ≤ Set Fy (Eq. F3.1-1), Se of the effective section with the extreme compression fiber at Fn (web under stress gradient by Appendix 1 1.1.2, flange by 1.3, the lip under stress gradient by 1.2.2, neutral axis iterated), Set with the extreme tension fiber at Fy; φb = 0.90, Ωb = 1.67
Shear	G2.1 / G2.3: λv = √(Vy/Vcr), Vy = 0.6 h t Fy, Vcr of the flat web with kv = 5.34; Vn = Vy, 0.815√(Vcr Vy) or Vcr; φv = 0.95, Ωv = 1.60
Web crippling	G5 Eq. G5-1 with Table G5-2 (single-web channel and C-sections, one-flange end or interior loading, fastened or unfastened), with the R/t, h/t ≤ 200, N/t ≤ 210, N/h ≤ 2.0 and N ≥ 3/4 in. limits
Distortional buckling	E4.1, E4.2 with holes, Pcrd by Appendix 2 2.3.1.3; F4.1, F4.2 with holes, Mcrd by 2.3.3.3, β = 1.0; φ, Ω of compression and flexure
Web holes	Compression Appendix 1 1.1.1 (circular Eqs. 1.1.1-1, -2, or E3.1.2 ignored when n dh / L is at most 0.015; noncircular two strips, k = 0.43), flexure F3.1.2, shear G3, web crippling G6
Sheathing	S240 Table B3.2-1 on the nominal stud load; S240 Eq. B1.2.2-1 without the sheathing
Stud packs	Back to back: E2 of the doubly symmetric section with (KL/r)m, Eq. I1.2-1, a/ri limit I1.2(a), F2 of the assembly, web crippling Table G5-1, H3 Eq. H3-2. Otherwise the sum (S240 B1.3.2)
Combined axial and bending	H1.2: P̄/Pa + M̄x/Max ≤ 1.0, the moment amplified for P-δ by B1 = Cm/(1 − αP̄/Pe1) (Eq. C1.2.1.1-3; α = 1.0 LRFD, 1.6 ASD; Cm = 1.0 for transverse loading; Pe1 with the nominal stiffness and Kx, as the effective length method C1.3 allows)
Combined bending and shear	H2: √((M̄/Maℓo)² + (V̄/Va)²) ≤ 1.0 at every station, Maℓo from F3 with Fn = Fy
Combined bending and web crippling	H3 Eq. H3-1 at interior point loads: 0.91 (P̄/Pn) + (M̄/Mnℓo) ≤ 1.33φ (LRFD, φ = 0.90) or 1.33/Ω (ASD, Ω = 1.70)
Deflection	Live / wind and total, service loads, with the effective moment of inertia at the service moment (stress iterated), against span / n

Check	Basis
Applicability	Table B4.1-1 (Effective Width Method): h/t < 200, flange b/t ≤ 90 when Is ≥ Ia (Ia at Fy) or ≤ 60 otherwise, lip d/t ≤ 60, R/t ≤ 10, do/bo ≤ 0.7, Fy < 80 ksi (and at least 25 ksi); Appendix 1 Table 1.3-1 D/w ≤ 0.8; KL/r ≤ 200 (advisory)

The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the member elevation, drawn horizontal with the bottom or left end at the left (pin supports, the area loads on the tributary width and the point loads by load type, the axial load P at the top end with its values in the sheet notes, the bridging lines, the span), the lipped C section with its depth, flange width, lip and thickness, and the shear, moment and deflection diagrams of the governing combinations aligned under the elevation. End clips, track, bridging members and their anchorage and the sheathing attachment are not part of the calculation and are not drawn.

Limitations

- The Direct Strength Method is not used. Distortional buckling uses the Appendix 2 closed form with β = 1.0.
- The hole conditions the inputs cannot express are yours: holes centred in the web, noncircular corner radii at least 2t, at least 10 in. clear from the end (Appendix 1 1.1.1) and the section depth from a hole to the member end (G3, G6). Hole limits are NOT FEASIBLE when exceeded.
- Sheathing strength other than the gypsum table is your entered limit. S240 Appendix 1 kφ is not computed: you enter it. Force accumulation along bridging and the anchorage are not computed; a note gives the 2 % brace force of B3.4.1.
- Packs: the connectors and end connections of I1.2 are confirmed by you, not designed. Box-type packs are not covered.
- The takedown does not place walls or check load path alignment (in-line framing, S240 B1.2.3) or the track.
- Simply supported members only; concentric axial load (no eccentricity), bending about the strong axis only (no weak-axis bending, torsion from eccentric loads, or axial tension).
- Inelastic reserve capacity (F2.4) is not used. The shift of the effective centroid under axial load is not considered. The member is taken as loaded through the shear center or restrained against twisting by the sheathing (Chapter F scope; H4 torsion is not checked). Combined bending and web crippling (H3) is checked at interior point loads; end reactions of a simple span carry no moment.
- Bridging, its anchorage and the track (AISI S240) are designed separately.
- **Checked against AISI S100-16 (1 November 2026):** E = 29,500 ksi, G = 11,300 ksi; Appendix 1 1.1, 1.1.2, 1.2.1, 1.2.2 and 1.3 with Table 1.3-1; E2, E2.1, E2.2; E3.1; F2.1, F2.1.1; F3.1; G2.1, G2.3; G5 and Table G5-2; H1.2, H2, H3; C1.2.1.1 (B1); Table B4.1-1. Corrections made: Table 1.3-1 now uses 3.57 RI^n + 0.43 for D/w ≤ 0.25; the lip under stress gradient by 1.2.2 (was uniform at its largest stress); Mnℓ limited by Set Fy and Se always at the compression fiber (Eq. F3.1-1); shear written in the S100-16 form (same values); web crippling cites Table G5-2 (the C-section table; Table G5-3 is for Z-sections; the coefficients were already those of G5-2), N ≥ 3/4 in. and end loading measured from the bearing edge; the B4.1 limits completed (R/t, do/bo, Fy < 80 ksi, b/t 90 when Is ≥ Ia); the H3 check added.
- **Checked against the SSMA Product Technical Guide (2022), p. 5:** the SSMA design thicknesses, inside radii and lip lengths of the designations (equal to its Steel Thickness and Stiffening Lip Length tables).
- **Still to verify:** the IBC 2021 Table 1604.3 note for the 0.42 wind factor.

Verification examples

Verification status: 93 values compared in 33 examples (27 published, 6 hand calculations from the cited equations): 91 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published values: the **SSMA Product Technical Guide** (Steel Stud Manufacturers Association, 2022, free from ssma.com; computed to AISI S100-07/S2-10 and S100-12, ASD) tabulates the gross and torsional properties, the effective section modulus Sxe and allowable moment Mal for local buckling, the allowable shear Vag, the effective moment of inertia Ixe for deflection (at the allowable moment) and the allowable web crippling loads of the SSMA studs. The SSMA examples run a 10 ft member with the compression flange braced (Fn = Fy, local buckling only, as Mal is defined), ASD; for Ixe the wind service moment is set equal to Mal.

Example	What it checks	Published (SSMA)	ILX Calcs
SSMA-600S162-54	50 ksi: A, Ix, Sx, Rx, Iy, Ry, J, Cw, Xo, m, Ro, β, Vag (p. 10)	0.556, 2.860, 0.953, 2.267, 0.180, 0.570, 0.594 × 10^{-3}, 1.337, −1.049, 0.663, 2.562, 0.832, 2,823 lb	0.556, 2.860, 0.953, 2.268, 0.181, 0.570, 0.594 × 10^{-3}, 1.337, −1.049, 0.663, 2.562, 0.833, 2,822
SSMA-600S162-54	Sxe, Mal (deviations SSMA-SXE, SSMA-COLD-WORK)	0.916 $in.^3$, 30.33 kip-in. (includes cold work)	0.928, 27.80
SSMA-600S162-33	33 ksi: A, Ix, Sx, Iy, J, Cw, Xo, m, Ro, β, Sxe, Mal, Vag (p. 10)	0.344, 1.793, 0.598, 0.116, 0.137 × 10^{-3}, 0.861, −1.072, 0.677, 2.587, 0.828, 0.577, 11.41 kip-in., 638 lb	0.344, 1.793, 0.598, 0.116, 0.137 × 10^{-3}, 0.861, −1.072, 0.677, 2.587, 0.828, 0.578, 11.42, 638
SSMA-800S162-54	50 ksi: A, Ix, Sx, Sxe, Mal, Vag, Ixe (p. 11)	0.670, 5.736, 1.434, 1.229, 36.79 kip-in., 2,091 lb, 5.600 $in.^4$	0.670, 5.736, 1.434, 1.230, 36.83, 2,091, 5.600
SSMA-1000S162-54	50 ksi: Ix, Sxe, Mal, Vag, Ixe (p. 12)	9.950, 1.572, 47.07 kip-in., 1,661 lb, 9.391 $in.^4$	9.950, 1.573, 47.11, 1,660, 9.391
SSMA-800S162-43	33 ksi: Sxe, Mal, Vag (p. 11)	1.019, 20.14 kip-in., 1,051 lb	1.019, 20.14, 1,051
SSMA-600S200-33	33 ksi: Mal, Ixe (p. 11)	12.28 kip-in., 2.058 $in.^4$	12.29, 2.059
SSMA-WC-600-54	Web crippling, Condition 1 (end, one flange, fastened), 54 mil, 50 ksi, N = 1 / 3.5 / 6 in. (p. 59)	599, 909, 1,116 lb	599, 909, 1,116
SSMA-WC-600-33	Same, 33 mil, 33 ksi, N = 3.5 / 6 in.	240, 297 lb	240, 297

Deviations (`docs/verification/deviations.json`): SSMA-COLD-WORK, the SSMA allowable moments marked with footnote 2 include the cold work of forming (S100 A7.2), which ILX does not use; SSMA-SXE, the SSMA Sxe of the 600S162-54 at 50 ksi is 1.4 % below the S100 effective section that ILX and the independent hand calculation CS-4 both give. The narrow 362S162 sections of the guide also list Sxe below the S100 effective section (0.444 against 0.468 $in.^3$ at 50 ksi) and are not used as examples.

Compression and combined load. The SSMA Combined Axial and Lateral Loads tables reproduce only with the standard 1-1/2 × 4-1/2 in. punchout in the effective area (Appendix 1 1.1.1) and the H1.2 interaction with the B1 amplifier, so they verify the hole provisions and H1.2 together. Distortional buckling did not govern any of these rows.

Example	What it checks	Published (SSMA p. 32)	ILX Calcs
SSMA-AX	12 allowable axial loads, 8 to 16 ft, 800S162-43 to -97, 5 psf, 4 ft bracing, punched	for example 800S162-54, 12 ft at 16 in.: 5.24 kips	within 0.3 %; 5.25 kips here
SSMA-MAD	Mad at kφ = 0 through this module (pp. 10, 11)	32.81, 18.33, 9.47 kip-in.	as published

Not published, hand calculations only: E4 and E4.2 (hand values in the test suite), kφ, sheathing braced design, Eq. B1.2.2-1, stud packs (I1.2, the built-up Cw), takedown and live load reduction. The AISI Cold-Formed Steel Design Manual (D100) and Framing Design Guide (D110) would close that (research/DOCUMENTS_NEEDED.md).

The S100-16 Commentary gives the yield load and yield moment of the Manual's 9CS2.5x059 (Figure C-2.2.2-2(a), Commentary p. 2-5). The strengths are also verified by hand calculations in closed form (`backend/tests/test_verify_cfs_stud.py`): the section properties by the Manual's linear-method formulas (u = 1.57r, centroid 0.637r, own inertia 0.149r³) and its square-corner formulas for m and Cw, and the strengths by the S100-16 equations, independent of the engine. 600S162-54, Fy = 50 ksi, 12 ft, 16 in. on center, bridging at 48 in.:

Example	What it checks	Hand calculation	ILX Calcs
CS-1	Gross properties	A = 0.5563 in², Ix = 2.861 in⁴, Iy = 0.1805 in⁴, m = 0.6634 in., Cw = 1.337 in⁶	0.5563, 2.860, 0.1805, 0.6634, 1.337
CS-2	Axial, Kx Lx = 12 ft, Ky Ly = Kt Lt = 48 in.	Fe = 39.86 ksi, Fn = 29.58 ksi, Ae = 0.3999 in², φcPn = 10,055 lb, Pn/Ωc = 6,572 lb	39.86, 29.58, 0.3999, 10,055, 6,572
CS-3	Flexure, Cb = 1.0; shear; wind deflection (30 psf)	φbMn = 37,530 lb-in, Mn/Ωb = 24,970 lb-in, φvVn = 4,291 lb, Δ = 0.0929 in.	37,534, 24,973, 4,290, 0.0929
CS-4	Flexure, flange braced (Fn = Fy)	Se = 0.9284 in³, φbMn = 41,778 lb-in	0.9285, 41,781
CS-5	Web crippling, one-flange end, fastened, N = 1.5 in.	φwPn = 1,010 lb, Pn/Ωw = 679 lb	1,010, 679
CS-6	Combined axial (1,000 lb D) and wind (30 psf)	0.357 (LRFD), 0.368 (ASD)	0.357, 0.368
C-2.2.2-2a	9CS2.5x059, Fy = 55 ksi (published, S100-16 Commentary Figure C-2.2.2-2(a))	Py = 48.42 kips, My = 126.55 kip-in.	48.47, 125.92

The hand calculations agree within 0.1 %. The published 9CS2.5x059 values come from a finite strip (centerline) model; ILX uses the linear method with rounded corners and agrees within 0.2 % (Py) and 0.6 % (My).

CHAPTER 170

Cold-formed steel shear wall

A Type I cold-formed steel shear wall segment sheathed with wood structural panels or steel sheet: nominal unit shear from the sheathing and screw spacing, the aspect-ratio rules, the overturning forces on the chord studs and the hold-downs. AISI S400-20 (seismic) and AISI S240-20 (wind), chord studs by AISI S100-16, ASCE 7-22 load combinations, LRFD or ASD. The seismic provisions and tables were checked on 1 November 2026 against AISI S400-15 with Supplement S1-16, the edition on disk; S400-20 is taken as unchanged in them until it is available.

Inputs of the cold-formed steel shear wall

- **Wall:** the segment length w and height h, and the stud spacing (the chord carries the gravity load of half a spacing).
- **Sheathing and screws:** 15/32 in. Structural 1 sheathing (4-ply), 7/16 in. OSB or 0.018 in. steel sheet, on one side or on both sides (same sheathing and screws: the strengths add), the edge screw spacing (6, 4, 3 or 2 in.; field 12 in.) and the stud and track thickness. The seismic value is read from the S400 table row for that thickness: wood structural panel rows apply to the listed thicknesses only (S400 Table E1.3-1 note 5), the steel sheet row to 33 mil or thicker; where two rows apply the larger value is used when the wall's aspect ratio is within the row's limit; the 68 mil rows require No. 10 screws (No. 8 otherwise). For any other assembly of the tables (other steel sheets) or a tested assembly, choose *Entered* and give the nominal unit shear for wind and for seismic (one side) and the maximum aspect ratio.
- **Loads:** the in-plane shears W and E at the top of the wall (strength level, E including ρ; both directions are checked) and the gravity line loads on the wall by load type. The dead load resists overturning with the factor of each combination (0.9D, 0.6D); every gravity load adds to the chord compression. **Seismic amplification for chords and hold-downs:** Ω0 = 3.0 is prefilled (ASCE 7-22 Table 12.2-1 for cold-formed steel light-frame walls sheathed with wood structural panels or steel sheet); the seismic part of the chord and hold-down forces is multiplied by it. **SDS** adds the vertical seismic effect Ev = 0.2SDS D (ASCE 7-22 12.4.2.2) to the seismic combinations: (1.2 + 0.2SDS)D and (0.9 − 0.2SDS)D in LRFD (2.3.6), (1.0 + 0.14SDS)D, (1.0 + 0.105SDS)D and (0.6 − 0.14SDS)D in ASD (2.4.5). It is 0 by default (Eh only, as in SDC B); a note says so while E is present.
- **Chords and hold-downs:** the chord stud (SSMA designation), the number of studs per chord, Fy, the bracing spacing of the chord (Ky Ly = Kt Lt; Kx Lx is the wall height), the distance from the wall end to the chord centroid e (d = w − 2e), and the hold-down's design strength (LRFD) or allowable strength (ASD) from its evaluation report for the stud thickness used.

Auto-design (editor toolbar) varies the sheathing, one or both faces, edge screw spacing, framing thickness and the chord studs (section and count). Objectives: lowest cost index (sheathing, screws, framing, chords). Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Unit shear, wind	v = V/w against φ vn (φ = 0.65) or vn/Ω (Ω = 2.00), vn from AISI S240-20 Table B5.2.2.3-1

Check	Basis
Unit shear, seismic	v = E/w against φ vn (φ = 0.60) or vn/Ω (Ω = 2.50, S400 E1.3.2 / E2.3.2), vn from S400 Table E1.3-1 (wood structural panels) or E2.3-1 (steel sheet) by stud and track thickness
Aspect ratio	Full strength to h/w = 2:1; rows marked for it (S400 table note 3) to 4:1 with vn × 2w/h (Eqs. E1.3.1.1-2 / E2.3.1.1-2); the others, and 0.018 in. steel sheet, to 2:1
Wall length	At least 24 in. (S400 E1.3.1.1 / E2.3.1.1)
Framing thickness	Studs, track and chord studs at least the assembly's minimum thickness; seismic rows by thickness
Hold-down tension	T = (V h − fD wD w (w/2 − e))/d for every combination with W or E, against the entered hold-down strength
Chord stud tension	S100-16 D2: n Ag Fy, φt = 0.90, Ωt = 1.67
Chord stud compression	C = V h/d + the gravity load on the chord; n × Pn of a single stud by S100-16 E2 / E3 (as the cold-formed steel stud), φc = 0.85, Ωc = 1.80

The drawings (on screen, in the report and on the PDF / DXF drawing sheet) are the shear wall elevation (the wall with its chord studs at each end, the gravity loads by type, the in-plane shear V at the top, the hold-down location (T) and the compression chord (C), length and height) and the chord stud section. The sheathing, screw schedule, stud spacing, loads by type and the governing chord forces T and C with their combinations are in the sheet notes. The hold-down product, anchor rods, track anchorage, collectors, the sheathing panel layout and the arrangement of a built-up chord are not part of the calculation and are not drawn.

Limitations

- Shear wall deflection and drift (S400 E1.4.1.4 / E2.4.1.4; Eq. E1.4.1.4-1 in S400-15) are not computed; check the story drift separately.
- Type I walls with blocked sheathing only; Type II (perforated) walls, unblocked sheathing, gypsum board, fiberboard and strap-braced walls are not covered.
- The seismic chord and hold-down forces use the amplified load (Ω0 × E). S400 E1.4.1.2 / E2.4.1.2 take the required strength from the wall's expected strength ΩE Vn (ΩE = 1.8, E1.3.3 / E2.3.3) but not more than the load with overstrength; the expected-strength value is not computed, so the module's force is never less than the required one (conservative).
- Built-up chords are taken as n individual studs, each braced at the entered spacing (no composite action). The chord studs are checked for axial load only (no out-of-plane wind bending of an exterior chord). Net-section fracture at the hold-down screws, the screws to the hold-down, the shear anchorage, the track and the collectors are not checked.
- **Checked against AISI S400-15 with S1-16 (1 November 2026):** the seismic unit shears of Tables E1.3-1 and E2.3-1 (U.S.), φ = 0.60 and Ω = 2.50, the summing of two sheathed sides (E1.3.1.1.2 / E2.3.1.1.3), the 2w/h rule to 4:1, the 24 in. minimum length and the capacity design of chords and hold-downs (E1.4.1.2). Corrected: the seismic values now follow the table rows by stud and track thickness (the module had used one row per sheathing taken from AISI S213-07, with 3 in. and 2 in. values the S400 table does not give for 33 mil framing, and 700 / 780 plf for every thickness: 7/16 in. OSB on 43 or 54 mil framing is 825 plf at 6 in.), and the 4:1 aspect ratio only for the rows that allow it.
- **Still to verify:** the same tables and factors in S400-20; the wind values and φ / Ω of AISI S240-20 Table B5.2.2.3-1 (not on disk; values from AISI S213-07); the equation number of the deflection equation in S400-20.

Verification examples

Verification status: 27 values compared in 8 examples (6 published, 2 hand calculations from the cited equations): 27 within tolerance, 0 recorded deviations. Code basis: written from an older edition, with the differences noted. Not yet compared line by line with the target edition.

The seismic unit shears are checked against the published values of AISI S400-15 with S1-16 (Table E1.3-1, p. 19; Table E2.3-1, p. 30; Eq. E1.3.1.1-2 for 2 < h/w ≤ 4, p. 17; both faces E1.3.1.1.2, p. 18; φv = 0.60 / Ωv = 2.50, E1.3.2, p. 21) in S400-T1 to S400-T6. The arithmetic that follows (overturning, hold-down, chord studs) is checked by hand calculations in closed form (`backend/tests/test_verify_cfs_shearwall.py`). No free published worked example of a cold-formed steel shear wall design was found (the AISI D113 Shear Wall Design Guide is sold), so the chord and hold-down forces are compared with hand calculations only, and deflection is not computed by the module.

Example	What it checks	Published (S400-T) or hand calculation (SW)	ILX Calcs
SW-1	8 ft × 9 ft, 7/16 in. OSB one side, 6 in. edges, W = 4 kips, D = 0.3 kip/ft, L = 0.4 kip/ft, two 600S162-54 chords	v = 500 plf, φvn = 591.5 plf; T = 3.62 kips (0.9D + 1.0W); C = 5.20 kips; φcPn = 20.35 kips; ASD v = 300 plf, vn/Ω = 455 plf, T = 2.10 kips	500, 591.5, 3.62, 5.20, 20.35, 300, 455, 2.10
SW-2	4 ft × 10 ft (h/w = 2.5), Structural 1 both sides, 4 in. edges, E = 3 kips, Ω0 = 3.0	v = 750 plf, φvn = 0.60 × 990 × 2 × 0.8 = 950.4 plf; T = 24.19 kips; ASD v = 525 plf, vn/Ω = 633.6 plf, T = 16.94 kips	750, 950.4, 24.19, 525, 633.6, 16.94
S400-T1	7/16 in. OSB, 43 mil, 6 in. edges (S400-15 Table E1.3-1, p. 19)	vn = 825 plf: φvn = 495, vn/Ω = 330 plf	495, 330
S400-T2	7/16 in. OSB, 54 mil, 4 in. edges, h/w ≤ 2	vn = 1,410 plf: φvn = 846 plf	846
S400-T3	15/32 in. Structural 1, 68 mil, 2 in. edges	vn = 2,190 plf: φvn = 1,314 plf	1,314
S400-T4	0.018 in. steel sheet, 43 mil (33 min.), 6 in. edges (S400-15 Table E2.3-1, p. 30)	vn = 390 plf: φvn = 234 plf	234
S400-T5	15/32 in. Structural 1, 33 mil, 4 in. edges, 4 ft × 12 ft (h/w = 3, footnote 3 row): vn × 2w/h	vn = 990 plf × 8/12 = 660 plf: φvn = 396, vn/Ω = 264 plf	396, 264
S400-T6	7/16 in. OSB both faces, 33 mil, 6 in. edges (E1.3.1.1.2: the faces add)	2 × 700 = 1,400 plf: φvn = 840, vn/Ω = 560 plf	840, 560

All values agree within 0.01 %.

CHAPTER 171

Cold-formed steel joist

A lipped C-section floor or roof joist on any supports, to the AISI S100-16 North American Specification with Supplement S2-20, by the Effective Width Method, with ASCE 7-22 load combinations. Simple spans, continuous spans, overhangs and cantilevers, an unpunched or punched web, flexure with distortional buckling, shear, web crippling at each bearing, the combined checks and deflection.

Inputs of the cold-formed steel joist

- **Section:** an SSMA designation (800S162-54 is an 8 in. web, 1-5/8 in. flange, 54 mils) with its design thickness, inside radius and lip from the SSMA tables, or custom dimensions. The yield stress is yours: 33 ksi for 33 and 43 mils and 50 ksi for 54 mils and thicker are the usual values. Nothing is added for the cold work of forming (A3.3.2): if you take that increase, enter the raised value (see Verification).
- **Web holes:** unpunched, or punched with holes centred in the web: noncircular (depth and length, the SSMA standard punchout is 1-1/2 × 4-1/2 in.) or circular, their spacing, and the nearest distance from a hole to the edge of a bearing (for the web crippling factor). You also choose how the holes enter the distortional buckling stress (Appendix 2 2.3.4.3): patterned along the web (Eq. 2.3.4.3-2, prefilled because the input describes a regular pattern), or a hole within the buckling wavelength (Eq. 2.3.4.3-1, up to about 4 % lower on Mnd for short wavelengths).
- **Length and supports:** the joist length and a table of supports (position, pin or fixed, flange bearing or clip, bearing length N), and the joist spacing, which is the tributary width of the area loads.
- **Loads:** area loads (on the joist spacing, full length or part), line loads (uniform or linearly varying) and point loads, each with its load type (D, L, Lr, S, R, W, E), plus the optional self-weight. Every ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combination of the types present is analysed. On more than one span or overhang, *Pattern live load* places the live load L span by span (ASCE 7-22 4.3.3). See Loads and combinations.
- **Flange bracing:** for the top flange (in compression under positive moment) and the bottom flange (in compression over interior supports, on overhangs and under uplift): continuous bracing by sheathing, or bracing at a spacing you enter (0 = at the supports only). K applies to both Ky and Kt. Cb is computed from the moment diagram of each segment (1.0 on a segment with a free end) or entered.
- **Distortional restraint:** the rotational restraint kφ of each flange (0, prefilled, is the unrestrained value and the one the SSMA tables use; a higher value is your own, from the sheathing and its fastening), and the moment gradient factor β: 1.0, which the code permits, or Eq. 2.3.3.3-3 for segments between brace points. A brace point is taken as a distortional restraint too (Lm = the segment).
- **Web crippling:** whether the flanges are fastened to the supports, and whether to check point loads and their bearing length.
- **Deflection:** live load and total load limits as span / n (the IBC 2021 Table 1604.3 floor values 360 and 240 are prefilled; 0 turns a check off). Overhangs are checked on twice their length (note h of the table).

Auto-design proposes the lightest SSMA designation and, where it helps, a wider spacing; the yield stress stays locked unless you unlock it, and a custom section is kept as entered.

Checks performed

Check	Basis
Flexural strength, positive and negative moment	F2 yielding and lateral-torsional buckling per unbraced segment, F3.1 local buckling (Se at Fn, Appendix 1), both φb = 0.90, Ωb = 1.67
Distortional buckling, positive and negative moment	F4.1, Fcrd by Appendix 2 2.3.3.3, with holes 2.3.4.3 and F4.2
Shear strength	G2 (kv = 5.34), with holes G3 (qs), φv = 0.95, Ωv = 1.60
Combined bending and shear	H2, Eq. H2-1, at every station of every load case
Bearing stiffener at a support, and its screws	F5.2 (C-section stud or track, two-flange) or F5.1; J4.3
Clip screws at a support, when entered	J4.3. The clip itself and its attachment to the support stay yours
Web crippling at each flange-bearing support and point load	G5, Table G5-2, one-flange end or interior loading, fastened or not; with holes G6 (Rc)
Combined bending and web crippling	H3, Eq. H3-1, at interior supports and interior point loads
Live load and total load deflection	ASD service combinations, the effective moment of inertia at the service moment
Limits	Table B4.1-1 and Table 1.3-1 (section), G3, G6 and Appendix 1 1.1.3 (holes), G5 (R/t, h/t, N/t, N/h, N at least 3/4 in.)

End or interior loading. A bearing whose edge is within 1.5 h of the member end is end loading; otherwise interior. The overhang increase of Eq. G5-2, which the code permits as an alternative, is not taken.

Holes. For a hole depth below 0.38 h the web is taken without the hole in local buckling; at 0.38 h or more, the compression part of the web beside the hole is an unstiffened element at f1 (Appendix 1 1.1.3(a)). No reduction of the lateral-torsional buckling stress is needed within those hole limits (F2.2 exception). Deflection ignores holes (1.1.3(b)). The shear check and H2 take a hole at the point of largest shear, wherever the holes are.

Shear at a hole. S100-16 G3 reads "in accordance with Section G2, with Vcr computed using G2.3, multiplied by the reduction factor". The calculation multiplies the G2 nominal strength by qs, the wording of S100-12 C3.2.2 and what the SSMA Vanet values reproduce. Where the web is in the inelastic range of G2 this is the lower reading (a 600S162-68 at 50 ksi: 2,880 lb against 3,075 lb if qs multiplied Vcr alone).

What the calculation does not decide. You give the bracing and restraint values; the bridging, blocking and their anchorage (AISI S240), and the sheathing attachment behind kφ, are yours. The hole conditions the inputs cannot express are yours to confirm: holes centred in the web, noncircular corner radii at least 2t, and at least the section depth from the member end to a hole (G3, G6, Appendix 1 1.1.3). The calculation states them in a note.

The drawings show the joist elevation with its supports, loads and brace points, the C-section, and the shear, moment and deflection diagrams of the governing combinations. The notes give the web holes, which are not drawn.

Limitations

- Single lipped C-sections bent about their major axis. Back-to-back or built-up joists, tracks, Z-sections and axial load are not covered. Inelastic reserve (F2.4, F3.1.3, F4.3) and the Direct Strength Method are not used.
- A bearing stiffener: hand calculations only, no published example on hand. Its bearing on the support, web holes beside it and H3 at a stiffened support are not checked. CFSEI F100 and F104 are not on disk. The stiffener's

own screws take all of the reaction, on the conservative side. The joist's Fu for these screws is your entry, or 0 pairs it with Fy as the SSMA Guide does: 45 ksi with 33, 65 with 50.

- Two-flange web crippling is not checked. A point load within 1.5 h of a flange-bearing support gets a note saying so.
- Bearing stiffeners and clips are not designed: a support marked as a clip has no web crippling check, and a hole within the bearing needs a stiffener (G6), which you confirm by the hole distance you enter.
- Floor vibration is not checked, and the diaphragm role of the sheathing is not covered.

Verification examples

Verification status: 71 values compared in 36 examples (36 published, 0 hand calculations from the cited equations): 69 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published worked example of a joist to S100-16 was on hand (the AISI Cold-Formed Steel Design Manual is listed as a document needed). The calculation is compared with the published tables of the SSMA Product Technical Guide (2022), which are computed to S100-07 / S2-10 and S100-12, ASD; the provisions compared read the same in S100-16.

Source	What it checks	Published	ILX Calcs
p. 11, 800S162-54, 50 ksi	Mal, Mad, Vag	36.79, 32.81 kip-in, 2,091 lb	36.83, 32.81, 2,091
p. 11, 800S162-43, 33 ksi	Mal, Mad, Vag	20.14, 18.33 kip-in, 1,051 lb	20.14, 18.33, 1,051
p. 11, 800S200-54, 50 ksi	Mal, Mad	44.87, 37.37 kip-in	44.99, 37.37
p. 10, 600S162-33, 33 ksi	Mal, Mad	11.41, 9.47 kip-in	11.42, 9.47
p. 13, 1200S162-68, 50 ksi	Mal, Mad	79.19, 66.14 kip-in	79.19, 66.14
p. 10, 600S162-54, 50 ksi	Vanet at the standard punchout	1,947 lb	1,947
p. 10, 600S162-68, 50 ksi	Vanet	2,879 lb	2,880
p. 59, 600 / 54 mil	Interior one-flange, fastened, N = 1, 3.5, 6 in.	1,295, 1,713, 1,991 lb	1,295, 1,713, 1,991
p. 59, 800 / 54 mil	End one-flange, fastened, N = 1, 3.5, 6 in.	575, 872, 1,070 lb	575, 872, 1,070
p. 43, 800S162-54 at 16 in.	Span at L/360 and L/480 live load, 20 psf	21 ft 1 in., 19 ft 2 in.	21 ft 1 in., 19 ft 2 in.

The full list is on the Verification page. **Rows with footnote 2.** Where the guide marks a section "includes cold work of forming", its Mal and Mad use a raised yield stress: for 600S162-54 at 50 ksi it prints 30.33 and 25.90 kip-in, the calculation at 50 ksi gives 27.80 and 24.23 (a listed deviation, the calculation is lower). With the raised value worked out from the same row (30.33 × 1.67 / 0.916 = 55.3 ksi) it gives 30.35 and 25.89.

Not compared. The double-span columns and the moment-governed rows of the span tables exceed the guide's own Mal by 10 to 14 % and cannot be reproduced from the section values, so they are not used.

Hand calculations. No published value covers kφ above 0, Lm below Lcrd, β, the hole terms of Appendix 2 2.3.4.3, F4.2, G6 or Appendix 1 1.1.3: those are checked in the test suite against hand calculations from the cited equations, not against a publication.

CHAPTER 172

Cold-formed steel header

A header over a wall opening, to AISI S240-20 Section B3.3 and the AISI S100-16 North American Specification with Supplement S2-20, with ASCE 7-22 load combinations. Two C-sections back to back or boxed, double and single L-headers, and inverted L assemblies, on a simple span.

Inputs of the cold-formed steel header

- **Header:** back to back (webs together, S240 B3.3.1), box (webs outside, tracks top and bottom, B3.3.2), double L (B3.3.3), single L (B3.3.4), with or without inverted angles below the track (B3.3.5).
- **Section:** an SSMA designation or custom dimensions for the C-sections; a designation such as 800L150-54 (8 in. vertical leg, 1-1/2 in. top flange, 54 mils) or custom dimensions for the angles. The yield stress is yours. Nothing is added for cold work.
- **Span and loads:** the span, and tributary area loads (a pressure times a tributary width), line loads and point loads, each with its load type. Every ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combination of the types present is analysed. Downward is positive. Enter wind uplift as a W load: W acts in both directions.
- **Web holes** (C-sections): unpunched, or punched with holes centred in the web, their shape, size, spacing and nearest distance to a bearing.
- **Web crippling** (C-sections): end bearing on the jack studs with its length, or clips (not checked); whether the flanges are fastened; point loads and their bearing length. For a box: the track thickness and flange, and whether the track flanges sit over the C-sections. S240 Eq. B3.3.2.3-1 applies only then, and its thickness tt is fixed at 0.0346 in. in the Standard. You also choose S240 Eq. B3.3.2.5 or S100 Eq. H3-1 for bending with web crippling: S240 permits either.
- **Load points** (L-headers): that a cripple stud sits at every load point, the bearing length and the wall width. S240 limits them.
- **Web connections** (back to back): the connection spacing s, the distance g between the rows nearest the flanges, the screws at each connection, their size and the tensile strength (0 takes the ASTM A1003 minimum: 45 ksi for 33 ksi steel, 65 ksi for 50 ksi steel).
- **Distortional buckling:** S240 B3.3 evaluates bending by S100 F3 with the full yield stress, so F4 is off as prefilled. You may turn it on, with a rotational restraint kφ. The SSMA header tables do limit by F4, so with F4 off the result is higher than those tables where F4 governs.
- **Deflection** (C-sections): live load and total load limits as span / n. 0 turns a check off.

Auto-design proposes the lightest listed C-section or angle; the header kind stays, the yield stress stays locked unless you unlock it, and a custom section is kept as entered.

Checks performed

Check	Header	Basis
Flexural strength	Back to back, box	S240 B3.3.1.1, B3.3.2.1: S100 F3 with Fn = Fy, twice one C; φb = 0.90, Ωb = 1.67
Distortional buckling, if you turn it on	Back to back, box	S100 F4, Appendix 2 2.3.3.3, twice one C
Shear strength	Back to back, box	S100 G2, with holes G3, two webs
Combined bending and shear	Back to back, box	S100 Eq. H2-1, every station
Web crippling at the ends and point loads	Back to back	S100 G5, Table G5-1 per web, with holes G6
Web crippling at the ends and point loads	Box	S100 G5, Table G5-2 for single webs; interior one-flange loading times α, Eq. B3.3.2.3-1, with φ = 0.85, Ω = 1.80
Combined bending and web crippling	Back to back	S100 Eq. H3-2
Combined bending and web crippling	Box	S240 Eq. B3.3.2.5 or S100 Eq. H3-1
Web connection spacing and tension at point loads	Back to back	S240 B1.3, S100 I1.1 Eqs. I1.1-1, I1.1-2; screw pull-out J4.4.1
Live and total load deflection	Back to back, box	Two C-sections with the effective inertia at the service moment, tracks ignored (S240 Commentary B3.3.2)
Gravity flexure	Double, single L	S240 Eqs. B3.3.3.1.1, B3.3.4.1.1; φ, Ω by leg
Uplift flexure	Double L	S240 Eq. B3.3.3.1.2: Mnu = R Mng, φ = 0.80, Ω = 2.0
Inverted assembly	Double, single L	S240 B3.3.5.2, B3.3.5.3
Limits (a) to (j)	Double, single L	S240 B3.3.3, B3.3.4: NOT FEASIBLE outside them

L-header section. Mno is the effective section modulus with the extreme compression fiber at Fy times Fy (S240 Commentary Eq. C-B3.3.3-1): yielding of the tension fiber is not a limit state. The top flange is an unstiffened element. The vertical leg, with its free edge in tension, takes Appendix 1 Eqs. 1.1.2-3 to -5 whatever its depth, as the Commentary says.

Inverted double L, a reading. S240 B3.3.5.2 sums the gravity and uplift nominal strengths and names no factor for the sum. The calculation applies each part's own factor. It is stated in the check. Confirm it if you rely on it.

Web connections at a point load. S100 Eq. I1.1-2 asks the connection nearest a short-bearing point load to take Ps s m / (2 g). Screw pull-out in thin webs is small, so this check often fails at cripple stud loads with the S240 Figure C3.4.4-1 minimum screws. Add screws at the load or a reinforcing plate (I1.1(b)), or take the load through a distribution member and enter it as a line load if that is what your detail does. Turning the check off is your decision as engineer of record: S240 B1.3 sends built-up sections to I1.

The drawings show the elevation with the loads, the assembly section, and the shear, moment and deflection diagrams. Upward loads are in the diagrams but not on the elevation.

Limitations

- Simple span, bending about the major axis. Out-of-plane load on the header is not covered.
- Bracing is the Standard's: F3 at the yield stress takes the header as braced by its tracks and cripple studs. Lateral-torsional buckling is not computed.
- Tracks add no strength or stiffness, except α. Jack, king and cripple studs, tracks and their screws are not designed (S240 Figures C3.4.4, C3.4.5).
- **L-headers get no deflection number.** The S240 Commentary says the procedure is undefined, because the header is an indeterminate assembly; tests measured less than L/240 at nominal load.
- L-headers get no shear or web crippling check, as S240 allows. The Commentary suggests considering shear below a span to depth of 3: an advisory says so.
- A single L-header has no uplift strength (S240 B3.3.4.1.2 is reserved): uplift on one without inverted angles is NOT FEASIBLE.
- Screw pull-over and screw tension are not computed for the web connections. Two-flange web crippling is not checked. Holes in angles are not covered.

Verification examples

Verification status: 48 values compared in 38 examples (35 published, 3 hand calculations from the cited equations): 39 within tolerance, 9 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published worked example of a header to S240 was on hand (research/DOCUMENTS_NEEDED.md).

Source	What it checks	Published	ILX Calcs
SSMA Product Technical Guide (2022), p. 56	15 allowable uniform loads of two C-sections, 3 to 12 ft, shear, F4 and L/360 governing	for example 2 × 800S162-54 at 8 ft: 683.6 plf	683.6; 14 within 1 %, one at 1.03 % (CFS-SSMA-IXE)
SSMA p. 60	16 web crippling loads, back to back, Conditions 1 and 2	800S162-54, N = 3.5 in.: 4,200 and 5,477 lb	within 1 %
CFSEI DG-8 (2000), Table 3, p. 7	12 double L-header spans at 33 ksi, a cross-check	2 × 800L150-43, 16 psf ground snow: 6 ft 4 in.	listed deviation CFS-DG8-BASIS

SSMA tables. They treat a back-to-back or box header as twice one C, and limit by F4; those rows run with the F4 option on. Their web crippling column is not compared: an "e" in the table means web stiffeners.

DG-8. The guide prints spans with no calculation. The comparison assumes a 14 ft tributary width and a roof snow load of 0.7 times ground snow, which the guide does not state. Spans for legs up to 8 in. come within 4 %. Spans for the 10 in. legs are about 15 to 18 % longer in the guide, which predates the S240 factor Ω = 2.25 for legs over 8 in. It supports the section modulus; it does not verify the L-header provisions.

Hand calculations, not publications. The box factor α, Eqs. H3-2 and B3.3.2.5 as applied, I1.1, the L-header effective section, the uplift factor, the inverted assembly and the back-to-back section properties.

CHAPTER 173

Cold-formed steel bridging

The intermediate bracing of wall studs: the brace force at a stud, its build-up along a run, and the strap, screws and anchor that carry it. To AISI S240-20 B3.4.1, with AISI S100-16 C2.2.1 and Chapter J.

Inputs of the cold-formed steel bridging

- **Studs:** an SSMA designation, stud spacing, and the spacing of the brace rows a. In S100 C2.2.1 W takes the load within 0.5a each side of the brace.
- **Studs between anchors:** each stud adds its brace force on the way to the anchor. Where you put blocking or struts is your layout.
- **Bridging:** flat straps on both flanges, or U-channel through the web punchouts.
- **Each strap's share of the axial brace force:** S240 gives 2 % of the compression per brace. How two straps share it is your judgment. 0.5 is prefilled. 1.0 puts it all on one.
- **Strap:** width, thickness, Fy and Fu. **Screws:** size, how many at each stud, how many at the anchor.
- **Channel:** its nominal strength with its factors, entered from the product's data.
- **Loads:** by load type on a stud: axial compression per stud, and a pressure across the wall.

Auto-design varies the strap thickness and width, the screws at each stud and at the anchor. Studs, the run between anchors and loads are kept.

Checks performed

Check	Basis
Brace force, bending	S240 B3.4.1.1; S100 C2.2.1, Eq. C2.2.1-3
Brace force, axial	S240 B3.4.1.2, 2 %
Brace force, combined	S240 B3.4.1.3
Strap yielding	S100 D2
Strap rupture at a screw	S100 J6.2
Screws at each stud and at the anchor	S100 J4.3.1, J4.3.2
U-channel axial strength (entered)	Product data

Limitations

- Strength only. The provisions read give a force and no stiffness requirement.
- A uniform load through the web only: Eq. C2.2.1-3. Point loads are not modelled.

- The U-channel's strength is your entry: this type does not compute it, and the clip is taken as 54 mil at 33 ksi for its screws.
- Blocking or a strut at the anchor, its fastening to the stud, and the wall's lateral system are yours.
- With both flanges sheathed, S100 C2.2 asks for no further bracing, and this type is not needed.
- Single anchors at the run's ends: a run anchored at both ends may share the force, which is your entry through the number of studs.

Verification examples

Verification status: 7 values compared in 6 examples (3 published, 3 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published worked example of bridging is in the free library. CFSEI Tech Note W400-23 and the AISI Framing Design Guide are paid and would give them.

- **Published value:** the shear center distance m of three sections against the SSMA Guide (2022). It is the one published number Eq. C2.2.1-3 rests on, and it says nothing about the method.
- **Hand calculations:** the axial and wind forces, their accumulation and the strap yield strength, from S240 B3.4.1 and S100 C2.2.1, D2. No published value checks these.

CHAPTER 174

Cold-formed steel strap braced wall

One story of a wall braced by an X of flat straps, working in tension only: wall shear, chord studs, strap connection, hold-down force and drift. To AISI S240-20 B5.3 and AISI S400-20 E3, with AISI S100-16 for the members.

Inputs of the strap braced wall

- **Wall:** length w between the chords, height h. **Anchor offset:** S240 Eq. B5.3.4.2-1 takes L with the anchor offsets. Enter 0 where the anchor is at the chord.
- **Straps:** on both sides or one, width, thickness, Fy and Fu. One diagonal works in each direction.
- **Seismic design by AISI S400 E3:** on by default. S240 A1.2.2 requires S400 in seismic design categories D, E and F, and wherever R is not taken as 3. Turning it off treats E as a plain load; whether you may is your determination.
- **Connection method:** the three Methods of S400 E3.4.1(a). Method 2 is checked. Methods 1 and 3 are your statement, and the sheet takes them as given.
- **Ry and Rt:** from Table A3.2-1 by Fy, or entered from tests (A3.2.1).
- **Wall finish strength vfinish:** S400 Eq. E3.3.3-1 takes at least 0.2 Vn / w. Enter 0 to use that.
- **Overstrength Ω0 and Cd:** prefilled at 2 and 3.5, from ASCE 7-22 Table 12.2-1 line A.19, flat strap bearing wall systems (R = 4). The S400-20 User Note calls it line A.18, from an earlier edition. Change them if your system differs.
- **SDS:** for Ev in the seismic combinations.
- **Strap connection:** screws (size, number at each end, number across the strap for the net section, gusset), or a strength you entered for a weld or a tested connection.
- **Chord studs:** SSMA designation, how many in each chord, Fy, Kx, bracing, punched or unpunched. **Say which web you have.** Standard SSMA studs are punched, and punched is prefilled. It changes Ae, the moment at shallow depths, and distortional buckling. The sheet states it.
- **Distortional buckling:** computed, or restrained by your determination.
- **Moment from single-sided straps:** on by default for straps on one side.
- **B2:** the story's sidesway multiplier, from your own analysis. 1.0 is prefilled. It acts only when the chord has a moment.
- **Track, top of the hold-down:** for a narrow wall's frame action.
- **Hold-down:** available strength and movement, from the product's report.
- **Drift:** wind factor and limit, Cd, Ie and allowable drift ratio are all your entries.
- **Loads:** by load type: in-plane shear V at strength level, gravity compression P on each chord, and the chord force from the stories above.

Auto-design varies the strap thickness and width, the chord studs of the given depth and their count, and the screws at each strap end. The wall, the loads, the steel and the entered hold-down are kept.

Checks performed

Check	Basis
Wall shear, strap yielding	S400 Eqs. E3.3.1-1, -2, E3.3.2; S240 B5.3.2 with S100 D2
Expected strength factor	S400 Eq. E3.3.3-1, Table A3.2-1
Capacity protected force: the lesser of the expected strength and the load with overstrength	S400 B3, B3.4, E3.4.2
Connection Method 2	S400 Eqs. E3.4.1-1, -2
Strap rupture at the screws	S100 J6.2
Screws at each strap end	S100 J4.3
Chord force	S240 Eq. B5.3.4.2-1, with the stories above
Chord compression	S100 E2, E3, E4
Chord tension	S100 D2
Chord axial and bending	S100 H1.1, H1.2, C1.2.1.1
Eccentric moment of single-sided straps	S400 and S240 Commentaries, half the stud depth
Narrow wall frame moment, from h:L of 2 (above 1.9 by S400)	S240 B5.3.1.1, S400 E3.4.1(b), Commentary Eqs. C-B5.3.1-1 to -6
Hold-down tension	Your entered strength, S240 B5.3.4.2
Drift	S240 B5.3.5, S400 E3.4.4; ASCE 7-22 12.8.6, 12.12.1

Limitations

- **Minor-axis bending of the chord is a lower bound of our own.** S100 gives no effective section for a C bent about its minor axis in a form we could apply directly, so all compressed flats are taken at the extreme fiber stress, with the flange's compressed zone and the lips as unstiffened when the tips are in compression. It errs low. F2.1.2 is applied with CTF = 1.0.
- **Under ASD the expected strength is taken in full.** The Standard shows no ASD reduction in what we read. This errs on the safe side.
- The frame analysis takes the Commentary's closed form. Its deflection is not used for drift, as the Commentary says.
- Drift uses the truss flexibility plus the hold-down movement you entered. Frame action is left out.
- Chord studs add as individual members, with no composite action and no built-up strength.
- Not covered: the track's connections and the collectors (the force is reported), the hold-down and anchor products, gussets as members, reduced-width fuses, strap screws at interior studs, and foundations.
- Not checked, for you to confirm: pretensioning, track and shear device details of S400 E3.4.1(c), (d), and the limits of E3.2.3 with masonry or concrete walls. Results list them.

Verification examples

Verification status: 19 values compared in 11 examples (5 published, 6 hand calculations from the cited equations): 19 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published worked example of a strap braced wall is in the free library. The AISI Framing Design Guide D110 and the shear wall guide D113 are paid and would give them. Everything below is what we could check without them, and most of it is hand calculation. No published value checks the design method as a whole.

- **Published values:** Ry and Rt from S400 Table A3.2-1 at four strengths; the chord tension strength from the gross area SSMA prints for 600S162-54.
- **Hand calculations, from the cited equations:** the wall shear, the expected strength factor, the capacity protected connection force both ways, the chord forces, the frame action equations and the drift.
- **Not verified by any outside value:** the minor-axis strength and the combined chord check of a narrow wall.

CHAPTER 175

Cold-formed steel purlin

A line of C or Z purlins over one to six spans, with bolted laps at the supports. Girts follow the same equations. To AISI S100-16 with S2-20: Chapters F, G, H and Section I6, laid out by regions as the MBMA Roof Framing Design Guide (2nd Ed., 2024) examples are.

Inputs of the purlin

- **Section:** from a list named as the AISI Design Manual names its sections, or custom dimensions, which is what a manufacturer's purlin usually needs. End bays and interior bays each take a section, of one depth and profile: the webs lap. Lip length is measured along the lip from the outer flange line. A Z's lip angle is 50° on the Manual's sections.
- **Web: punched or unpunched.** Purlins are usually unpunched, and that is prefilled. The sheet states which. S100's web hole provisions in the Effective Width Method are for C-sections, so a punched C is checked (F3.1.2, F4.2, G3, G6) and a punched Z is **not feasible** here. Design that one by the Direct Strength Method or tests.
- **Spans and laps:** per span, its length and how far a lap runs into it from each support. Ends are pinned. With no lap, the two spans must be one section, continuous over the support.
- **Laps bolted for continuity:** you confirm it. The analysis takes it.
- **Roof system:** through-fastened or standing seam.
- **kφ (through-fastened):** the panels' rotational restraint for distortional buckling, from tests or the manufacturer. The prefilled 0.300 kip-in./rad/in. is the MBMA example's, not a value for your panel.
- **R for through-fastened uplift:** Table I6.2.1-1, with conditions (i) to (n) confirmed by you, or entered from full-scale tests by K2.1.
- **R (standing seam):** gravity and uplift, end and interior bays, from AISI S908 base tests of your panel, clip and purlin. The prefilled values are the MBMA example's.
- **Effective length factor, Z-section Fcre, Cb, β:** the code's choices. Eq. F2.1.3-2 and Cb of 1.67 are prefilled, as the Guide uses them, and both are on the safe side. β is by Eq. 2.3.3.3-3, or 1.0.
- **Supports:** bearing length and whether the flanges are fastened, or web plates or clips, which take away G5 and H3. Past 1.3 for thicker to thinner, H3 takes the thicker section at 1.3 t or twice the thinner, as the Guide allows.
- **Loads:** on this line, parallel to the web, with self-weight in the dead load. **Wind acts as entered:** a negative row is uplift, a positive row inward pressure, each a case of its own. ASCE 7 gives them as separate pressures, so neither sign is assumed from the other.
- **Patterns:** L by ASCE 7-22 4.3.3 and snow by 7.5.1 are on. Lr by 4.3.3.1 is off: that section asks for it where the roof live load is reduced below 20 psf, so turn it on if yours is.
- **Deflection limits:** your entries. IBC Table 1604.3's values for roof members with no ceiling are prefilled.
- **Anchorage:** I6.4.1. Purlin count and spacing, top flange direction, roof slope, the purlins with a device, its stiffness and strength, the panel area and diaphragm stiffness.

Auto-design varies the listed sections of the given profile and depth, end bays and interior bays. The spans, laps, loads, steel, roof and its R values and the anchorage are kept. A custom section is kept as entered.

Checks performed

Region or check	Basis
Positive moment, through-fastened: local	S100 F3.1, Fn = Fy (panels brace the top flange)
Positive moment, through-fastened: distortional, with kφ	S100 F4.1, Appendix 2 2.3.3.3, Table 2.3.1.3-1
Positive moment, standing seam	S100 Appendix A I6.2.2, R from S908
Negative moment, lap end to inflection point: lateral-torsional and local	S100 F2.1.3 (Z) or F2.1.1 (C), F3.1 at Fn
Negative moment, lap end to inflection point: distortional, kφ = 0, β	S100 F4.1, Eq. 2.3.3.3-3
Within a lap	Sum of the two sections' F3 strengths, the Guide's assumption
Uplift, between inflection points, through-fastened	S100 I6.2.1 with Table I6.2.1-1, simple span values and r where they apply
Conditions (a) to (h), (o) of I6.2.1	Checked: NOT FEASIBLE outside them
Conditions (i) to (n) of I6.2.1	Your confirmation, and the sheet says so: NOT FEASIBLE if you do not confirm
Uplift, standing seam	S100 Appendix A I6.2.2
Shear; shear in a lap, summed	S100 G2.1 (G3 for a punched C)
Bending and shear at every station	S100 Eq. H2-1
Web crippling; the sheets of a lap added	S100 G5, Tables G5-2, G5-3 (G6 for a punched C)
Bending and web crippling at interior supports	S100 Eq. H3-3 nested Z, H3-2 back-to-back C, H3-1 otherwise
Deflection, with Ieff at the service moment	S100 L1, your limits
Anchorage force, stiffness, top flange movement	S100 Eqs. I6.4.1-1 to -8; Guide Eqs. 5.3.1-1, -2, 4.1-14 for the movement

A negative moment region next to an interior support takes Chapter F, since S100 says I6.2.1 does not apply there. A negative moment region not next to one takes the R method. That is the geometry of the diagram, whichever combination made it.

Limitations

- **Lapped regions rest on assumptions.** S100 gives no method of analysis for laps. Ix over a lap is the sum of the two sections, the strength in a lap is the sum of their strengths, and a lap is taken as restrained against lateral-torsional and distortional buckling. These are the MBMA Guide's assumptions and they are yours to accept. The results state them.
- **The section list is not from a source on hand: confirm your section against the maker's data.** The list of depths and flange widths follows the AISI Design Manual's names from memory, and its tables are paid. The check results and the sheet say so whenever a listed section is used. The lips of the C-sections at x085 and x105 have nothing on hand to check them at all. Custom dimensions take your maker's numbers and carry no such line.
- **The section list's lip lengths are not all printed in what we have.** The AISI Design Manual's tables are paid. Three are printed in free reports, three are read back from printed gross properties and land on all of them, and

C-section lips at x085 and x105 are carried on from the others with nothing to check them. Metal building purlins differ from the Manual's in any case: for a real purlin, enter its dimensions.

- **Not built:** S100 I6.1, the analytical route for standing seam. Discrete bracing of a standing seam roof (the Appendix A alternative). Overhangs and cantilevers. Point loads and loads on part of a span. Axial load in strut purlins (I6.2.3, I6.2.4). Mid-point and third-point anchorage over several spans: those lines average three bays and have rows of their own, and the check says NOT FEASIBLE rather than guess. I6.4.2 torsional bracing.
- **H3 past the limits of Eq. H3-3:** S100 gives no equation for nested Zs out there. This calculation uses the stricter Eq. H3-1 on the summed strengths, and says so. That choice is ours.
- **R by Table I6.2.1-1 with bays varying more than 20 percent:** simple span values on all bays, with r on them. S100 names the adjacent bays. Taking all of them is on the safe side.
- Bolts in the laps and at the supports, anchorage devices, panels and their fasteners, and I6.3 are not designed.

Verification examples

Verification status: 183 values compared in 12 examples (12 published, 0 hand calculations from the cited equations): 173 within tolerance, 10 recorded deviations. Code basis: the provisions it uses were read in the target edition.

From the free MBMA Roof Framing Design Guide, 2nd Ed. (2024), with printed page numbers. The Guide takes section properties from the paid Design Manual; here they are computed and compared with what it prints.

- **Example 3.3.1.1, pp. 64 to 79:** four span Z, through-fastened, ASD. Section properties, every moment, shear and reaction of the non-prismatic analysis, F3, F4 with kφ, F2.1.3, the laps, shear, H2, G5, H3, I6.2.1.
- **Example 3.3.2.1, pp. 93 to 104:** the same line, standing seam, ASD.
- **Example 3.3.2.2, pp. 106 to 118:** four span C, standing seam, LRFD.
- **Examples 5.4.1, 5.4.2, 5.4.3, pp. 193 to 204:** anchorage at third points and at the supports. **Example 5.4.4, pp. 205 to 207:** opposed C-sections, on the library, in the behaviour tests, as its tributary widths are unequal.
- **AISI RP20-5 p. 5, RP21-01 pp. 5, 19:** gross properties and lips.
- **Hand calculations:** the non-prismatic analysis against the three-moment equation, the Z column of Table 2.3.1.3-1 against its C column at 90°, the R table, r and the 20 percent rule.

Where they part from the Guide, each is recorded plainly on the Verification page:

- **The half-wavelength of a Z-section** comes out about 2 percent longer than the Manual's table. The buckling stress lands on the printed value.
- **The Guide's Example 3.3.2.2 torsional stress** leaves the square off the polar radius, 452 ksi for about 116 ksi. Here the elastic stress is 262 ksi for its 542 ksi. No strength changes in that example.
- **The Guide's LRFD uplift** takes 1.6 wind though it names ASCE 7-16. Here it is ASCE 7-22, 1.0 wind.
- **The Guide's Example 3.3.1.1 H3** prints 1.00 at or below 0.97 and calls it OK. It is not. Here that check, with the Guide's twice-the-thinner choice, fails at 1.03. With the thicker section at 1.3 t, which the Guide also allows and which is prefilled, it passes.
- **Not verified by any published value:** S100 F2.1.3 Eq. -1, the 20 percent rule and r, a punched C, deflection (no printed value), anchorage at mid-span, top flange movement at third points, and the partial snow cases.

CHAPTER 176

Cold-formed steel curtain wall

A non-load-bearing wind stud, to AISI S240-20 and the AISI S100-16 North American Specification with Supplement S2-20. It covers the stud, its track, and the deflection track at its top, and reports the limiting height the way the SSMA tables print it.

Inputs of the cold-formed steel curtain wall

- **Stud:** an SSMA designation or custom dimensions; one stud or two, and where it sits in the wall. That choice sets C in S240 Table B3.2.5.1-1. Web holes are an option.
- **Supports:** single span, two spans, a parapet (a span and an overhang) or a knee wall (a cantilever).
- **Wind:** components and cladding at the strength level, from your wind calculation. It acts both ways. ASD takes 0.6 of it. The wind pressure is not derived here.
- **Superimposed vertical load:** S240 defines a curtain wall as carrying at most 100 lb/ft, or at most 200 lb. Over that, the calculation is not feasible: use the cold-formed stud type.
- **Flange bracing:** discrete bracing on both flanges; structural sheathing on one and discrete bracing on the other (S240 B1.2.2.3: at most 8 ft, studs at most 24 in.); or structural sheathing on both.
- **Distortional buckling:** computed by default. You may mark it restrained if that is your judgment. kφ comes from S240 Appendix 1.
- **Deflection:** a wind factor, 0.42 by S240 B1.1.2(c), and a limit, span / n. Your finish decides the limit. An overhang is on twice its length.
- **Track and top details:** track, deflection track or clip at the top; track or clip at the base. Whether both flanges are screwed, the screw, the track thickness, steel and flange, the slip gap.

Auto-design varies the section (lightest first) and the yield stress when unlocked. Wind, height, spacing and the track are kept.

Checks performed

Check	Basis
Vertical load within the definition	S240 Definitions
Flexure	S100 F2, F3; S240 B1.2.2.3
Distortional buckling	S100 F4, Appendix 2 2.3.3.3
Shear	S100 G2, G3
Bending and shear	S100 H2
Stud to track	S240 B3.2.5.1(a), (d), (e); Eq. B3.2.5.1-1; S100 G5
Thin track	S240 B3.2.5.1(b), Eq. B3.2.5.1-2
Stud at a deflection track	S240 B3.2.5.2.1; S100 G5

Check	Basis
Deflection track	S240 B3.2.5.2.2, Eqs. B3.2.5.2-1, -2
Interior support	S100 G5, H3
Deflection	S240 B1.1.2(c), B3.5; IBC 2021 Table 1604.3

The limiting height is the least of flexure, shear and deflection for one or two equal spans, and on two spans bending and shear together at the interior support, with a punchout next to it if you say so. It is reported beside the checks and is not itself a check: web crippling is not in it, and the checks handle that.

Limitations

- No axial load is combined with bending: the stud carries little, inside its definition. Use the cold-formed stud type otherwise.
- Bridging forces and their anchorage are in the cold-formed steel bridging type. Slide clips and their screws are in the connection type; their reactions are in the results.
- S240 B3.2.5.1(c), the jamb with two studs and a track of the same thickness, is not built.
- Web crippling on a two-stud position outside the equation's range uses each stud as a single web.
- The wind pressure comes from you.

Verification examples

Verification status: 24 values compared in 23 examples (21 published, 2 hand calculations from the cited equations): 24 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

- **Published table values:** SSMA Product Technical Guide (2022) pp. 26 to 28, 20 limiting heights and spans in single and double span tables. The table's pressures are at 0.6 W and reduced by 0.70 for deflection, which is the 42 % of S240; its 5 psf column is not reduced. Cells marked as limited by web crippling are not used. The two span values are compared with a punched web, as the guide prints them: for depths up to 4 in. the punchout reaches 0.38 of the flat web, and its moment is at the punchout.
- **Hand calculations:** S240 Eqs. B3.2.5.1-1 and B3.2.5.2-1 from their stated form. No published worked example of a track or deflection track is on disk; the CFSEI tech notes W100 to W103 and the AISI Framing Design Guide are paid.

CHAPTER 177

Cold-formed steel rafter

A sloped lipped C-section rafter with the thrust at its heel and the ceiling joist or tie that resists it, to AISI S240-20 Section B4.2 and the AISI S100-16 North American Specification with Supplement S2-20, with ASCE 7-22 load combinations.

Inputs of the cold-formed steel rafter

- **Section:** an SSMA designation or custom dimensions. Fy and Fu are yours. Nothing is added for cold work.
- **Roof geometry:** horizontal span from plate to ridge, slope as rise in 12, a horizontal overhang, and the rafter spacing.
- **Ridge:** a ridge board with a tie at the heel makes a three-hinged frame, and the tie takes the thrust. A structural ridge beam or bearing wall holds the ridge up, with no thrust.
- **Area loads:** by load type, each on the slope, on the horizontal projection or normal to the roof. Normal is positive toward the roof, negative for suction. Both rafters of a pair take the same load, so enter the heavier side.
- **Point loads:** vertical, placed horizontally from the plate.
- **Bracing:** each flange braced by sheathing or at a spacing you enter. S240 B4.5(b) states what sheathing counts. The rotational restraint kφ is yours, from S240 Appendix 1; 0 means none.
- **Compression basis:** continuously braced (S240 B4.2.2(b): Fn = Fy with E3, and distortional buckling) or discretely braced (Chapter E at your brace spacing). Tick the box when your braces also restrict flange rotation, which makes Lm the spacing.
- **Kx:** on the sloped length from plate to ridge. It enters E2 and the amplifier B1.
- **Bearing:** flange bearing with its length N, or a clip, which turns web crippling off.
- **Tie:** section, steel, the net area ratio An/Ag (your judgment; 0.85 is prefilled and is not a code value), and the screws at the heel.
- **Deflection limits:** span / n, on the length along the slope and on twice an overhang. 0 turns a check off.

Auto-design varies the section (lightest first) and the screws at the heel, and the yield stress when unlocked. Spacing, tie and loads are kept.

Checks performed

Check	Basis
Flexure, positive and negative moment	S240 B4.2.3; S100 F2, F3
Distortional buckling in flexure	S240 B4.2.3; S100 F4, Appendix 2 2.3.3.3
Compression	S240 B4.2.2; S100 E2, E3
Distortional buckling in compression	S240 B4.2.2; S100 E4, Appendix 2 2.3.1.3
Tension	S240 B4.2.1; S100 D2, D3

Check	Basis
Shear	S240 B4.2.4; S100 G2
Web crippling at the plate	S240 B4.2.5, B1.6; S100 G5
Compression and bending, with B1	S240 B4.2.6; S100 H1.2, C1.2.1.1
Tension and bending	S240 B4.2.6; S100 H1.1
Bending and shear; bending and web crippling	S240 B4.2.7, B4.2.8; S100 H2, H3
Deflection	IBC 2021 Table 1604.3, limits entered
Tie in tension; heel screws	S100 D2, D3, J4.3

The perpendicular load components bend the rafter as a simple span along the slope. The parallel components and the end reactions give the axial force by statics at every station. With a ridge board, moments about the heel give the thrust H = R / sin(slope), where R is the perpendicular reaction at the ridge.

Limitations

- The tie is at the heel. A raised tie is not built.
- Both rafters take the same load. Unbalanced snow and wind on one side are your entry on the heavier side.
- Web holes, bearing stiffeners and clips are not designed. The tie's bending under a ceiling load is a job for the cold-formed joist type.
- Under net uplift the tie goes into compression and the ridge pulls apart: the force is reported, and its check is yours.
- The rafter's net section is taken as the gross section in D3.
- S240 B4.2.2(b) prints F4 (a flexure section) for the compression distortional check. This calculation reads it as E4, the compression section, and says so in its note.
- The tie net area ratio is your judgment. The module does not derive it from the screw pattern.

Verification examples

Verification status: 17 values compared in 6 examples (3 published, 3 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published worked example of a cold-formed rafter is on disk. The AISI Framing Design Guide D110 would give one; it is on the list to buy.

- **Published section strengths:** the SSMA Guide (2022) p. 11, Mal, Mad and Vag of three sections, read back through the module. They confirm the section strengths, not the rafter's statics or combined checks.
- **Hand calculations:** three cases of statics, closed form: thrust, moment and axial force of a ridge board frame and a ridge beam rafter under plan and normal loads. No published value checks these.

CHAPTER 178

Cold-formed steel fastener connection

A group of screws, bolts or welds joining two cold-formed sheets under shear and tension, to Chapter J of the AISI S100-16 North American Specification with Supplement S2-20, with ASCE 7-22 load combinations. You enter the forces on the group; they are shared equally among the fasteners.

Inputs of the cold-formed steel connection

- **Fastener:** screws (No. 6 to 1/4 in.), bolts (1/4 to 3/4 in., ASTM A307, F3125 A325 and A490, A354 BD, A449), a fillet weld or a flare groove weld.
- **Screw strengths Pss and Pts:** the manufacturer's or tested values. At 0 the module uses the CFSEI F701-12 values printed in the SSMA Guide (2022). The specification asks you for these, so please use your product's.
- **Penetration tc:** the lesser of the depth of penetration and t2 (J4). 0 uses t2.
- **Washer and nonuniform pull-over:** a washer sets the pull-over diameter (J4.4.2). Tick nonuniform for an eccentric load that takes 50 % of the pull-over strength (J4.5.1).
- **Bolts:** threads in or out of the shear plane, single or double shear, hole type and washers set the factor mf (Table J3.3.1-2). Tick hole deformation when it is a design consideration (J3.3.2).
- **Welds:** total length, the load along or across the weld. The weld leg, the electrode and the flare throat matter above 0.10 in. Enter the throat your procedure qualifies.
- **Pattern:** the number of fasteners, how many lie across the load, the spacings, and the end and edge distances.
- **Sheets:** thickness by mils (SSMA design thickness) or custom, Fy and Fu of each. The sheet at the head is t1, the sheet at the tip is t2.
- **Rupture:** on by default (J6). A sheet width turns on tension rupture of a flat sheet or strap. Other shapes need their own net section.
- **Forces:** nominal shear and tension on the whole group by load type. Eccentricity is yours to add.

Auto-design varies the number of fasteners in whole rows and their size, or the weld length. Sheets, forces and the fastener kind are kept.

Checks performed

Check	Basis
Spacing, end and edge distance	S100 J3.1, J3.2, J4.1, J4.2
Sheet shear per screw	J4.3.1, Eqs. J4.3.1-1 to -5
Screw shear, screw tension	J4.3.2, J4.4.3
Pull-out, pull-over, minimum head or washer	J4.4, J4.4.1, J4.4.2
Combined shear with pull-over, pull-out, tension in the screw	J4.5.1, J4.5.2 (inside their stated limits), J4.5.3
Bolt bearing, each sheet	J3.3.1, J3.3.2, Tables J3.3.1-1, J3.3.1-2

Check	Basis
Bolt shear, tension, combined	Appendix A J3.4, Table J3.4-1
Fillet weld	J2.5, Eqs. J2.5-1 to -7
Flare groove weld	J2.6, Eqs. J2.6-1 to -4
Shear, tension and block shear rupture of each sheet	J6.1, J6.2, J6.3, Table J6-1

Limitations

- Forces are shared equally. Eccentric groups and prying are not analysed.
- Screws, bolts and anchors into wood, concrete or masonry are not covered: S100 J7 sends them to product data, and S240 B1.5.6 to the other material's code.
- Power-actuated fasteners, arc spot, arc seam and top arc seam welds are not built.
- Tension rupture covers a flat sheet only. Shear lag in an angle or channel is not computed.
- The two combined equations for pull-over and pull-out have narrow limits. Outside them the module says so and does not apply them.
- A thinnest part over 3/16 in. is not feasible here: Chapter J sends it to AISC 360.
- Pull-over at a bolt head has no equation in the specification. It is a note.

Verification examples

Verification status: 68 values compared in 30 examples (28 published, 2 hand calculations from the cited equations): 68 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No free source on disk prints a worked connection example. The AISI Cold-Formed Steel Design Manual would have them; it is on the list of documents to buy.

- **Published table values:** SSMA Product Technical Guide (2022) p. 70, 18 screw rows of shear, pull-out and pull-over and 10 weld rows, are read back through the module. All are within 2 % or 5 lb, the rounding of a table. The table prints the 2016 equations under their older section numbers (E4 for J4), and uses the longer-weld equations J2.5-3 and -4 for its longitudinal fillet column.
- **Hand calculations, from the cited equations:** bolt bearing, bolt shear, shear rupture, the screw case with t2 well over t1, the interpolation, the bearing factor table, F'nt and the fillet ranges. No published value checks these.

CHAPTER 179

Cold-formed steel truss

Cold-formed steel truss: pick a layout, span and pitch, and get C-section chords and webs sized and checked to AISI S100-16 and AISI S240-20 Chapter E, for roof and floor trusses, with ASCE 7-22 load combinations.

Inputs of the cold-formed steel truss

- **Roof or floor, truss type:** roof trusses offer 18 layouts, floor trusses the parallel-chord ones.
- **Geometry:** span between the outer bearing lines, pitch (rise in 12), heel height, depth on parallel chords, flat length on a hip, rise on a bowstring, room width and height on an attic, overhangs, panels (Auto: about 6 ft on a pitched truss, about 1.5 depths on parallel chords; a count the layout cannot have snaps to the nearest one and the results say so).
- **Supports:** pin, roller or none at each end, intermediate supports (moved to the nearest chord joint, and the results say where), bearing on the bottom or top chord of a parallel-chord truss.
- **Loads:** area loads in psf times the truss spacing: top chord dead, roof live, snow, unbalanced snow, floor live, bottom chord dead and live. Live load on half the span as well (ASCE 7-22 4.3.3, on by default, because web forces reverse under it). Wind rows (pressures normal to the top chord from your wind analysis, each checked from both directions), total horizontal seismic, SDS for the vertical effect, and point loads anywhere on a chord, also between panel points.
- **Members:** Size the members automatically is on by default: at every calculation the lightest section of each group that passes is chosen. Turn it off to enter the sections and have them checked as they are.
- **Bracing:** what holds each chord out of plane, and the webs: as needed (a compression web that fails unbraced gets a lateral brace at mid-length before anything is upsized, and the results, schedule and sheet name it), none, or every web. These are your judgment as engineer of record, and the braces themselves are not designed.
- **Members with their own section:** a mark from the schedule and a section, for one heavy web or chord panel. The rest keep the section of their group.
- **Serviceability:** live and total deflection limits as span / n, and the horizontal movement limit at a roller.

Auto-design varies the top chord, bottom chord and web sections, lightest first, within the given families, and fixes the result (it turns automatic sizing off). It never changes the geometry, loads, bracing or code options.

Analysis. A plane frame, first-order, with chords continuous over the panel points and pinned at heels and pitch breaks, and webs pinned at both ends, or every panel point pinned if you choose. Loads between panel points are carried in chord bending, so a chord has axial force and bending together. Every load case is solved once and combined with ASCE 7-22 2.3.1 or 2.4.1 (and 2.3.6 or 2.4.5 with seismic). Deflections always use the ASD combinations.

- **Sections:** SSMA lipped C designations, depth in the plane of the truss, Fy and Fu. Net area An/Ag for screw holes is your input because connections are not designed.
- **Effective lengths by AISI S240-20 E4.4.2.1:** on by default (Kx, Ky, Kt and Cm = 0.85 as the standard prescribes). Off: the K values you enter and Cm = 1.0.
- **C webs attached back to back with the chord's web:** adds the eccentricity interaction of E4.4.4.1 to compression webs.

Checks performed

Check	Basis
Analysis model	AISI S240-20 E4.3
Tension	AISI S100-16 D2, D3; S240-20 E4.4.3, E4.4.5
Compression	S100-16 E2, E3; S240-20 E4.4.2, E4.4.4
Flexure	S100-16 F2, F3, F4; S240-20 E4.4.2.2
Shear; bending + shear	S100-16 G2, H2; S240-20 E4.4.6.2(b)
Axial + bending	S100-16 H1.1, H1.2
Web eccentricity	S240-20 E4.4.4.1
Deflection	Your limits; S240-20 E4.7 full section

The report prints the steps of the governing member of each group and lists every member in the schedule. Turn on Report every member step by step, under Serviceability, to print every member.

Limitations of the cold-formed steel truss

- **Connections are not designed.** Screws, gusset plates (S240-20 E4.5), fastening and copes (E4.6), heel joints and anchorage. Proprietary truss sections need the manufacturer's values and tests. The forces in every member at every joint are listed for that design.
- One section each for the top chord, bottom chord and webs, apart from the members you give their own.
- One lateral brace per web at most, at mid-length.
- Work lines meet at the joints: joint eccentricity and secondary moments are not modeled. First-order analysis, with moments amplified member by member where the standard does so.
- Gable-end, gambrel, piggyback and unequal-pitch layouts are not generated. Use the 2-D truss analysis for any other geometry.
- Ponding, fire, floor vibration, system bracing design, wind pressure calculation and trusses carrying other trusses (use point loads) are outside this calculation.
- Not applied: S240-20 E4.4.6.1 joint eccentricity (the model is a single node without it). E4.4.6.3 chord shear between fastener groups: the shear check keeps kv = 5.34, which the stiffened-panel equations only raise. Connections, screws, gussets (E4.5), copes (E4.6) and proprietary sections stay out, as the owner decided for the truss designer.
- Added in wave 7: S240-20 E4.4.2.3 at compression chord panel points (the load on the flange is entered, and webs bearing on the flange add the step in the chord's shear; webs lapped back to back pass their force through fasteners and are not counted, which is our reading and not stated in the standard's text), S100-16 E4 distortional buckling in compression with no rotational restraint counted (it also limits the available axial strength in H1.2), and web crippling at the reactions, on one web without a stiffener. These are hand-checked only, and saved calculations can now show one more failing check.
- In Eq. E4.4.4-1, May takes local buckling only; global buckling about the minor axis is not evaluated.

Verification examples

Verification status: 18 values compared in 8 examples (3 published, 5 hand calculations from the cited equations): 18 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

These results carry the label "Preview: not yet verified, do not rely on these results" on the screen, in the report and on the sheet. The frame solver is checked against closed-form beams and the pin-jointed solver. Published values are SSMA's Mal and Vag only; compression, the combined checks and S240 are hand calculations. No published cold-formed truss or effective-width compression example is on file.

CHAPTER 180

Cold-formed steel clip angle

A bent clip angle that joins a stud or joist to the structure: leg A is screwed to the member, leg B is screwed, welded or anchored to the support. To AISI S100-16 (with S2-20) Chapter J, D2 and A1.2, with the ASCE 7-22 load combinations.

Inputs of the clip angle

- **Clip:** thickness (mils or custom), leg A and leg B, the length along the bend, Fy and Fu.
- **Leg B bending section and factors:** S100 has no provision for a flat leg bent out of its plane. This type computes it as a rational engineering analysis by S100 A1.2(c): Mn = Fy S with S = L t²/6 (prefilled) or the plastic Z = L t²/4, with the A1.2(c) factors for connections (φ 0.55, Ω 3.00, prefilled) or for members (φ 0.80, Ω 2.00). Both are your choice.
- **Leg A to the member:** the member's thickness and Fu, which sheet is under the screw head, the screw size and count (one line along the clip), the line's distance from the back of leg B, and the end distance.
- **Leg B to the structure:** screws into steel (support thickness and Fu), a fillet weld along the clip (shear only), or anchors and power-actuated fasteners whose available shear and tension you enter from the product's evaluation report at the method chosen, with an interaction exponent (1.0 linear, prefilled).
- **Fastener line on leg B:** its distance from the back of leg A sets the lever of leg B bending.
- **Prying factor:** your allowance on fastener tension for leg B prying on the support. 1.0 is no prying.
- **Forces:** by load type. V acts along the clip (shear in every fastener). N acts in the plane of leg A, perpendicular to the support: positive pulls leg B off the support, negative presses it on (no tension, no leg bending).

Auto-design varies the clip thickness (33 to 118 mils) and the number of fasteners in each leg. The legs, length, attachment kind, member, support and forces are kept.

Checks performed

Check	Basis
Screw spacing, end and edge distances	S100 J4.1, J4.2
Leg A screws: sheet shear (tilting, bearing) under √(V² + N²)	S100 J4.3.1
Leg A screws: screw shear	S100 J4.3.2 (CFSEI F701-12 values as printed by SSMA)
Leg A shear rupture toward its free edge (N) and along the clip (V)	S100 J6.1
Leg A tension yielding	S100 D2
Leg B bending at the bend	S100 A1.2(c), rational analysis
Leg B screws: sheet shear, screw shear, pull-out, pull-over, screw tension, combined	S100 J4.3, J4.4, J4.5.3

Check	Basis
Fillet weld along the clip, sheet tearing and throat	S100 J2.5
Anchors: interaction of the entered strengths	Product evaluation report

Limitations

- Forces are shared equally among the fasteners of a leg. A moment on the fastener group, from an eccentric force, is yours to add to the forces.
- Leg B bending uses the lever from the fastener line to the inside face of leg A. Prying on the support is your factor.
- A weld takes shear only: a combination that pulls leg B off the support with a weld is not feasible here.
- Anchors into concrete and power-actuated fasteners are not designed: their strengths are your entries.
- The member and the supporting structure beyond their sheets are not designed.

Verification examples

Verification status: 11 values compared in 6 examples (0 published, 6 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a cold-formed clip angle is in the free library. The values are hand calculations of the cited equations from the inputs (sheet shear, pull-out, pull-over, tension yielding, shear rupture, leg bending), shown in the test file so each can be checked on paper. A CFSEI Technical Note or AISI D110 example would replace them.

CHAPTER 181

Cold-formed steel parapet

A C-stud that rises from its base at the roof past a kicker brace to a free top: the parapet wind on it, the stud, the kicker down to the roof framing and the screws at the kicker's ends. To AISI S100-16 (with S2-20), with the parapet wind of ASCE 7-22 30.6.

Inputs of the parapet stud

- **Stud:** an SSMA designation or custom dimensions, Fy and Fu, the spacing and the height above its base support.
- **Flange bracing:** sheathing on both flanges (bending at Fn = Fy) or discrete braces at a spacing (lateral-torsional buckling with Cb = 1.0). Which applies is your call.
- **Weak-axis and torsional length for compression:** the bridging or sheathing fastener spacing you rely on (S240 B1.2.2).
- **Kicker:** its height on the stud, its run to the roof framing (the angle follows), an SSMA designation, Fy and the effective length factor K (1.0 for pinned ends, prefilled), the screws and their count at each end, and the roof framing's thickness and Fu.
- **Wind from ASCE 7-22 30.6:** V, exposure, Kzt, Kd (0.85 prefilled), the ground elevation for Ke (0 takes Ke = 1.0), the height of the parapet's top, and the location: interior (wall zone 4 with roof edge zone 2) or corner (wall zone 5 with roof corner zone 3). The roof must slope 7° or less (Fig. 30.3-2A). The Fig. 30.3-2A note 5 option applies the parapet note to the roof face; it is off by default, which takes the roof value as printed.
- **Or the net pressures entered:** toward the roof and away from it, at the strength level.
- **Deflection:** the wind factor (0.42, S240 B1.1.2 and the IBC Table 1604.3 footnote) and the limits: the tip on twice the cantilever, the back span on its span.

Auto-design varies the stud and the kicker (SSMA designations, lightest first) and the screws at each end of the kicker. Heights, the kicker's position, the wind and the limits are kept; Fy when unlocked.

Checks performed

Check	Basis
Parapet wind, load cases A and B	ASCE 7-22 30.6, Eq. 30.6-1, Figs. 30.3-1, 30.3-2A, 26.10
Stud flexure	S100 F2, F3
Stud shear	S100 G2
Stud bending and shear	S100 H2
Stud compression below the kicker (case B) and compression with bending	S100 E2, E3, H1.2 with B1 (C1.2.1.1)
Stud tension with bending (case A)	S100 D2, H1.1

Check	Basis
Kicker compression and tension	S100 E2, E3, D2
Kicker screws at each end: sheet shear, screw shear	S100 J4.3.1, J4.3.2
Tip and back span deflection	IBC Table 1604.3 (limits entered), S100 L1

The stud carries both faces of the parapet, so the internal pressure acts on both and cancels: the stud's net pressure is qp Kd times the difference of the two faces' (GCp). Wind is analysed alone (1.0W for LRFD, 0.6W for ASD); the stud's weight is vertical and not counted in bending.

Limitations

- Buildings with h up to 60 ft and roofs up to 7°, for the computed pressures. Otherwise enter the net pressures.
- One kicker per stud, pinned at both ends, landing at the level of the stud's base.
- The base track and its anchorage (the base reaction is reported), the roof framing at the kicker's lower end, bridging, the cap and the finish are not designed.
- Web crippling of the stud at the kicker and at the base is not checked: the kicker is screwed to the web, and the base detail is yours.

Verification examples

Verification status: 6 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 6 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a parapet stud with a kicker is in the free library. The values are hand calculations: one ASCE 7-22 30.6 pressure case worked from the equations and figures, and the statics of the stud and kicker under entered pressures. A CFSEI Technical Note or AISI D110 example would replace them.

CHAPTER 182

Cold-formed steel diaphragm

A roof or floor diaphragm of wood structural panels screwed to cold-formed steel joists, spanning between two shear wall lines under a uniform in-plane load. To AISI S240-20 B5.4.2 for wind (and seismic outside S400), or AISI S400-20 F2 where the seismic force-resisting system is designed by S400, with AISI S100-16 for the chords.

Inputs of the diaphragm

- **Standard:** S240 B5.4.2 (φ 0.65, Ω 2.00) or S400 F2 (φ 0.60, Ω 2.50). Both print the same table of nominal unit shears.
- **Sheathing:** Structural I or other graded panels, 3/8, 7/16 or 15/32 in., blocked with the screw spacing at the boundary and continuous edges (the other edges follow the table), or unblocked in the configuration that applies. The panel material (OSB or plywood) enters only the deflection.
- **Load duration:** the table values are for short-term load (wind, seismic); S240 B5.4.2.2 multiplies them by 0.75 for normal and 0.67 for permanent load.
- **Diaphragm:** length L perpendicular to the load and depth b parallel to it; the in-plane line loads by type (W, E), nominal. For seismic, enter the diaphragm design force (ASCE 7-22 12.10).
- **Framing and requirements:** the joist designation and spacing, the sheathing screw size and field spacing are checked against the listed requirements. The others (steel grades, track, framing screws, panel standards, panel widths, blocking, strap blocking, and in S400 the edge distance) are your confirmation. In S400, a diaphragm that supports masonry or concrete walls must be blocked (F2.2.2), and R over 3 brings F2.5.
- **Chords and collector:** the chord member (an SSMA designation), how many per side, Fy and the length between braces for compression; the shear wall length along the support line (shorter than b makes a collector) and, for seismic, the overstrength factor Ω0 for the collector (S400 F2.3).
- **Deflection:** under the W or E loads at their nominal value, with the sheathing shear modulus G (about 177,300 psi for 7/16 in. 24/16 OSB from the S240 Commentary, prefilled), the chord splice term Σ Δci Xi, and a limit (0 reports the deflection only).

Auto-design varies the panel thickness, the edge screw spacing (blocked) and the chord members. The standard, panel grade, blocking, the diaphragm's size, the loads and the framing are kept.

Checks performed

Check	Basis
Aspect ratio, joist thickness and shape, screw size, field screw spacing, joist spacing	S240 B5.4.2.1 / S400 F2.4.1.1
Other listed requirements, confirmed	S240 B5.4.2.1 / S400 F2.4.1.1
Blocking with masonry or concrete walls	S400 F2.2.2
Unit shear v = w L / (2 b) against the table value	S240 B5.4.2.2, B5.4.2.3 / S400 F2.4
Chord tension and compression, M / b with M = w L² / 8	S100 D2, E2, E3

Check	Basis
Collector tension and compression	S100 D2, E2, E3; S400 F2.3
Deflection, blocked and unblocked	S240 Eqs. B5.4.2.4-1, B5.4.2.4-5

Limitations

- A simple-span diaphragm under a uniform load, between two support lines. Openings, cantilevered or continuous diaphragms and point loads are not modelled.
- The collector is checked with the chord member's strength.
- The shear walls, the transfer of the shear into them, chord splice connections and the joists' gravity design are not part of this calculation.
- Steel deck diaphragms (AISI S310) are a separate design.

Verification examples

Verification status: 23 values compared in 9 examples (6 published, 3 hand calculations from the cited equations): 23 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

The nominal unit shears are **published values**: cells of S240-20 Table B5.4.2.2-1 and S400-20 Table F2.4-1, read back through the available strength. The unit shear, chord force and deflection are **hand calculations** of the cited equations. No published worked example of a cold-formed steel diaphragm is in the free library; AISI D110 would give one.

CHAPTER 183

Cold-formed steel ceiling framing

A ceiling joist (an SSMA C-section) or a hat-channel furring member, over one to four equal spans between hangers or supports, under the ceiling's dead load, mechanical loads, any live load and the vertical seismic force. To AISI S100-16 (with S2-20; AISI S220-20 sends nonstructural members to S100), with ASCE 7-22 13.3.1.6 and the load combinations.

Inputs of the ceiling framing

- **Member:** a C-section by SSMA designation or custom dimensions, or a hat channel by its depth, crown width (the face under the board), flange width on each side and design thickness. Fy.
- **Top flange brace spacing (C):** the spacing of bridging or blocking that braces the compression flange in the spans; 0 takes it as braced. Your call. Over interior supports the bottom flange is in compression and is taken as braced by the board.
- **Hat braced:** the hat's lateral-torsional buckling is taken as prevented by the board screwed to the crown and the flanges screwed at each support. You confirm it; this type has no other method for a hat.
- **Spans:** the span between supports, the number of equal continuous spans, and the member spacing (the tributary width). ASTM C754 spacing tables are yours to apply.
- **Loads:** area loads (D or L) and point loads (fixtures, equipment) by type; self-weight. **Vertical seismic:** Ev = 0.2 SDS D (ASCE 7-22 13.3.1.6 with 12.4.2.2) as the load type E, up and down, in the seismic combinations.
- **Hangers:** diameter and tensile strength of the wire or rod. **Deflection:** the total load limit (L/360 prefilled, IBC Table 1604.3).

Auto-design varies the C-section (SSMA designations, lightest first); Fy when unlocked. A hat channel or a custom section is kept.

Checks performed

Check	Basis
Flexure of a C: lateral-torsional buckling over the brace spacing (Cb = 1.0) and local buckling	S100 F2, F3
Flexure of a hat: effective section (flanges k = 0.43, crown k = 4, webs under gradient), Fn = Fy	S100 F3, Appendix 1 1.1, 1.1.2, 1.2.1
Shear (the two webs of a hat)	S100 G2
Bending and shear	S100 H2
Hanger tension, 0.75 Fu Ab, φ 0.75, Ω 2.00	AISC 360-22 J3.7, applied to the hanger
Hangers stay in tension	ASCE 7-22 13.3.1.6 (advisory)

Check	Basis
Total load deflection, effective inertia at the service moment	IBC Table 1604.3 (limit entered), S100 L1

Limitations

- The hat's properties are computed on its square-corner centerline: corner radii are left out, and the webs are taken vertical.
- Web crippling is not checked: the reactions are taken through hangers, clips or screws, not by bearing.
- The hanger strength rule is the threaded-rod rule, conservative for an unthreaded wire; product or code values for wire may be higher.
- The lateral bracing of a suspended ceiling (ASCE 7-22 13.5.6), the attachments to the structure above and ASTM C754 prescriptive limits are not part of this calculation.

Verification examples

Verification status: 8 values compared in 4 examples (0 published, 4 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published ceiling framing example is in the free library. The values are hand calculations: the hat's gross properties on the stated model, the statics of one and two spans, the vertical seismic combination and the hanger strength.

CHAPTER 184

Cold-formed steel girt

A C- or Z-section wall girt spanning between columns, its outside flange through-fastened to the wall panel: the wind toward the wall and away from it, the gravity load carried by sag rods, the girt's strength in each region and its deflection. To AISI S100-16 (with S2-20) Chapters F, G, H and I6.2.1, with the wall pressures of ASCE 7-22 Chapter 30.

Inputs of the girt

- **Section:** a listed Z- or C-section (named as the AISI Design Manual names them; confirm the dimensions against the maker's data) or custom dimensions; Fy and Fu (55 and 70 ksi are usual for girts).
- **Spans:** one to four equal spans; more than one is continuous and prismatic. Lapped girts are modelled by the cold-formed steel purlin type. The girt spacing is its tributary height.
- **Wind:** the net components and cladding pressures you enter (strength level, toward and away from the wall), or computed for wall zone 4 or 5 of ASCE 7-22 Fig. 30.3-1 (h at most 60 ft) from V, exposure, h, Kzt, Ke and the enclosure. The effective wind area is the span times the larger of the spacing and a third of the span (ASCE 7-22 26.2). The 0.9 factor of note 5 (roof slope 10° or less) is an option, off by default.
- **Suction, free flange in compression:** the R method of S100 I6.2.1 (the default) or discrete bracing of the free flange at the sag rods. With the R method each condition (a) to (o) of I6.2.1 is a check: those this type can see from the section, spans and steel, and those it cannot (laps, panels, insulation, fasteners) as inputs you confirm. Insulation thickness reduces R on a simple span (Eq. I6.2.1-2).
- **Rotational restraint kφ** of the panel on its flange for F4; 0 is conservative.
- **Gravity:** carried by sag rods (the default: the girt is checked about its weak axis between rods, and the rod in tension), or by the wall panel in its plane, which is your judgment and switches the weak-axis check off. The panel weight, the rods per span, the girts hung from one rod, the rod's diameter and Fu.
- **Deflection:** the wind factor (0.42, IBC Table 1604.3 footnote) and your limit. L/90 is prefilled, the IBC value for girts supporting metal siding (L/120 for flexible finishes).

Auto-design varies the section (listed sections of the profile entered, lightest first) and the sag rods per span. Spans, spacing, wind, the panel conditions and Fy (unless unlocked) are kept.

Checks performed

Check	Basis
Bending, panel flange in compression (pressure at mid-span, suction over a support)	S100 F3 with Fn = Fy
Distortional buckling, panel flange, kφ entered	S100 F4, Appendix 2 2.3.3.3
Bending under suction, free flange in compression: Mn = R Mno	S100 I6.2.1, Table I6.2.1-1, conditions (a) to (o)
or braced at the sag rods	S100 C2.2, F2, F3, F4

Check	Basis
Bending over an interior support under pressure (continuous)	S100 F2, F3, F4 (I6.2.1 does not apply there)
Shear, bending and shear	S100 G2, H2
Wind and gravity bending, weak axis between sag rods	S100 H1.2 with no axial load
Sag rod tension	AISC 360-22 J3.7
Wall pressures (computed)	ASCE 7-22 26.10, Fig. 30.3-1, Eq. 30.3-1
Deflection	IBC Table 1604.3 (limit entered), S100 L1

Wind governs the strong axis alone: LRFD 1.0W, ASD 0.6W, in both directions. The weak-axis check pairs each combination's dead load factor with its wind.

Limitations

- Equal spans, no laps, uniform wind. Openings, jambs and point loads are not modelled.
- The weak-axis strength is the gross weak-axis section modulus at Fy, and the moment between sag rods is taken as simple ($w\,a^2/8$). This is a simplification to confirm against an effective-section method for weak-axis bending, which this type does not build.
- The girt is taken as bolted through its web to clips at the columns: web crippling is not checked, and the clips are not designed.
- Axial load from wall bracing (S100 I6.2.3) is not included.
- The plan for this type cited S100 "I6.3" for discrete bracing; in S100-16, I6.3 is the standing seam panel system. Discrete bracing here is C2.2 with F2.

Verification examples

Verification status: 12 values compared in 4 examples (0 published, 4 hand calculations from the cited equations): 12 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a wall girt is in the free library. The section properties and Se come from the same Z-section library the purlin type verifies against the MBMA Roof Framing Design Guide and AISI RP20-5 (Se of 8ZS2.75x085 at 55 ksi, 2.84 in^3, Guide p. 83). The moments, R, the wall pressure and the sag rod are hand calculations from the cited equations. An AISI D100 or MBMA girt example would replace them.

CHAPTER 185

Built-up cold-formed steel member

Two lipped C-sections joined back to back or toe to toe (a box), as a column, a beam or both: the member strengths of the pair and the connections that make it act as one. To AISI S100-16 (with S2-20) I1.1 and I1.2, with Chapters E, F, G and H.

Inputs of the built-up member

- **C-section:** an SSMA designation or custom dimensions, and Fy.
- **Built-up section:** back to back (webs in contact) or a box (lips in contact). S100 gives no torsional or lateral-torsional buckling method for a box of two open sections: the box needs your confirmation that its connections close it, and torsion is then left out.
- **Length and loads:** a simple span with an axial load (compression positive) and a uniform transverse load bending the strong axis, by load type.
- **Bracing:** Kx, Ky, Kt, the weak-axis and torsional brace spacing, a compression flange braced throughout or not, Cb (1.0 is conservative), distortional buckling with kφ.
- **Connections:** the intermediate connector spacing a, the screw, the screws at each tie, the end connection (screws in a line, or a weld), the distance g between the screw rows of a back-to-back web (0 takes D − 2 in.), Fu (0 takes the ASTM A1003 minimum).
- **Deflection limit:** span over n, entered.

Auto-design varies the C-section (lightest first) and the connector spacing. Length, loads, bracing, the pack and Fy (unless unlocked) are kept.

Checks performed

Check	Basis
Compression with the modified slenderness	S100 E2, E3, Eq. I1.2-1
Connector spacing a/ri at most half the governing slenderness	S100 I1.2(a)
End connectors at most 4 diameters apart over 1.5 times the width	S100 I1.2(b)
Each intermediate tie carries 2.5 % of the available axial strength	S100 I1.2(c), J4.3
Distortional buckling in compression and bending	S100 E4, F4 (one C)
Bending	S100 F2 (the doubly symmetric pair), F3
Connection spacing in bending, back to back	S100 I1.1, Eq. I1.1-1, J4.4.1
Shear, bending and shear	S100 G2, H2
Compression and bending	S100 H1.2 with B1 (C1.2.1.1, Cm = 1.0)
Deflection	IBC Table 1604.3 (limit entered), S100 L1

Limitations

- Simple span, uniform transverse load about the strong axis. Weak-axis bending, end moments and point loads are not modelled.
- Distortional buckling is that of one C. S100 does not credit the lips in contact in a box.
- The box's torsional and lateral-torsional buckling are excluded on your confirmation only.
- Bearing (web crippling) and the member's end connections to the structure are not designed.
- I1.1 in bending takes one screw at each connection for Ts (pull-out only; pull-over and screw tension are not computed).

Verification examples

Verification status: 7 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a built-up cold-formed member is in the free library. The values are hand calculations from the cited equations, on the gross properties of one C that the stud type verifies against the SSMA tables: the factored loads, the modified slenderness, the end connector count, the L/6 limit and the box's moment of inertia. AISI D100 examples would replace them.

CHAPTER 186

Cold-formed steel jamb studs

The studs beside a wall opening: the wind on their tributary width, the gravity load the header brings down, the jamb's strength as one stud or a pack, its deflection and its connection to the track at the head and the base. To AISI S100-16 (with S2-20) and AISI S240-20 B3.2.5.

Inputs of the jamb studs

- **Stud and jamb:** an SSMA designation or custom dimensions, Fy, and the jamb: one stud (with what faces the opening), two to four studs side by side (each counted alone, braced like the others), two back to back (S100 I1.2 with the connector spacing a), or a box (with your confirmation that the connections close it).
- **Wall and loads:** the stud height (simple span), the opening width and the stud spacing. The tributary width for wind is half the opening plus half the stud spacing, as usual practice takes it; enter another value if your opening frame says otherwise. The wind is the components and cladding pressure at the strength level. The header reaction and any load from above go in the axial table by load type.
- **Bracing and deflection:** bridging spacing (0 for sheathing on both flanges), Kx, Cb, distortional buckling with kφ, the wind factor for deflection (0.42, S240 B1.1.2) and the limit (L/360 prefilled for brittle finishes at openings; IBC Table 1604.3 lists the others).
- **Head and base:** a track (stud-to-track by S240 B3.2.5.1), a deflection track at the head (B3.2.5.2), or a clip designed elsewhere; the track's thickness, Fy, Fu and flange, the screw, whether both stud flanges are screwed.

Auto-design varies the stud (lightest first) and, for studs side by side, their number. The opening, wind, loads, tracks and Fy (unless unlocked) are kept.

Checks performed

Check	Basis
Compression	S100 E2, E3 (and I1.2 for back to back and box)
Distortional buckling, compression and bending	S100 E4, F4
Bending	S100 F2, F3
Shear, bending and shear	S100 G2, H2
Compression and bending	S100 H1.2 with B1 (C1.2.1.1, Cm = 1.0)
Connector spacing	S100 I1.2(a)
Deflection under wind	IBC Table 1604.3 (limit entered), S240 B1.1.2, S100 L1
Stud to track at the head and base	S240 Eq. B3.2.5.1-1, Table B3.2.5.1-1 (at an opening)
Track at back-to-back jamb studs	S240 Eq. B3.2.5.1-3, Table B3.2.5.1-4
Outside those ranges, or one flange screwed	S240 B3.2.5.1(e): S100 G5, halved next to an opening

Check	Basis
Deflection track at the head: stud web crippling	S240 B3.2.5.2.1

Limitations

- The window or door frame's loads are taken as a uniform wind on the tributary width; the frame's anchor points are not modelled.
- Studs side by side are not in Table B3.2.5.1-1: their track check is S100 G5, halved next to an opening.
- A deflection track at a jamb carries the jamb's reaction over a short length, which Eq. B3.2.5.2-1 does not cover: check it with the cold-formed steel track type, or use a clip.
- The I1.2(b) and (c) end and tie connections of a back-to-back jamb are checked by the built-up member type, not here.
- The header, its connection to the king studs, the sill and the track anchors are not designed.

Verification examples

Verification status: 9 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of jamb studs is in the free library. The values are hand calculations from the cited equations: the tributary width and wind line load, the reactions, the moment and S240 Eqs. B3.2.5.1-1 and -3. The member strengths come from the libraries the stud and header types verify against the SSMA tables. A CFSEI technical note or AISI D110 example would replace them.

CHAPTER 187

Cold-formed steel track

The top or bottom track of a wall, standard or deflection (slip) track: the stud's lateral reaction into it, the stud's and the track's strength there, the movement gap and the track's fasteners to the structure. To AISI S240-20 B3.2.5 with AISI S100-16 G5 and J4.

Inputs of the track

- **Studs:** an SSMA designation or custom dimensions, Fy, the spacing, the configuration of Table B3.2.5.1-1 (single stud, double studs, at an opening or not), and the lateral reaction per stud at this track by load type.
- **Track:** standard or deflection; thickness (mils), Fy and Fu (ASTM A1003: 45 ksi for 33 ksi steel, 65 ksi for 50 ksi), the flange; for a standard track whether both stud flanges are screwed, and the screw size.
- **Deflection track:** the flange, the slip gap e (stud end to track web), the structure's movement that closes the gap and your tolerance (both your values), and the distance from the stud's web to the track's end.
- **Track to the structure:** screws into a steel support (its thickness and Fu, S100 J4.3), or anchors or power-actuated fasteners with the available strength you take from their evaluation report; the spacing along the track and the fasteners at each location.

Auto-design varies the track thickness (thinnest first), the fastener spacing and their number. Studs, reaction, gap and track type are kept.

Checks performed

Check	Basis
Stud to standard track, both flanges screwed	S240 Eq. B3.2.5.1-1, Tables B3.2.5.1-1, -2
Track thinner than an interior stud	S240 Eq. B3.2.5.1-2
Track at back-to-back jamb studs	S240 Eq. B3.2.5.1-3, Table B3.2.5.1-4
Otherwise: stud web crippling, halved next to an opening	S240 B3.2.5.1(d), (e); S100 G5
Deflection track within Table B3.2.5.2-1, fastened at no more than the stud spacing	S240 B3.2.5.2.2
Deflection track: stud web crippling, bearing at most the engagement or 1 in.	S240 B3.2.5.2.1, S100 G5
Deflection track flange	S240 Eqs. B3.2.5.2-1, -2
Stud web at least wdt/2 from the track's end	S240 B3.2.5.2.2
Gap at least the movement plus the tolerance	Your criteria
Screws into a steel support	S100 J4.3.1, J4.3.2
Anchors or power-actuated fasteners	Evaluation report (entered)

Limitations

- The reaction is the stud's end reaction you enter; the stud itself is designed by the stud or curtain wall types.
- The track's web between fasteners, its splices and slide clips are not designed.
- Anchorage into concrete is your entry from the product's report: this type does not apply ACI 318 Chapter 17 or S100 J7.
- Screw shear Pss is the value SSMA prints from CFSEI F701-12; use the manufacturer's value if it differs.

Verification examples

Verification status: 10 values compared in 4 examples (0 published, 4 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a track or deflection track is in the free library. The values are hand calculations of S240 Eqs. B3.2.5.1-1, -3, B3.2.5.2-1, -2, S100 G5 and J4.3.1. CFSEI technical notes W100 to W103 or AISI D110 would replace them.

CHAPTER 188

Web holes in cold-formed members

A C-section checked at the section of a web hole, punched, unpunched or reinforced, under the forces you enter there. To AISI S100-16 (with S2-20) F3.1.2, F4.2, G3, G6 and Appendix 1 1.1.1 and 1.1.3.

Inputs of the web hole check

- **Member:** an SSMA designation or custom dimensions, and Fy.
- **Web hole:** punched (circular, or noncircular with depth dh and length Lh), none (the unpunched member, to compare), or reinforced. The hole spacing along the member enters the G3 / G6 clear distance and the compression limits.
- **Reinforced:** S100-16 has no strength for a reinforced hole. The usual detail is a patch of the same or thicker steel lapped past the hole and screwed around it. Once you have designed it and confirm it, the member is checked as unpunched. The patch and its screws are your design.
- **Forces at the hole:** axial compression, moment and shear by load type, from your analysis of the member; the ASCE 7-22 combinations follow.
- **Bracing:** Kx Lx, Ky Ly, Kt Lt, a braced compression flange, Cb, distortional buckling with Lm and kφ.
- **Bearing near the hole:** the case (end or interior one-flange), the bearing length N, the flange fastening, the hole's distance from the edge of bearing x, and the reaction by load type.

Auto-design varies the section (lightest first). The hole, forces and bracing are kept.

Checks performed

Check	Basis
Hole limits (depth, length, dh/h, h/t, 9/16 in.)	S100 G3, G6, Appendix 1 1.1.3
Clear distance between holes at least 18 in.	S100 G3(d), G6(d)
Compression hole limits	S100 Appendix 1 1.1.1
Flexure at the hole	S100 F2 (gross), F3.1.2, Appendix 1 1.1.3(a)
Distortional buckling with the hole	S100 F4.2, Appendix 2 2.3.4.3
Shear at the hole, c/t at least 5	S100 G2, G3
Bending and shear	S100 H2
Compression at the hole	S100 E2, E3, Appendix 1 1.1.1
Compression and bending	S100 H1.2 with B1 (C1.2.1.1)
Web crippling next to the hole	S100 G5, G6 (Eqs. G6-1, G6-2)

Limitations

- The hole is taken centred in the web.
- The forces at the hole are yours; this type does not analyse the member.
- A hole within the bearing length needs a bearing stiffener (G6): not covered.
- A reinforced hole is checked unpunched on your confirmation; the patch is not designed (CFSEI technical note on web hole reinforcement, not on disk, to confirm against its text).
- The E3.1.2 exemption for small circular holes is not applied (conservative).

Verification examples

Verification status: 27 values compared in 15 examples (13 published, 2 hand calculations from the cited equations): 27 within tolerance, 0 recorded deviations. Code basis: not yet recorded for this module.

Published values: the SSMA Product Technical Guide (2022), pp. 10-12 (Vanet at the 1-1/2 × 4-1/2 in. punchout for six sections; Mal, Mad and Vag of five unpunched sections) and p. 59 (web crippling). **Hand calculations:** a circular hole by G3 and a hole near an end bearing by G6. No published example of a punched member in compression or of F4.2 is on disk (AISI D100).

CHAPTER 189

Cold-formed steel track anchorage

The bottom (or top) track of a wall fastened to concrete or steel: uplift and shear along and across the track per foot, the fastener at each location and the track sheet around it. To AISI S100-16 J3.3, J4, J5 and J7, with the loads combined by ASCE 7-22 2.3.1 or 2.4.1.

The track type (`cfs_track`) designs the stud-to-track connection under the studs' out-of-plane reaction. This type adds what that one does not: uplift, in-plane shear, the S100 power-actuated fastener equations (J5 and J7.2), pull-over of the track and the combined tension and shear check.

Inputs of the track anchorage

- **Track:** thickness (mils), Fy, Fu (ASTM A1003: 45 ksi for 33 ksi steel, 65 ksi for 50 ksi) and the web width; the fastener sits on its centreline.
- **Loads per foot of track,** nominal, by load type: uplift (dead load on the track negative), shear along and across the track. For a Type II shear wall, AISI S240-20 B5.2.4.3 asks the bottom track between the wall ends to be anchored for an uplift equal to the unit shear v plus any roof uplift: enter it as an uplift row.
- **Fasteners:** power-actuated fasteners (PAFs) into concrete or steel, screws into steel, or anchor bolts or screw anchors into concrete; the spacing and the number at each location.
- **PAFs:** shank diameter, head or washer diameter (at most 0.6 in. counts, J5), the pull-over factor αw of J5.2.3, the hardness (HRC 52 or more gives Fuh = 260 ksi), and the pull-out strength from the product's evaluation report (S100 gives no equation: J5.2.2, J7.1.2).
- **Concrete:** f'c, normal weight or not, thickness, embedment hET and edge distance, which decide whether S100 J7.2.2 applies; otherwise the PAF shear strength from the report.
- **Steel support:** thickness t2, Fy2, Fu2, and the bearing factor αb of J5.3.2.
- **Anchors:** diameter, washer, and the anchor's tension and shear strengths from ACI 318-19 Chapter 17 (the anchor types of this program) or its report.

Auto-design varies the fastener spacing (widest first), the number at a location and the track thickness. The fastener kind and size, the support and the loads are kept.

Checks performed

Check	Basis
PAF in concrete: pull-out in shear, Pnos = 1,450 lb (φ 0.50, Ω 3.25), with conditions (a) to (e)	S100 J7.2.2, Table J7.2.1-1
PAF tension strength Pntp, shear strength Pnvp	S100 J5.2.1, J5.3.1
PAF pull-out in tension	Evaluation report (S100 J5.2.2, J7.1.2)
Pull-over of the track	S100 J5.2.3 (PAFs), J4.4.2 (screws); J7.1.2 for anchors with the same form (to confirm against the code text)

Check	Basis
PAF in steel: bearing and tilting, pull-out in shear	S100 J5.3.2, J5.3.3
Track bearing at a PAF in concrete	S100 J7.1.3 with the Eq. J5.3.2-1 form (to confirm against the code text)
Screws: sheet shear, screw shear, pull-out, screw tension	S100 J4.3.1, J4.3.2, J4.4.1, J4.4.3
Track bearing at an anchor bolt	S100 J3.3.1 (mf = 0.75: no washer under both head and nut)
Anchor tension and shear	ACI 318-19 Ch. 17 or the report (entered)
Spacing and edge distance	S100 Tables J5.1-1, J7.2.1-1; J4.1, J4.2
Combined tension and shear	S100 J5.4, Commentary Eq. C-J5.4-1 with n = 1 (linear)

Limitations

- The concrete's own capacity (breakout, pryout, splitting) is in the strengths you enter; this type does not apply ACI 318 Chapter 17.
- The track's bending between fasteners under uplift, the studs, the stud-to-track connection and hold-downs are not designed.
- Combined tension and shear uses the linear form of Commentary Eq. C-J5.4-1 for every fastener kind; S100 J5.4 only asks for the interaction to be considered.
- The pull-over of the track at an anchor washer and the track bearing at a PAF in concrete use S100 equations written for steel-to-steel connections, a rational reading (to confirm against the code text).

Verification examples

Verification status: 16 values compared in 4 examples (1 published, 3 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: not yet recorded for this module.

The J7.2.2 shear strength (1,450 lb with φ 0.50 and Ω 3.25) is the value printed in AISI S100-16. **No published worked example of a track anchorage is in the free library;** the other values are hand calculations of the cited S100 equations.

CHAPTER 190

Cold-formed steel floor vibration

Walking vibration of a floor of cold-formed C-section joists bearing on walls: the joist panel's natural frequency, its effective weight and the predicted acceleration against the engineer's limit, by AISC Design Guide 11, 2nd Ed. (2016), the method the cold-formed steel industry applies to these floors. Every limit, damping ratio and driving force is an input with the Guide's value prefilled; the Guide gives recommendations, not code limits.

The steel floor vibration type covers beam, joist and girder floors with the girder and combined modes; this type is the joist panel of a cold-formed floor, with a sheathed floor option and the Guide's high-frequency method, which light cold-formed floors often need.

Inputs of the floor vibration

- **Joists:** an SSMA designation or custom dimensions, the spacing, the span, the floor width across the joists (Bj is at most two thirds of it), a free edge (Cj = 1.0), and continuity (the effective weight × 1.5 of DG11 4.1.2, your call).
- **Floor:** concrete on steel deck (depth above the deck, rib height, unit weight, f′c, and whether the slab acts with the joist: DG11 3.2 takes composite action for vibration where the deck is fastened) or sheathing (its stiffness Ds across the joists, your value from the panel's EI and any topping).
- **Day-to-day loads** (DG11 3.3, not design loads): the deck or slab, the actual superimposed dead load, and the live load (11 psf for paper offices in the Guide; less for residences).
- **Criteria:** damping ratio β (Table 4-2, cumulative), driving force Po (65 lb), the acceleration limit ao/g (0.5 %g offices and residences), how the limit is taken above 9 Hz, and a minimum frequency (DG11: floors below 3 Hz should be avoided).

Auto-design varies the SSMA joist (lightest first) and the joist spacing, for the least joist steel per square foot. The span, floor, loads, damping and limits are kept.

Checks performed

Check	Basis
Transformed moment of inertia (1.35 Ec, slab width S up to 0.4 L)	DG11 3.2
Natural frequency fn = 0.18 √(g/Δ)	DG11 Eq. 3-3
Effective width and weight, Bj = Cj (Ds/Dj)^¼ Lj ≤ ⅔ floor width, W = w Bj Lj	DG11 Eqs. 4-2, 4-3
Walking, fn ≤ 9 Hz: ap/g = Po exp(−0.35 fn)/(β W) ≤ ao/g	DG11 Eq. 4-1, Table 4-1
Walking, 9 to 15.4 Hz: equivalent sinusoidal peak acceleration ≤ the limit	DG11 Eqs. 2-10, 2-11, Table 2-2
Above 15.4 Hz	DG11 2.2.2: no criterion (a note)
Minimum frequency	DG11 4.1 (your value)

Limitations

- Joist panel only: joists on bearing walls or stiff supports. A floor on beams or girders needs the combined mode (the steel floor vibration type).
- Above 9 Hz the limit of DG11 Figure 2-1 is taken as rising in proportion to the frequency above 8 Hz (the ISO baseline shape the figure follows), to confirm against the code text; choose the flat limit to be conservative.
- The text of Eq. 2-10 in the library copy is garbled; it was rebuilt from Eqs. 1-6 and 2-7 to 2-9 as the Guide derives it, which gives its printed constant 154 (to confirm against the code text).
- Rhythmic activity, sensitive equipment, and the joists' strength and deflection are not part of this calculation.

Verification examples

Verification status: 7 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of a cold-formed joist floor is in the free library (the CFSEI technical note on floor vibration is not on disk). The values are hand calculations of DG11 Eqs. 3-3, 4-1 to 4-3 and 2-10 with the SSMA Guide (2022) section properties.

CHAPTER 191

Cold-formed steel soffit framing

Soffit and canopy framing of cold-formed C-section joists spanning from the wall to a line of hangers, under dead load and wind on the soffit in both directions, with the hangers to the structure above and a kicker taking the fascia's horizontal wind. To AISI S100-16 with the loads combined by ASCE 7-22 2.3.1 or 2.4.1.

Inputs of the soffit framing

- **Soffit joists:** an SSMA designation or custom dimensions, Fy, Fu, the spacing, the span from the wall to the hanger line, and the slope (the dead load is taken normal to the joist by cos θ).
- **Bracing:** the soffit board is on the bottom flange. Under net downward load the top flange is in compression and is braced at the bridging spacing you enter (0 = unbraced over the span), with Cb. Under net upward load the board braces the compression flange; its rotational restraint kφ (AISI S240-20 B1.2.2.3 and its Appendix 1) is your entry, 0 if unknown.
- **Loads:** the dead load and the wind pressure on the soffit, down and up: your ASCE 7-22 Chapter 30 components and cladding values (soffits take the pressure of the wall or overhang zone they adjoin, your reading). The deflection uses your wind factor (0.42, IBC Table 1604.3 footnote) and your limit.
- **Hangers:** an SSMA C-section, its yield stress, length (pinned ends, K = 1.0) and spacing along the hanger line.
- **Kicker:** an SSMA C-section at an angle from horizontal, from the hanger's foot up to the structure, for the fascia's horizontal wind (fascia height and pressure) in both directions.
- **Screws** at each end of a hanger or kicker.

Auto-design varies the SSMA joist, the hanger and the kicker (lightest first) and the screws at their ends. Span, spacing, loads and limits are kept.

Checks performed

Check	Basis
Flexure, net downward (top flange braced at the bridging)	S100 F2, F3, F4 (Lm = the bracing spacing)
Flexure, net upward (the board's flange in compression)	S100 F3 with Fn = Fy and F4 with kφ (S240 B1.2.2.3, applied to the soffit: to confirm)
Shear at the supports	S100 G2
Deflection under D ± 0.42 W at the service section	IBC Table 1604.3 (your limit), S100 L1
Hanger in tension and compression (joist reaction and the kicker's vertical part)	S100 D2, E2, E3
Kicker in compression and tension	S100 D2, E2, E3
Screws at the ends of the hangers and the kicker	S100 J4.3.1, J4.3.2

Limitations

- The wind pressures on the soffit and fascia are your entries; this type does not compute ASCE 7 pressures.
- Simple span only. The joist's inner support (wall, ledger or clip), the structure above, the board and its fasteners, and web crippling where the reaction does not pass through screws are not designed.
- The hangers and kicker are pinned struts with K = 1.0 about every axis; a carrier member between hangers spaced wider than the joists is yours to design.
- The axial part of the dead load along a sloped joist is reported, not added to the joist.

Verification examples

Verification status: 10 values compared in 1 example (0 published, 1 hand calculations from the cited equations): 10 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of soffit framing is in the free library. The values are hand calculations of the combinations, the statics, the hanger and kicker forces, S100 J4.3.1 and the deflection.

CHAPTER 192

Cold-formed steel bypass and infill framing

Exterior wind studs that pass the slab edges on slide clips (bypass), or studs set between the slabs with a deflection track or slide clips at the head (infill). The stud over its spans, the clips' loads, screws and travel, and the infill head. To AISI S100-16 and AISI S240-20 B3.2.5, with the loads combined by ASCE 7-22 2.3.1 or 2.4.1.

The curtain wall type covers single and double span wind studs with a deflection track; this type adds multi-span bypass studs on slide clips, the clip reactions and screws, the clip's travel against the slab movement, and the infill head on slide clips.

Inputs of the bypass framing

- **Studs:** bypass or infill; an SSMA designation or custom dimensions, Fy, Fu, the spacing, the floor-to-floor height (bypass, 2 to 4 equal spans between clips, a cantilever above the top clip) or the clear height (infill), and the bridging spacing.
- **Loads:** the wind pressure, both directions (your ASCE 7-22 Chapter 30 value), and the cladding's dead load carried down the stud to its base (0 when the slabs carry it).
- **Limits:** the wind factor for deflection (0.42, IBC Table 1604.3 footnote), the span limit (L/600 behind brick veneer, L/360 plaster, L/240 other brittle finishes: your choice) and the cantilever limit.
- **Slide clips:** the clip's available strength from its manufacturer (for the stud and the anchorage to the slab), its thickness and Fu, the screws stud to clip, the clip's travel each way, and the slab edge's movement and your tolerance.
- **Infill head:** a deflection track (thickness, Fy, flange, slip gap) or slide clips.

Auto-design varies the SSMA stud (lightest first) and the screws per clip. Heights, spacing, wind, clips, head detail and limits are kept.

Checks performed

Check	Basis
Flexure, bridging spacing for F2 and F4	S100 F2, F3, F4
Shear	S100 G2
Bending and shear at a clip	S100 H2, Eq. H2-1
Compression from the cladding weight, with bending and B1	S100 E2, E3, H1.2, C1.2.1.1
Deflection in a span and at the top of a cantilever	IBC Table 1604.3 (your limits)
Slide clip reaction	Manufacturer's report (entered)
Screws stud to clip	S100 J4.3.1, J4.3.2
Clip travel at least the movement plus the tolerance	Manufacturer's travel, your movement

Check	Basis
Infill deflection track: stud web crippling, track flange, range, gap	S240 B3.2.5.2, Eqs. B3.2.5.2-1, -2, Table B3.2.5.2-1; S100 G5

Limitations

- Equal spans between clips; the base is a pinned support (a track), designed in the track and anchorage types.
- The clip's anchorage to the slab edge is inside the clip strength you enter. The slab edge movement is your value.
- The compression from the cladding is taken at the base with the largest wind moment anywhere on the stud (conservative).
- Deflection uses the gross moment of inertia.

Verification examples

Verification status: 11 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published worked example of bypass or infill framing is in the free library. The values are hand calculations of two-span continuous beam formulas, S100 J4.3.1, S240 Eqs. B3.2.5.2-1, -2 and S100 G5.

CHAPTER 193

Sheathing-braced cold-formed steel stud

A wall stud braced by its sheathing, on both faces or on one face with discrete bracing of the other flange, under axial load and wind. The sheathing braced strength of AISI S240-20 B1.2.2 and B3.2, the stud without the sheathing (Eq. B1.2.2-1), the sheathing connections checked as braces (AISI S100-16 C2.3), and the all-steel alternative with bridging shown beside it.

The cold-formed steel stud type offers sheathing braced design with identical sheathing on both faces. This type adds one face with discrete braces, the brace demand on the sheathing connections, and the side-by-side comparison with all steel design.

Inputs of the sheathing-braced stud

- **Stud and loads:** an SSMA designation or custom dimensions, Fy, the height, the spacing (at most 24 in., S240 B1.2.2.1(b)), the axial load per stud by load type, and the wind pressure (both directions).
- **Sheathing:** both faces or one face; gypsum with its screws (the four rows of S240 Table B3.2-1) or other structural sheathing with your nominal stud load limit; the connector spacing (gypsum screws at most 12 in.); the sheathing's rotational restraint kφ (S240 Appendix 1, your entry, 0 if unknown).
- **One face sheathed:** the brace spacing of the unsheathed flange (S240 B1.2.2.3: at most 8 ft for curtain wall studs) and Ly = Lt for compression. S240 B3.2.1(b) gives twice the connector spacing for two faces; with one face the braces set it, your value.
- **Connections as braces:** the connector's lateral strength (available, for the method) and stiffness, from tests or a report. Left at 0, the demand is reported, not checked.
- **All-steel alternative:** the bridging spacing, also used for the stud without the sheathing.

Auto-design varies the SSMA stud (lightest first) and the sheathing connector spacing (widest first). Height, spacing, loads, sheathing and braces are kept.

Checks performed

Check	Basis
Studs at most 24 in. on center; gypsum screws at most 12 in.	S240 B1.2.2.1(b), B3.2.1(b)
Compression: Lx the height, Ly = Lt twice the connector spacing (two faces), K = 1; distortional with kφ	S240 B3.2.1(b); S100 E2, E3, E4
Flexure: F3 with Fn = Fy and F4 with kφ; with one face, F2 over the brace spacing for the other direction	S240 B3.2.2(b); S100 F2, F3, F4
Axial load and bending with B1	S100 H1.2, C1.2.1.1
Nominal stud load against the gypsum connection limit	S240 Table B3.2-1 (or your limit)
The stud without the sheathing, 1.2D + (0.5L or 0.2S) + 0.2W	S240 B1.2.2.4, Eq. B1.2.2-1 (LRFD factors)

Check	Basis
Sheathing connections as braces: strength 0.01 Pra and stiffness	S100 C2.3, Eqs. C2.3-1, C2.3-2 (a rational check)
All-steel alternative with bridging	S240 B1.2.2.1(a) (advisory, not counted)

Limitations

- The C2.3 brace check is a rational check (S240 does not require it for sheathing braced design); the connector values are yours. It takes the axial load only.
- With one face sheathed, Ly = Lt is your entry: S240 states the two-face rule only (to confirm against the code text).
- Distortional buckling is taken without discrete restraint (Lm = Lcrd) unless kφ is entered.
- Tracks, bridging connections and the sheathing's own span and shear are not designed here.

Verification examples

Verification status: 8 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: not yet recorded for this module.

The Table B3.2-1 limits are the values printed in AISI S240-20. **No published worked example of a sheathing-braced stud is in the free library;** the brace demand and statics are hand calculations of S100 Eqs. C2.3-1 and C2.3-2.

CHAPTER 194

Cold-formed steel special bolted moment frame

A one-story cold-formed steel special bolted moment frame (CFS-SBMF): lipped C beams bolted to HSS columns with the Standard's 8-bolt groups, designed so that slip and bearing at the bolted connections dissipate the energy and the beams and columns stay elastic. To AISI S400-20 Section E4, with the members to AISI S100-16 (with S2-20) and the combinations of ASCE 7-22. The Standard gives this system for the United States and Mexico only, and in LRFD only.

Inputs of the moment frame

- **Frame:** the height from the column base to the beam centerline h, the beam span between column centerlines, the number of columns in the frame line n, and the unfactored gravity loads on the beam (D, Lr, S as line loads).
- **Seismic:** the design story drift Δ and the elastic lateral stiffness K of the frame line, both from your analysis (Δ = Cd δxe / Ie, ASCE 7-22 12.8.6); the code-level seismic shear of the frame line Eh, which caps the design shear at Ω0 Eh (S400 E4.3.1; 0 removes the cap); Ω0 (3.0) and Cd (3.5) of ASCE 7-22 Table 12.2-1 line C.12; Ie; SDS for the vertical effect Ev = 0.2 SDS D; and the allowable drift ratio (0.025 prefilled, Table 12.12-1; 0 when the one-story footnote applies).
- **Bolted connection:** the 8-bolt configuration of Table E4.3.3-1 (spacings a, b and c), one or two channels (N), the snug-tight bolt tension T (10 kips for 1 in. bolts unless a higher value is approved), the hole oversize (1/16 in. for standard holes), whether you specify another hole type, the bolt grade and threads, the edge distance, and an optional bolt bearing plate welded to the beam web with its welds.
- **Beam:** the lipped C dimensions, Fy, Fu, Rt (1.1 for sheet of Fy ≥ 50 ksi, S400 Table A3.2-1), the A653 Grade 55 confirmation, the brace spacing and Cb.
- **Column:** the tube's depth, width, design wall thickness and outside corner radius, the steel (A500 B or C, A1085), Fy, Fu, Rt, and the effective length factors in and out of the frame plane (Kx = 2.0 prefilled for a pin-based sway frame: your call).
- **System limits:** one story without column splices, all columns engaged, pin-based columns, one size and detail of beam, column and connection, level support. These are requirements of E4.4.1, on by default; a condition you clear makes the frame not feasible.

Auto-design varies the beam thickness, the column wall (with an outside corner radius of 2t) and the bolt grade, lightest frame first. The frame, loads, drift, stiffness, bolt configuration and member depths are kept. A thicker beam web raises the connection's expected strength, so a thicker beam is not always better.

Checks performed

Check	Basis
System, beam, column and bolt limits	S400 E4.4.1 to E4.4.4; S100 J3.2, Table J3-1

Check	Basis
Expected strength Ve = VS + VB, Me = Ve h	S400 E4.3.3, Eqs. E4.3.3-1 to -7, Tables E4.3.3-1, -2
Design shear not more than Ω0 Eh	S400 E4.3.1
Beam flexure at the connection (Md and the gravity end moment), at midspan (gravity)	S100 F2, F3; S400 E4.3.1.1
Beam shear, bending and shear	S100 G2, H2
Column compression, compression or tension with bending, shear	S100 E2, E3, F2.1.4, F3, H1.1, H1.2, G2
Stability coefficient θ and the 1/(1 − θ) amplification	ASCE 7-22 12.8.7
Bolt shear against the expected bearing of the weaker part	S100 J3.3.1, J3.4; S400 E4.4.4 (our reading)
Bolt bearing plate welds	S400 Eq. E4.3.1.2-1; S100 J2.5
Design story drift	ASCE 7-22 Table 12.12-1 (limit entered)

How the expected strength is found: the slip component VS and the slip drift ΔS follow directly. The bearing component needs the bearing drift $\Delta B = \Delta - \Delta S - n\, Me/(h\, K)$, and Me itself depends on VB, so the calculation solves Eq. E4.3.3-3 for VB by bisection. Below the slip drift there is no bearing (VB = 0); beyond ΔB,max, VB stays at VB,max. CDB is interpolated linearly between the points of Table E4.3.3-2. R0 and the relative bearing strength use the beam web (with any bearing plate, whose thickness adds to the web when the holes are drilled through both, Commentary C-E4.3.1.2) and one column wall.

The gravity moment at the connection is taken as the fixed-end moment $w\, L^2/12$, the upper bound for the semi-rigid connection. At midspan the gravity moment is $w\, L^2/8$. The column axial load at an exterior column is its gravity share plus $2\, Md / L$.

Limitations

- The design story drift and the frame stiffness come from your analysis; this calculation does not analyse the frame.
- Base plates and anchorage, the roof diaphragm and collectors are not designed.
- With one channel per side, torsion of the C beam must be accounted for (E4.4.2(e)); this calculation says so and does not compute it.
- The column model is a cold-formed tube with its webs fully effective, which the S400 flat-ratio limit guarantees.
- The ductile-hierarchy check of the bolts (bolt shear against the expected bearing strength) is this calculation's reading of E4.4.4, which requires that a ductile limit state controls without giving an equation.
- Canada: S400 has no CFS-SBMF provisions for Canada.

Verification examples

Verification status: 11 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published CFS-SBMF design example is in the free library. The values are hand calculations from S400-20 E4.3.3 (Eqs. E4.3.3-1 to -7 and both tables) with a very stiff frame, so that the elastic drift is negligible and the bearing drift is known in closed form, and one case of the Ω0 Eh cap. A published example (AISI D110, a CFSEI design example) would replace them.

CHAPTER 195

Cold-formed steel shear wall chord studs

The built-up boundary studs (chords) at the end of a stack of cold-formed steel shear walls, story by story from the top: the overturning accumulated down the stack with the gravity the chord collects, the chord in compression and tension, the hold-down at the base of each story and the anchorage at the bottom. To AISI S100-16 (with S2-20), with the ASCE 7-22 combinations.

Inputs of the chord studs

- **Wall stack:** one wall length L for every story and the distance e from the wall end to the chord centroid (the chords are d = L − 2e apart).
- **Stories, top first** (up to 8): the story height; the story shear from wind and from seismic (Eh), at the strength level; the dead load on the wall at the top of the story (it resists overturning); the gravity loads the chord collects at that level (D, L, Lr, S); the chord stud designation of that story; the hold-down's strength in the method's terms (φRn for LRFD, Rn/Ω for ASD, from its evaluation report); and the hold-down screws into the chord (0 when the product's rating already covers them).
- **Chord studs:** one stud, studs side by side, two back to back (connector spacing a, S100 I1.2), or a box closed by its connections (your confirmation), Fy and Fu, the bracing length for the weak axis and torsion (your call, S240 B1.2.2), Kx over the story height, distortional buckling with an optional kφ, and the holes in one stud cross-section at the hold-down.
- **Seismic:** the amplification on the seismic effect for chords and hold-downs (Ω0 = 3.0 prefilled for walls with wood structural panels, ASCE 7-22 Table 12.2-1) and SDS for Ev = 0.2 SDS D.
- **Hold-downs and anchorage:** the screw size, the hold-down's thickness and Fu under the screw heads, the anchor rod diameter and Fu, and the concrete anchorage strength from ACI 318-19 Chapter 17 (0 = not checked here).

Auto-design is not offered: the studs are chosen story by story in the stories table.

Checks performed

Check	Basis
Accumulated overturning, chord tension and compression	Statics; ASCE 7-22 2.3, 2.4
Chord compression (each story)	S100 E2, E3, I1.2 (and E4)
Chord tension yielding and rupture at the holes	S100 D2, D3
Hold-down tension at the base of each story	Evaluation report (entered)
Hold-down screws: sheet shear, screw shear	S100 J4.3.1, J4.3.2
Anchor rod tension	AISC 360-22 J3.7
Concrete anchorage	ACI 318-19 Ch. 17 (entered)

The overturning moment at the base of a story is the sum of V h over that story and the stories above. The tension is T = (M − fD ΣwD L (L/2 − e)) / d, with the dead load factor of each combination (0.9D, 0.6D, and less Ev in the seismic combinations); the compression is M/d plus the accumulated gravity on the chord. The seismic part of M is multiplied by your amplification. S400 E1.4 allows the shear wall's expected strength instead when it is smaller; this calculation does not compute it, so the amplified load is the safe side.

Limitations

- One wall length for the whole stack, the walls stacked over one another. Walls of different lengths or offsets need their own calculation.
- The sheathing and its unit shear (the cold-formed shear wall type), the wall drift, the chord's bearing on the track and the hold-down-to-hold-down continuity between stories are not designed.
- Out-of-plane wind on an exterior chord is not combined here: use the jamb stud type for that.
- Screws into a built-up chord are checked against one stud's thickness, the safe side.

Verification examples

Verification status: 5 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 5 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published multi-story chord stud example is in the free library. The values are hand calculations of the accumulated overturning and the chord forces for wind (LRFD and ASD) and for seismic with the amplification and Ev.

CHAPTER 196

Cold-formed steel truss bracing

Permanent bracing of cold-formed steel trusses: a continuous lateral restraint (PLR) across a run of trusses at one or more points of a compression web or chord, anchored by diagonal bracing every m trusses. The brace force and stiffness follow AISI S100-16 (with S2-20) C2.3, which AISI S240-20 B4.5 names for the bracing of roof framing; the restraint and diagonal members and their screws are checked to S100.

Inputs of the truss bracing

- **Braced member:** a web or a chord, its SSMA section, Fy and Fu, its length, the in-plane unbraced length Lx (the web, or the chord panel), the number of equally spaced restraint rows n (Lb = L/(n + 1)), Kx, and its compression by load type from the truss design (the truss designer's forces). The combinations of ASCE 7-22 give the required compression Pr.
- **Maximum restraint spacing:** 10 ft prefilled, the bottom chord suggestion of the S240 Commentary to E6; 0 turns the check off.
- **Restraint and diagonals:** the truss spacing, the trusses in the run, the trusses per diagonal m (your layout; diagonals at about 20 ft is common practice in the CFS-BCSI guide), the brace force as a percentage of Pr (1 % by S100 Eq. C2.3-1; 2 % is the S240 B3.4.1.2 practice for wall studs), the restraint and diagonal sections and Fy, the diagonal's run along the restraint (in truss spacings) and its rise, the brace effective length factor K and Fu.
- **Screws:** the size, the count at each truss and at each end of a diagonal.

Auto-design varies the restraint and diagonal sections (lightest first) and the screws. The member, its forces, the spacing, the diagonal layout and the brace percentage are kept.

Checks performed

Check	Basis
Member compression with the restraint (Lb for y and torsion)	S100 E2, E3
Restraint spacing against your limit	S240 Commentary E6 (limit entered)
Brace force Prb	S100 Eq. C2.3-1
Restraint stiffness at the farthest truss against βrb	S100 Eq. C2.3-2a / -2b
Restraint compression and tension (m Prb)	S100 E2, E3, D2; S240 Commentary B3.4
Diagonal compression and tension (m Prb Ld / run)	S100 E2, E3, D2
Screws at each truss and at the diagonal ends	S100 J4.3.1, J4.3.2

The brace forces of the m trusses between diagonals add along the restraint. The restraint and the diagonal take all m of them, the safe side. The stiffness the system gives the farthest truss is Prb / δ with δ = Prb [m / kd + s m (m − 1) / (2 E Ap)], kd = E Ad cos²θ / Ld being the diagonal's stiffness along the restraint and the second term the shortening of the restraint segments. This system model is ours (to confirm against a published bracing design); screw slip is not in it.

Limitations

- The truss and its connections are the truss designer's; this calculation takes the member's force as entered.
- The diaphragm or wall that the diagonals deliver to, laps and splices of the restraint, and temporary installation bracing are not designed.
- AISI S202 (truss responsibilities) and the CFS-BCSI guide are not in the library: the bracing layout is your input.

Verification examples

Verification status: 7 values compared in 2 examples (0 published, 2 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published cold-formed truss bracing example is in the free library. The values are hand calculations of Eqs. C2.3-1 and C2.3-2 (LRFD and ASD, one and two rows), the accumulation over m trusses, the diagonal force and the system stiffness.

CHAPTER 197

Cold-formed steel storage rack

A selective pallet rack of cold-formed upright frames and pallet beams: the beams with the connector's rotational stiffness, the connectors, the perforated uprights, the frame bracing, seismic forces in the down-aisle and cross-aisle directions, overturning and the base plate. The governing standard is ANSI MH16.1-2021 (RMI) with AISI S100-16 (with S2-20) for the members.

ANSI MH16.1-2021 is not in the library. Every MH16.1 provision in this calculation is written from knowledge and marked "to confirm against the code text"; every factor it sets is an input with its usual value prefilled. Read the Specification (free from MHI) before relying on the results.

Inputs of the storage rack

- **Rack:** beam levels at a uniform spacing (the first level at the same height), the number of bays and the beam length, the frame depth and the bracing panel height.
- **Loads:** the product load per level per bay (all its pallets), the number of pallets (the unit load for impact), the rack's own dead load, impact on the beams as a fraction of one pallet (0.25 prefilled), and the product load factors for gravity (1.4 LRFD, 1.0 ASD prefilled) and with seismic (1.0 prefilled).
- **Beams and connectors:** the beam C-section, one C or two toe to toe as a box (closed by its connections, your confirmation), Fy, the connector's rotational stiffness F and its moment and shear strengths in the method's terms (all from the manufacturer's tests), the locking device's upward strength, and the deflection limit (L/180 prefilled).
- **Upright frames:** the upright C-section and Fy, the Q factor from the stub column test, the net-to-gross area and section modulus ratios for the perforations, Kx (1.7 down-aisle), Ky (1.0) and Kt (0.8) over the panel, Cm, and the bracing section and Fy.
- **Seismic:** SDS, Ip (1.0; 1.5 in areas open to the public), the product share of the seismic weight (0.67), Cs from SDS Ip / R or entered, R down-aisle (6.0) and cross-aisle (4.0), the height ratio of the first story's inflection point (0.5 prefilled, 0 for a pinned base: your call), and the product share that resists overturning.
- **Base:** the base plate's bearing area, the slab's f′c and the anchor's uplift strength.

Auto-design varies the beam, upright and bracing sections, lightest first. The geometry, loads, connector properties, the MH16.1 factors and the seismic inputs are kept.

Checks performed

Check	Basis
Beam flexure at midspan with the connector's end restraint, shear	S100 F2, F3, G2; MH16.1 (to confirm)
Beam deflection under the product load	MH16.1 L/180 (to confirm)
Connector moment (gravity with impact; gravity with down-aisle seismic), shear, locking device	MH16.1 connector tests (entered)

Check	Basis
Upright compression with the Q factor	S100 E2; MH16.1 (to confirm)
Upright compression and down-aisle bending	S100 H1.2
Frame bracing compression and tension	S100 E2, E3, D2
Anchor uplift from cross-aisle overturning	Anchor strength (entered)
Base plate bearing	AISC 360-22 J8

The beam's end moment on springs F at both ends is Me = (w L²/12) / (1 + 2EI/(F L)); the midspan moment is w L²/8 − Me and the deflection subtracts the end moments' share. The seismic forces are the equivalent lateral force method: V = Cs W with Cs = SDS Ip / R (the short-period plateau, never less than 0.044 SDS Ip), distributed by w h. Down-aisle, the portal method gives the column shears (end columns V/(2 bays), inside columns twice that) with the inflection at mid-height above the first level; the connector moment at a level adds the column moments above and below the joint. Cross-aisle, the frame's diagonals take the story shear along their length.

Limitations

- MH16.1-2021 provisions are from knowledge, to confirm. The Specification's other load cases (top level only loaded, partially loaded, the vertical impact of a dropped load on uprights, rack-supported structures) are not run.
- The portal method replaces a semi-rigid frame analysis; the first-story inflection ratio is your input. P-delta is the S100 H1.2 amplification only.
- The slab, the anchors' concrete strength, perforation patterns, the bolts of the bracing and row spacers are not designed.

Verification examples

Verification status: 7 values compared in 1 example (0 published, 1 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published rack design example is in the free library. The values are hand calculations of the spring-end beam moment, the seismic base shear and bracing force, the Q-factor effective area and the base bearing.

CHAPTER 198

Cold-formed steel sloped and mansard wall

A C-stud of a sloped wall or mansard, from the floor up its slope to the roof, optionally propped by a horizontal kicker to the building: the stud as a beam-column under dead, snow, roof live and wind loads, the kicker, and the screws at the roof, the floor and the kicker. To AISI S100-16 (with S2-20), with the ASCE 7-22 combinations.

Inputs of the sloped stud

- **Stud:** an SSMA designation or custom dimensions, Fy and Fu, the spacing, the slope θ from the horizontal (15° to 90°) and the length along the slope from the floor to the roof; the flange bracing (sheathing on both flanges, or discrete braces at a spacing) and the weak-axis and torsional length for compression (your call, S240 B1.2.2).
- **Kicker (optional):** its position along the stud from the floor, its SSMA section, Fy, horizontal length and K (1.0 for pinned ends).
- **Loads:** dead load on the sloped surface; snow and roof live on the horizontal projection (ps from ASCE 7-22 Chapter 7 with the slope factor, drifts and sliding snow being yours); wind normal to the surface toward it and away from it at the strength level (components and cladding of ASCE 7-22 Chapter 30; which zone fits a steep mansard is your call).
- **Connections:** the screw size, the counts at the roof, the floor and each kicker end, and the supporting framing's thickness and Fu.
- **Deflection:** the wind factor (0.42, S240 B1.1.2 and the IBC Table 1604.3 footnote) and the span limits for wind and for snow or roof live.

Auto-design varies the stud and the kicker (SSMA designations, lightest first) and the screws at the floor, the roof and the kicker. The slope, lengths, the kicker's position, the loads and the limits are kept; Fy when unlocked.

Checks performed

Check	Basis
Stud flexure, shear, bending and shear	S100 F2, F3, G2, H2
Stud compression and bending (B1 over the span of the station)	S100 E2, E3, H1.2, C1.2.1.1
Kicker compression and tension	S100 E2, E3, D2
Screws at the roof, the floor and the kicker ends	S100 J4.3.1, J4.3.2
Deflection normal to the stud, wind and snow or roof live	IBC Table 1604.3 (limits entered), S100 L1

The loads split into a part normal to the stud and a part along it. Dead load acts on the sloped area: w cos θ normal, w sin θ along. Snow and roof live act on the horizontal projection, cos θ of each unit length, then split the same way. Wind is normal. The stud is pinned at the floor (which takes the force along the stud) and at the roof (normal only),

with the kicker as a third support. A horizontal kicker's force is H = Rn / sin θ from the normal reaction Rn; its part H cos θ along the stud loads the stud below the kicker. The floor screws carry the resultant of the normal reaction and the axial force.

Limitations

- The roof connection is taken to carry no load along the stud; a detail that does needs its own model.
- One horizontal kicker, pinned at both ends. An inclined kicker is not modelled.
- The roof and floor framing, track or clip details beyond their screws, web crippling at the supports, bridging and the finish are not designed.

Verification examples

Verification status: 9 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: not yet recorded for this module.

No published sloped or mansard stud example is in the free library. The values are hand calculations of the load split, the axial force at the floor, the simple-span moment (LRFD and ASD), and the two-span kicker reaction and force.

CHAPTER 199

Wind loads

Wind pressures on an enclosed or partially enclosed rigid building to ASCE 7-22: the velocity pressure (Chapter 26), the main wind force resisting system (MWFRS) by the directional procedure (Chapter 27, Part 1) with level forces, base shear and overturning, and components and cladding (C&C) for buildings with h ≤ 60 ft (Chapter 30, Part 1) and h > 60 ft (Part 2). The enclosure is entered or classified from the opening areas (26.12). Signs, rooftop equipment, chimneys, tanks and towers are the Wind on other structures calculation (Chapter 29). The results are service-level (strength-level wind, ASCE 7-22 uses W directly in the load combinations); copy them into the member calculations as load type W.

Inputs of the wind loads

- **Wind speed and site:** the basic wind speed V for the building's risk category (Fig. 26.5-1 or the ASCE Hazard Tool; the risk category is recorded and selects the map you read V from), the exposure category B, C or D (26.7, the engineer's judgment of the upwind terrain), the ground elevation factor Ke (Table 26.9-1 from the ground elevation, or Ke = 1.0, which is always permitted) and the directionality factor Kd (Table 26.6-1, 0.85 for buildings).
- **Topography:** Kzt entered (1.0 on flat terrain) or computed by 26.8 for a 2-D ridge, a 2-D escarpment or a 3-D axisymmetric hill from H, Lh and the distance x from the crest (negative upwind, positive downwind), with the Fig. 26.8-1 equations and the K1/(H/Lh) of the chosen exposure (26.8.2 permits the equations for increased accuracy; the tabulated K1 of the figure are the Exposure C values). ASCE 7-22 26.8.1 has three conditions: condition 1 (the building is in the upper half of a hill or ridge or near the crest of an escarpment, as shown in Fig. 26.8-1) is a check box for the engineer; conditions 2 and 3 (H/Lh ≥ 0.2; H ≥ 15 ft, 60 ft in Exposure B) are checked by the program. Where they are not met, Kzt = 1.0.
- **Building:** the enclosure classification (GCpi ±0.18 for enclosed and partially open, ±0.55 for partially enclosed, Table 26.13-1), entered or **classified from the openings**: the opening areas of each long wall, each end wall and the roof (doors, vents and windows that may be open in a design storm, Commentary C26.12); the gross areas follow from the building (end walls with the gable triangle, the roof surface). Each wall is taken in turn as the wall receiving positive pressure (26.12.2): open where Ao ≥ 0.8Ag; partially enclosed where Ao > 1.1Aoi, Ao > min(0.01Ag, 4 ft²) and Aoi/Agi ≤ 0.20; enclosed where every wall has Ao ≤ min(0.01Ag, 4 ft²) and Aoi/Agi ≤ 0.20 (the Table 26.13-1 criterion; the 26.2 definition omits the ratio); otherwise partially open; a building both open and partially enclosed is partially open (26.12.1). An open building is not feasible. **Single unpartitioned large volume:** a partially enclosed building may use GCpi × Ri, Ri = 0.5 (1 + 1/√(1 + Vi/(22,800 Aog))) ≤ 1.0 (Eq. 26.13-1), with Vi entered or the building volume and Aog the total openings (needs the opening areas). the roof (flat, gable, hip), the plan length along the ridge and width across it, the eave height and the roof slope θ in degrees. The mean roof height is h = eave + rise / 2, except that the eave height is used for θ ≤ 10° (notation of Figs. 27.3-1 and 30.3-1 to 30.3-2G). The natural frequency n1 states that the building is rigid (n1 ≥ 1 Hz, 26.2; approximate values in 26.11.3). G is 0.85 or computed by 26.11.4.
- **Levels:** the floor and roof heights. Each level takes the wall load between the mid-heights of the stories next to it; the top level also takes the wall above it and the horizontal component of the roof pressures. The lower half of the first story goes directly to the foundation (it is part of the base shear).

- **Components and cladding:** one row per element: surface (wall or roof), zone and effective wind area A (span × the larger of the width and span/3, 26.2). Walls use zones 4 and 5 (Fig. 30.3-1). Roofs with θ ≤ 7° use zones 1′, 1, 2 and 3 (Fig. 30.3-2A); gable roofs above 7° use zones 1, 2 and 3 of Figs. 30.3-2B (7° to 20°), 2C (20° to 27°) and 2D (27° to 45°), hip roofs those of Figs. 30.3-2E (7° to 20°), 2F (20° to 27°) and 2G (45°), with the note 6 linear interpolation between 2F and 2G for 27° to 45°. ASCE 7-22 has no zones 2e, 2n, 2r, 3e, 3r on these figures (they were 7-16 zones); a row with such a zone, or a roof above 45°, needs GCp+ and GCp− entered. Entered values override the figure for any row. **Parapet ≥ 3 ft** (θ ≤ 7°): Fig. 30.3-2A note 5, zone 3 suction taken as zone 2 and the positive GCp of zones 2 and 3 as wall zones 4 and 5 (the wall values of Fig. 30.3-1 without the note 5 reduction). **h > 60 ft (Part 2, Eq. 30.4-1, Fig. 30.4-1):** walls zones 4 and 5 (a = 10 % of the least width, ≥ 3 ft), positive pressure with qz at the component's height z (column Height z; blank = h) and suction with qh, qi = qh; roofs up to 7° zones 1, 2 and 3 (no positive roof curve is given; with a parapet ≥ 3 ft zone 3 is treated as zone 2, note 7); steeper roofs use the Part 1 roof figures with qh (note 6). The 30.4 exception (60 ft < h < 90 ft and h ≤ the least width: Part 1 figures permitted) is a check box.

Checks performed

Check	Basis
Velocity pressure qz, qh	Eq. 26.10-1 qz = 0.00256 Kz Kzt Ke V²; Kz = 2.41 (z/zg)^(2/α), z ≥ 15 ft (Table 26.10-1 note 1) with α and zg of Table 26.11-1; Ke = e^(−0.0000362 ze) (Table 26.9-1 note 2); Kzt by Fig. 26.8-1
Gust-effect factor	0.85 (26.11.1) or Eqs. 26.11-6 to 26.11-9 for a rigid building, per wind direction
MWFRS pressures, two directions	p = q Kd G Cp − qh Kd (GCpi) (Eq. 27.3-1), windward wall with qz at each height, leeward wall Cp by L/B, side walls −0.7, roof Cp by h/L and θ (Fig. 27.3-1), both GCpi signs
Level forces, base shear, overturning	Windward plus leeward wall pressures integrated over each level's band (internal pressures cancel), plus the horizontal component of the roof pressures (the larger of the two roof cases, never below zero: Fig. 27.3-1 note 6)
Minimum MWFRS load	27.1.5: 16 psf on the wall area and 8 psf on the roof area projected on a vertical plane, applied simultaneously; when it is larger, the level forces are those of the minimum case
Load cases	Fig. 27.3-8: Case 1 is computed; the Case 2 torsion 0.75 V × 0.15B is stated for each direction
Enclosure	26.2, 26.12.1, 26.12.2, Table 26.13-1 from the opening areas; Ri by Eq. 26.13-1
C&C pressures, h > 60 ft	p = q Kd (GCp) − qh Kd (GCpi) (Eq. 30.4-1), Fig. 30.4-1 by zone and area (linear in log A), qz at the component height for positive wall pressure; not less than 16 psf
C&C pressures	p = qh Kd [(GCp) − (GCpi)] (Eq. 30.3-1), GCp by zone and area from Figs. 30.3-1 and 30.3-2A to 2G (linear in log A between the printed breakpoints), walls × 0.9 for θ ≤ 10° (Fig. 30.3-1 note 5), edge distance a with the exception a ≤ 0.8h for θ ≤ 7° and a least dimension over 300 ft; not less than 16 psf (30.2.2)
Tornado	Note only: Ch. 32 is not computed

The drawings (on screen, in the report and as a PDF / DXF sheet) show the building section in each wind direction (wind from the left) with the external MWFRS pressures as bands whose depth shows the value: the windward wall by height, the leeward wall, and the roof surfaces or the roof zones from the windward edge with both Cp cases; the heights are dimensioned to scale, and a building longer than the sheet holds at 1/16" = 1'-0" is compressed along its length and marked NTS. When wall components were entered, a wall elevation shows the C&C zones 4 and 5 with the width a and their design pressures. The sheet notes give qh, the internal pressure to add, the side-wall pressure and the roof C&C pressures; the roof C&C zone extents, the load cases 2 to 4 of Fig. 27.3-8 and the lateral system are not part of the calculation and are not drawn. The level forces and base shears are in the results.

Limitations

- Not covered (reported as not feasible): open buildings (entered or classified), flexible buildings (n1 < 1 Hz), roof slopes of 90°, roof C&C above 45° or in zones the 7-22 figures do not have, without entered GCp, the Part 1 exception outside 60 ft < h < 90 ft and h ≤ the least width.
- Not covered and not checked: the envelope procedure (Ch. 28), other structures and building appurtenances (Ch. 29: the Wind on other structures calculation; solar panels), parapet loads (27.3.4, 30.8) and roof overhangs (30.7), elevated buildings' bottom surfaces (30.3.2.1, 30.4.2.1), qz at the highest opening for the positive internal pressure of a partially enclosed building (qh, permitted as conservative), monoslope, stepped, multispan, mansard and domed roofs, tornado loads (Ch. 32), the reduction factor Ri for large partially enclosed buildings when not chosen (Ri = 1.0, conservative), the note-b area reduction of the −1.3 roof Cp of Fig. 27.3-1 (not applied, conservative), qi = qz at the highest opening of a partially enclosed building (qh is used, which 27.3.1 permits as conservative), the exception of Fig. 27.3-1 note 6 for moment-frame roofs (the roof horizontal component is never taken below zero).
- Kz is computed with the note 1 formula of Table 26.10-1, which the code permits; the table's rounded values differ by up to 1.5 % (Exposure B at 30 ft: 0.689 against 0.70).
- Above 45° the Fig. 27.3-1 roof Cp is the single positive value (0.6 at 60°, 0.01θ to 80°, 0.8 above) with the note 2 zero for the negative case.
- A hip roof uses the Fig. 27.3-1 gable values with the parallel-to-ridge Cp on the hip ends, and the roof widths of the hip geometry for the horizontal component.
- Every table, equation and figure was checked against the printed ASCE/SEI 7-22 (Tables 26.6-1, 26.9-1, 26.10-1, 26.11-1, 26.13-1; 26.8, 26.10, 26.11.4, 27.1.5, 27.3.1, 27.3.5, 30.2.2, 30.3.2; Figs. 26.8-1, 27.3-1, 27.3-8, 30.3-1 and 30.3-2A to 2G read from the page images). No item is left to verify.

Verification examples

Verification status: 171 values compared in 30 examples (20 published, 10 hand calculations from the cited equations): 170 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Values from the standard (ASCE/SEI 7-22; pages of the PDF on file):

Example	What it checks	Standard	ILX Calcs
T26.10-1-B, -C, -D	Kz at 15, 30, 60, 100, 200 and 500 ft for Exposures B, C, D (18 values, p. 338)	the table, two decimals	within 1.5 % (formula of note 1)
F26.8-1-ridge, -escarpment, -hill	K1 at H/Lh = 0.40, K2 at tabulated x/Lh, K3 at z/Lh = 0.20 (9 values, p. 337)	0.58 / 0.67 / 0.55; 0.34 / 0.75 / 0.61; 0.42 / 1.00 / 0.45	within 1 %
F30.3-1	Wall GCp, zones 4 and 5 at 10 and 500 ft² (6 values, p. 378)	+1.0 / +0.7, −1.1 / −0.8, −1.4 / −0.8	same
F30.3-2A to 2G	Roof GCp at the breakpoint areas printed on each figure (56 values, pp. 380 to 386)	figure values	same

Example	What it checks	Standard	ILX Calcs
C26.11-1	Gust-effect example, Exposure B, h = 600 ft, B = D = 100 ft (p. 943)	$\bar{z}$ 360 ft, $I_{\bar{z}}$ 0.201, $L_{\bar{z}}$ 710 ft, Q^2 0.616	same; G 0.818 against the table's 0.860, a documented deviation (the table's G does not follow from its own $I_{\bar{z}}$ and Q)

Published worked examples of the directional procedure to ASCE 7-22 (AISC Design Examples v16.0, Part III building; WoodWorks five-story-over-podium design example, 2nd ed., 2025):

Example	What it checks	Published	ILX Calcs
AISC-III-wind	Four-story steel building, V = 107 mph, Exposure C, h = 55 ft, 213 ft × 123 ft (pp. III-59 to III-63)	Kh 1.11, qh 32.5 psf, qz 30.5 / 28.1 / 24.9 psf, leeward Cp −0.5 / −0.35, windward p 23.8 to 19.4 psf, leeward −6.78 / −3.25 psf, floor loads 411 / 378 / 353 plf	same within 1 % (Cp −0.354 against the rounded −0.35)
WW-wind-T	Six-story building, V = 120 mph, Exposure C, h = 62 ft, 189 ft × 76 ft, transverse wind (pp. 55 to 56)	Kh 1.137, qh 41.9 psf, windward p 30.6 / 17.8 psf, leeward −8.7 / −21.6 psf, roof Cp −1.15 / −0.77 / −0.63, roof p −41.3 / −29.8 / −25.4 / +1.0 psf	same
WW-wind-L	Same building, longitudinal wind (pp. 57 to 58)	leeward Cp −0.276, p −1.9 / −14.8 psf, roof Cp −0.9 / −0.5 / −0.3, roof p −33.7 / −21.6 / −15.5 psf	same

The examples' parapet pressures (27.3.4) are not compared (parapets are outside this calculation). No published C&C worked example to ASCE 7-22 is on file (the ASCE wind loads guide remains on the documents list); C&C is checked against the figure values above and by hand. Hand calculations in closed form:

Example	What it checks	Hand calculation	ILX Calcs
W1	Flat roof, Exposure C, V = 100 mph, h = 15 ft, 100 ft × 50 ft	Kh = 0.851, qh = 21.79 psf, V = 30.70 kips and 12.99 kips, M = 230.2 kip-ft	same (within 0.1 %)
W1	C&C, wall zone 4 at 100 ft², zone 5 at 10 ft², roof zones 3 at 10 ft² and 2 at 50 ft²	+17.06, −26.67, −62.60, −39.07 psf	same
W2	Kz: B at 33 ft, D at 60 ft, C at 30 ft; Ke at 5,000 ft	0.707, 1.317, 0.980; 0.834	same
W3	Kzt, 2-D escarpment, H/Lh = 0.4, x = 50 ft downwind, z = 30 ft	1.486	same
W4	G of a rigid building, h = 30 ft, B = 100 ft, Exposure C	0.863	same
W5	Fig. 27.3-1 Cp, gable at 20°, h/L = 0.19	roof −0.3 / +0.2, leeward roof −0.6, leeward walls −0.5 and −0.3	same

Wave 6 additions (C&C for h > 60 ft, enclosure from the openings):

Example	What it checks	Standard / hand calculation	ILX Calcs
F30.4-1	Fig. 30.4-1 GCp at the breakpoints: walls zone 4 at 20 ft² (+0.9 / −0.9), zone 5 at 500 ft² (+0.6 / −1.0), roofs zone 1 at 10 ft² (−1.4), zone 2 at 500 ft² (−1.6), zone 3 at 10 ft² (−3.2), zone 3 with a parapet (−2.3, note 7) (p. 395)	figure values	same
W-P2	h = 100 ft, V = 115 mph, Exposure C: qh, wall zone 4 at z = 40 ft (qz), roof zone 3	42.44, +33.42 / −38.96, −121.93 psf	same

Example	What it checks	Standard / hand calculation	ILX Calcs
W-ENC	168 ft² door in one end wall, no other openings; unpartitioned volume	partially enclosed, GCpi 0.55; Ri 0.990, GCpi 0.545	same

CHAPTER 200

Seismic base shear (ELF)

The seismic base shear and its distribution over the height by the equivalent lateral force procedure of ASCE/SEI 7-22 Section 12.8, with the period, the seismic response coefficient Cs and its limits, story forces, story shears, overturning moments and story drifts.

Inputs of the seismic base shear

- **Seismic hazard:** SDS and SD1 (design spectral accelerations, from the ASCE Hazard Tool or 11.4.4), S1 (used for Eq. 12.8-7 when S1 ≥ 0.6 g and for SDC E / F when S1 ≥ 0.75 g), the long-period transition period TL (Chapter 22) and the risk category (Table 1.5-1). Ie comes from Table 1.5-2 (1.00, 1.00, 1.25, 1.50).
- **Cs method (12.8.1.1):** *Method 2* (prefilled) uses SDS and SD1 (Eqs. 12.8-3 to 12.8-5). *Method 1* uses the design spectral acceleration Sa at the period T (Eq. 12.8-2) from the multi-period design spectrum you enter (Sa = 2/3 of the USGS multi-period MCER spectrum at the periods of 11.4.5.1, 0 to 10 s): Sa is interpolated linearly, extended beyond 10 s by 11.4.5.1 item 3, and taken as the maximum Sa where T is below the period of the maximum. ASCE 7-22 permits either method where the multi-period spectrum is available; where Exception 2 of 11.4.5 applies (no multi-period values), Method 1 is not used.
- **SDS cap of 12.8.1.3 (option):** Cs and Ev with SDS = 1.0 but not less than 0.7 SDS. The calculation checks criteria 1 (no irregularities, from the irregularity input), 2 (five stories or fewer; count each mezzanine as a level), 3 (T ≤ 0.5 s), 6 (risk category I or II) and 7 (R ≥ 3); criteria 4 (ρ = 1.0 permitted by 12.3.4.2) and 5 (site class not E or F) are yours. Where a checked criterion fails the calculation is NOT FEASIBLE until the option is cleared.
- **SDC by Table 11.6-1 alone (option):** the exception of 11.6 where S1 < 0.75 g. Checked: (1) Ta < 0.8 Ts, (2) the period used for drift (the analytical period entered, else Ta) < Ts, (3) Cs by Eq. (12.8-2), that is Method 1, as the 7-22 text reads; condition (4) (rigid diaphragms, or vertical elements not more than 40 ft apart) is yours. When a condition fails, the SDC stays the more severe of the two tables and an advisory names it.
- **Structural system:** a system of Table 12.2-1 (R, Ω0, Cd, the Table 12.8-2 structure type and the height limits are taken from the table) or *Other system*, where R, Ω0, Cd, the structure type for Ta and whether it is a moment frame are entered. *Flexible diaphragms* applies footnote b (Ω0 of 2½ or more reduced by ½). The redundancy factor ρ (12.3.4) is your input: 1.0 in SDC B and C or when one of the conditions of 12.3.4.2 is met, 1.3 otherwise. The irregularity input feeds criterion 1 of 12.8.1.3 only.
- **Period:** Ta = Ct hn^x (Eq. 12.8-8) or Ta = 0.1N (Eq. 12.8-9: 12 stories or fewer, a system entirely of steel or concrete moment frames, average story height at least 10 ft). An analytical period may be entered; it is capped at Cu Ta (Table 12.8-1). With 0 the approximate period Ta is used.
- **Levels:** the height of each level above the base, its seismic weight wx (12.7.2) and, optionally, the elastic displacement δxe from the strength-level analysis. Rows may be entered in any order. The allowable drift coefficient Δa / hsx of Table 12.12-1 is an input (0.020 for risk category I or II, "all other structures", is prefilled; 0 = no limit, for the single-story case of note a).

Checks performed

Check	Basis
Fundamental period	Ta by Eq. 12.8-8 or 12.8-9 (Table 12.8-2 Ct, x); Cu by Table 12.8-1 (linear interpolation between the tabulated SD1 values); T = min(T analytical, Cu Ta) (12.8.2)
Seismic response coefficient	Method 2: Cs = SDS/(R/Ie) (Eq. 12.8-3), not above Eq. 12.8-4 (T ≤ TL) or 12.8-5 (T > TL); Method 1: Cs = Sa/(R/Ie) (Eq. 12.8-2). Both: not below Eq. 12.8-6 (0.044 SDS Ie ≥ 0.01) and, where S1 ≥ 0.6 g, Eq. 12.8-7 (0.5 S1/(R/Ie)); 12.8.1.3 SDS cap where selected
Base shear	V = Cs W (Eq. 12.8-1), W = Σ wx
Vertical distribution	Fx = Cvx V, Cvx = wx hx^k / Σ wi hi^k (Eqs. 12.8-12, 12.8-13), k = 1 to 2 by T (12.8.3); story shears (Eq. 12.8-14) and overturning moments (12.8.5)
Story drift	δx = Cd δxe / Ie (Eq. 12.8-16, δdi neglected as 12.8.6.5 permits), Δ = δx − δx-1, against Δa (Table 12.12-1), divided by ρ for systems of moment frames only in SDC D to F (12.12.1.1); the governing story is shown
Structural height limit	Table 12.2-1 for the SDC: a check where the table has no footnote (for example 65 ft for light-frame walls in SDC D to F); an advisory where a footnote (12.2.5.4 to 12.2.5.7, footnote j) can permit more
Analysis procedure	12.6 of ASCE 7-22 permits the ELF procedure for every structure (the Table 12.6-1 restrictions of 7-16 were removed; C12.6)
Seismic design category	Tables 11.6-1 and 11.6-2; S1 ≥ 0.75 g gives E or F; the 11.6 exception where selected (reference)
Load effects	Eh = ρ QE (Eq. 12.4-3), Ev = 0.2 SDS D (Eq. 12.4-4a; zero permitted in SDC B and at the soil–foundation interface, 12.4.2.2 Exception 2) and Ω0 reported for the combinations

The drawings (on screen, in the report and as a PDF / DXF sheet) show a schematic building elevation with the levels, their seismic weights wx and the story forces Fx as arrows, the story heights dimensioned and drawn to scale (the building width is not an input and is drawn schematic; a building too tall for 1/16" = 1'-0" is drawn not to scale), and beside it the story shear Vx and the overturning moment at the base of each story. The lateral system, the diaphragms and the horizontal distribution are not part of the calculation and are not drawn.

Limitations

- SDS, SD1 and S1 (and, for Method 1, the multi-period spectrum) are entered; the site coefficients and the site-specific procedures (Chapter 21) are not computed. For a site-specific spectrum, enter its values.
- Table 12.2-1 values are built in for the 15 listed systems; for any other line choose *Other system* and enter R, Ω0, Cd. The detailing requirements of the system and the provisions of 12.2.5 are not checked.
- Ta by Eq. 12.8-10 (masonry or concrete shear walls, Cw) is not offered; Eq. 12.8-8 is always permitted.
- The irregularities themselves (Tables 12.3-1, 12.3-2), the redundancy conditions of 12.3.4.2 and their additional requirements are not evaluated; ρ and the irregularity class are your inputs.
- Accidental torsion and its amplification (12.8.4.2, 12.8.4.3), P-delta (12.8.7), diaphragm forces (12.10), diaphragm deformation δdi and the drift with the period free of the Cu Ta limit (12.8.6.2) are outside this calculation; enter δxe from an analysis that includes them. 12.8.6.1 lets that analysis omit Eq. 12.8-6.
- The seismic weight W is the sum of the level weights entered (what 12.7.2 includes is the engineer's).
- The 11.6 exception condition (3) reads "Equation (12.8-2) is used to determine Cs"; in ASCE 7-22 that equation is Method 1 (in 7-16 the same number was SDS/(R/Ie)). The calculation follows the 7-22 text (to confirm against the code text: ASCE 7-22 errata or supplement for the 11.6 cross-reference).

Verification examples

Verification status: 157 values compared in 39 examples (36 published, 3 hand calculations from the cited equations): 153 within tolerance, 4 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Verified against ASCE/SEI 7-22 (PDF on file; pages are PDF pages): the tabulated values below are read from the rendered table pages, and the equations, limits and exceptions of 11.4.5.1, 11.6, 12.3.4, 12.4.2, 12.6, 12.8 and 12.12 from the text (PDF pp. 66 and 167 to 195).

Published worked examples to ASCE/SEI 7-22: the AISC Design Examples v16.0 building (Part III), FEMA P-2192-V1 (2020 NEHRP Provisions Design Examples, Chapters 4 to 7) and the WoodWorks five-story-over-podium design example (2nd ed., 2025). The period, Cs limits, base shear, force distribution, overturning and drift are compared with the values printed in each example:

Example	What it checks	Published	ILX Calcs
AISC-III-seismic	Four-story steel building, SDC B, R = 3 (pp. III-64 to III-69)	Ta 0.404 s, Cs 0.0430 (max 0.0792), V 356 kips, Fx 64.8 / 146 / 97.2 / 47.3 kips, Mx 940 / 3,790 / 7,940 / 12,700 kip-ft, Ev 0.0258 D	same within 0.5 %
P2192-4.5	23-story coupled concrete walls: T capped at Cu Ta, Cs, V, story drift from δx (pp. 147 to 151)	Ta 1.2 s, T 1.68 s, Cs 0.048, V 2,090 kips, drifts 0.28 % to 1.11 %, Δa 2.40 in.	same; the example's 0.044 SDS Ie = 0.029 uses SD1 in place of SDS (program 0.0484, a documented deviation; no effect on Cs)
P2192-5.5	18-story composite plate shear walls: k, Cvx (pp. 191 to 193)	Ta 1.21 s, T 1.70 s, k 1.6, Cs 0.048, Cvx 0.118 / 0.123 / 0.046	same; V 1,253 kips against 1,238 (the example rounds Cs to 0.048), a documented deviation that also moves Fx and the overturning moment by 1.2 %
P2192-6.4	Three-story CLT shear walls (p. 229)	Cs 0.333	same
P2192-7.4	One-story wood hall, Risk Category III (pp. 267 to 268)	Ta 0.22 s, Ie 1.25, Cs 0.192, V 16.57 kips	same (Cs,max 0.516 against 0.524: the example rounds Ta)
P2192-7.5	Six-story SCBF (pp. 273 to 275)	Ta 0.49 s, Cs 0.200, V 1,736 kips, Fx 302 / 478 / 382 / 287 / 191 / 96 kips	same
P2192-7.6	One-story tilt-up, R = 4 (pp. 286 to 287)	Ta 0.26 s, Cs 0.250, V 2,090 kips	same
WW-seismic	Five wood stories over a podium: Cs, Fx, drift (WoodWorks pp. 17 to 20, 49 to 50)	Cs 0.186, V 458 kips, Fx 133.5 / 129.6 / 97.2 / 64.8 / 32.4 kips, Δ 1.15 / 1.23 / 0.78 in., Δa 2.40 in.	same within 0.5 %

The procedure is also verified against hand calculations in closed form:

Example	What it checks	Published / hand value	ILX Calcs
T12.2-1 (15 lines)	R, Ω0, Cd of A.1, A.8, A.10, A.16, A.17, B.1 to B.4, B.26, C.1, C.3 to C.5, H (pp. 173 to 175)	45 values	same
T12.8-1	Cu at SD1 = 0.1, 0.15, 0.2, 0.3, 0.4 (p. 186)	1.7, 1.6, 1.5, 1.4, 1.4	same

Example	What it checks	Published / hand value	ILX Calcs
T12.8-2	Ct, x of the four structure types (p. 186)	0.028 / 0.8, 0.016 / 0.9, 0.03 / 0.75, 0.02 / 0.75	same
T1.5-2	Ie for risk categories I to IV (p. 66)	1.00, 1.00, 1.25, 1.50	same
EQ-1	Four-story SCBF, Cs by Eq. 12.8-3	Ta = 0.398 s, Cs = 0.1667, V = 633.3 kips, F4 = 215.1 kips, base overturning 25,009 kip-ft, Δ = 1.50 in. ≤ 3.12 in.	same within 0.1 %
EQ-2	Ten-story SMF, T capped at Cu Ta, Eq. 12.8-6	Ta = 1.290 s, T = 1.806 s, k = 1.653, Cs = 0.0352, V = 176 kips, F10 = 41.1 kips	same within 0.1 %
EQ-3	Twenty-story SMF, S1 = 0.8 g, Eq. 12.8-7	Ta = 2.246 s, Cs = 0.050, V = 1,000 kips	same

CHAPTER 201

Load takeoff

Gravity loads on a beam or a column from area loads over its tributary area, with the live load reduction of ASCE/SEI 7-22 Section 4.7 and the roof live load reduction of Section 4.8. The results are service loads by load type, ready to enter in a member calculation (which applies the load combinations).

Inputs of the load takeoff

- **Member:** a beam (span, tributary width, whether it supports a floor or the roof, and an optional self-weight added to D) or a column (tributary area per floor, the number of floors it carries, and the roof with its own tributary area).
- **Area loads:** one row per component, on the floor or on the roof, with its load type: D (dead), L (live, Table 4.3-1), Lr (roof live), S (snow, Chapter 7) and R (rain, Chapter 8).
- **Live load reduction:** the reduction is permitted, not required, so it is a choice. KLL is selected from Table 4.7-1. The occupancy selects the assembly rule (4.7.5, not reduced) or the passenger vehicle garage rule (4.7.4). Live loads above 100 psf follow 4.7.3 automatically. *One-way slab* limits AT to 1.5 times the span squared (4.7.6).
- **Roof live load reduction:** on or off, with the roof slope F in inches of rise per foot (4.8.2 R2).

Checks performed

Check	Basis
Loads by type	Line load w = q × tributary width (beam) or point load P = q × area × floors (column), each load type separately
Live load reduction	L = Lo (0.25 + 15/√(KLL AT)) (Eq. 4.7-1) where KLL AT ≥ 400 ft², not less than 0.50 Lo (one floor) or 0.40 Lo (two or more floors) (4.7.2); Lo > 100 psf and garages: not reduced for one floor, at most 20 % for two or more (4.7.3, 4.7.4); assembly: not reduced (4.7.5); one-way slabs: AT ≤ 1.5 ℓ² (4.7.6)
Roof live load reduction	Lr = Lo R1 R2 (Eq. 4.8-1 with R1 and R2 of 4.8.2), 12 psf ≤ Lr ≤ 20 psf, for ordinary roofs with Lo ≤ 20 psf (4.8.2)

For a beam the key results also list the simple-span end reactions of each load type.

The drawings (on screen, in the report and as a PDF / DXF sheet) show the tributary plan to scale: for a beam the span and the tributary strip (drawn centred on the beam; the calculation uses its width) with the tributary area, and the beam with its service line loads by load type on a simple span; for a column the tributary area per floor and of the roof, each drawn as a square of the same area around the column symbol. The sheet notes list the area loads and the reductions used. The member size, its supports and the framing are not part of the calculation.

Limitations

- One tributary area for all the floors a column carries (the same plan on every floor) and one for the roof.
- Concentrated live loads (4.4), partition loads (4.3.2) and impact loads are entered as area loads or left to the member calculation; snow drifts and unbalanced snow come from a snow calculation.

- Roof live loads above 20 psf are not reduced by Eq. 4.8-1 (it is for ordinary roofs, 4.8.2). An occupiable roof (4.8.3 permits the 4.7 reduction) is entered as L on the roof and is carried unreduced.
- KLL comes from Table 4.7-1; a calculated KLL (permitted by the table footnote) cannot be entered.
- The loads are service values; the load combinations are applied in the member calculation.

Verification examples

Verification status: 51 values compared in 19 examples (15 published, 4 hand calculations from the cited equations): 51 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Checked against ASCE/SEI 7-22 (PDF pp. 78 to 79): Eq. 4.7-1, the 0.50 Lo / 0.40 Lo limits, 4.7.3 to 4.7.6, Eq. 4.8-1 with R1 and R2 of 4.8.2, and Table 4.7-1. The seven KLL values of Table 4.7-1 are golden values (T4.7-1, p. 78). Published worked takeoffs: the AISC Design Examples v16.0 building (Part III, to ASCE/SEI 7-22; floor D 75 psf, L 80 psf, roof D 20 psf, Lr 20 psf, snow 24.6 psf):

Example	What it checks	Published	ILX Calcs
AISC-III-B45	45 ft interior beam at 10 ft (pp. III-25 to III-26)	KLL 2, L 60.0 psf, wD 0.750 kip/ft, wL 0.600 kip/ft	same
AISC-III-B30	30 ft interior beam at 10 ft (p. III-31)	L 69.0 psf, wL 0.690 kip/ft	same
AISC-III-G30	30 ft interior girder (p. III-43)	L 45.2 psf (AT rounded to 1,130 ft²)	45.3 psf (AT 1,125 ft²)
AISC-III-C2C	Interior column, 3 floors + roof, 1,125 ft² (pp. III-52 to III-55)	L 32.0 psf (0.40 Lo), Lr 12.0 psf, PD 276, PL 108, PS 27.7, PLr 13.5 kips	same
AISC-III-C2A, -C1A, -C1B, -C1C	Exterior and corner columns (pp. III-49 to III-55)	L 33.2 / 40.2 / 51.1 / 35.2 psf, R1 0.882 / 0.658, Lr 12.0 / 17.6 / 20.0 / 13.2 psf, PL 68.7 / 35.7 / 19.0 / 54.9 kips, PS and PLr	same within 1 %

The example's edge girder uses a KLL calculated from the influence area (1.98, Table 4.7-1 footnote); the program offers the tabulated KLL only, so that case is not compared. The arithmetic is also checked by hand calculations in closed form:

Example	What it checks	Hand calculation	ILX Calcs
LT-1	Interior beam, 30 ft × 10 ft, KLL = 2	L = 43.12 psf, wD = 0.500 kip/ft, wL = 0.431 kip/ft	same
LT-2	Interior column, 3 floors × 900 ft² and the roof	PD = 234 kips, L = 0.40 Lo = 20 psf, PL = 54 kips, Lr = 12 psf, PLr = 10.8 kips	same
LT-3	Roof beam, At = 400 ft², F = 6	R1 = 0.8, R2 = 0.9, Lr = 14.4 psf	same
LT-4	Storage column, Lo = 125 psf, two floors	L = 0.8 Lo = 100 psf, PL = 80 kips	same
T4.7-1	KLL of the seven elements of Table 4.7-1	4, 4, 3, 2, 2, 2, 1	same

CHAPTER 202

Snow loads

Roof snow loads to ASCE 7-22 Chapter 7: the flat and sloped (balanced) roof snow loads, the minimum load for low-slope roofs, the rain-on-snow surcharge, the unbalanced load of hip and gable roofs, drifts at a higher roof or an adjacent structure, at parapets and at roof projections, sliding snow from an upper roof and ice dams on overhangs. ASCE 7-22 snow loads are strength-level loads: the load combinations use 1.0S in LRFD and 0.7S in ASD (`calcs/loads.py`). Copy the results into the member calculations as load type S.

Inputs of the snow loads

- **Ground snow and site:** the risk category (it selects the ground snow load map and the minimum load limit pm,max of Table 7.3-4), the ground snow load pg for that risk category (7.2: the ASCE Hazard Tool, Figs. 7.2-1A to D, Table 7.2-1 for Alaska), the winter wind parameter W2 (Fig. 7.6-1 or the Hazard Tool), the surface roughness (26.7, plus the two special rows of Table 7.3-1) and the exposure of the roof (fully exposed, partially exposed, sheltered, per the Table 7.3-1 footnotes: a parapet taller than the balanced snow or large equipment makes a roof partially exposed). Ce comes from Table 7.3-1 or is entered.
- **Thermal condition:** heated with an unventilated roof (Table 7.3-3 by pg and the roof's thermal resistance Rroof, linearly interpolated as note a permits; Rroof above 50 gives 1.2, note b), cold ventilated or unheated (1.2), freezer (1.3), continuously heated greenhouse (0.85), or Ct entered (Table 7.3-2).
- **Roof:** flat or monoslope, gable or hip; the slope θ; the width across the ridge (W, the eave to ridge distance, is half of it for gable and hip roofs and the full width for a monoslope); the length; an unobstructed slippery surface (7.4: metal, slate, glass, smooth membranes, with room below the eaves for the sliding snow and no snow guards); simple prismatic rafters from eave to ridge (7.6.1: with W ≤ 20 ft the unbalanced load is pg on the leeward side); the eave overhang and whether an unventilated roof drains over its eaves (7.4.4).
- **Parapet and roof projection:** the parapet height above the roof and the roof length upwind of it (7.8); a roof projection (penthouse, equipment, screen wall) with its height, the length of its side (no drift under 15 ft) and the clear opening under it (no drift with 2 ft clear above the balanced snow), and the greater roof length upwind or downwind of it. **Windward drift taller than hc:** 7.7.1 gives the windward drift width as eight times the windward drift height but says nothing for a windward drift cut off at the clear height hc. The engineer chooses: height hc and width 8hc (as the AISC Design Examples v16.0, Part III, Table III-1), or height hc with the full width 8 × 0.75hd (conservative).
- **Higher roof or adjacent structure:** the height of the step h (to the upper roof edge, including any parapet), the upper roof length upwind of the step, this roof's length from the step (the fetch of the windward drift, and the 0.6 cap of the leeward drift), and the horizontal separation s (0 for the same structure; 7.7.2 for adjacent structures). **Sliding snow** from the upper roof: its slope, eave to ridge distance W, slipperiness and its own Ce and Ct (Table 7.3-1, Tables 7.3-2 and 7.3-3 for that roof).

Checks performed

Check	Basis
Balanced load	pf = 0.7 Ce Ct pg (Eq. 7.3-1); ps = Cs pf (Eq. 7.4-1); Cs from Fig. 7.4-1(a) for Ct ≤ 1.1, (b) for 1.1 < Ct < 1.2, (c) for Ct ≥ 1.2, the dashed lines for unobstructed slippery roofs of (b) and (c), written as the straight lines of Commentary C7.4; γ = 0.13pg + 14 ≤ 30 pcf (Eq. 7.7-1); hb = ps / γ
Minimum load	7.3.3: pm = pg ≤ pm,max (Table 7.3-4) for monoslope, hip and gable roofs under 15°, a separate load case
Rain-on-snow	7.10: 8 psf on the balanced case where 0 < pg ≤ pm,max and θ < W/50
Partial loading	7.5: stated with 0.5ps for the member analysis; gable members from 1/2 to 7 on 12 exempt
Unbalanced (hip, gable)	7.6.1, Fig. 7.6-2: between 1/2 and 7 on 12, 0.3ps windward, ps leeward plus hd γ/√S over 8hd√S/3 from the ridge (hd from Eq. 7.6-1 with lu = W; the extent is limited to W); short simple rafters: pg leeward, windward unloaded
Drift at a roof step	7.7.1: leeward hd from the upper roof length (≤ 0.6 of the lower roof length), windward 0.75hd from the lower roof length; w = 4hd, or 4hd²/hc ≤ 8hc with height hc; windward w = 8 × its height; none where hc/hb < 0.2; the larger height governs and the other is reported; a drift wider than the lower roof tapers to zero at its far end
Adjacent structure	7.7.2: leeward drift only for s < 20 ft and s < 6h, height ≤ (6h − s)/6, width min(6hd, 6h − s); the windward drift truncated in the separation (Commentary Fig. C7.7-4)
Parapet, projection	7.8: windward drift 0.75hd with lu as defined there; the projection exceptions
Sliding snow	7.9: 0.4 pf W per foot of eave over 15 ft (less over a narrower lower roof), slippery upper roofs over 1/4 on 12, others over 2 on 12; separated structures with h/s > 1 and s < 15 ft over 15 − s
Ice dams	7.4.4: 2pf on the overhang of an unventilated roof with Ct ≤ 1.1 draining over its eaves
Low ground snow	7.2 exception: stated (pg ≤ 10 psf with lu ≤ 100 ft, or ≤ 5 psf with lu ≤ 300 ft); the loads are always shown

The type is an analysis: a result reads **Calculated**. The drawings show the roof section with the balanced load, the unbalanced load of a gable or hip roof, each drift on its lower roof (load band from ps + pd at the step to ps at the drift width, the step height and the drift width dimensioned; a long lower roof cut with a break line) and the sliding load. The roof is drawn schematically one foot deep; the framing is not part of the calculation.

Limitations

- Not covered: curved, sawtooth, folded plate, barrel vault and dome roofs (7.4.2, 7.4.3, 7.6.2 to 7.6.4), air-supported structures (7.4.5), intersecting drifts (7.7.3), roof steps in series (Commentary C7.7), ponding (7.11 and Chapter 8), open-frame equipment structures (7.13), wind tunnel or flume studies (7.14).
- One drift of each kind per calculation (one step, one parapet, one projection); run a calculation per location.
- Fig. 7.4-1(a) carries a 5° mark with no curve: there is no slippery line for Ct ≤ 1.1 (Commentary C7.4: the ice dam at the eave obstructs sliding), so the unobstructed slippery input has no effect there.
- The density of a lower roof drift uses the ground snow load entered (the roof the drift sits on).
- Sliding snow is not capped by the upper roof's height above the lower roof (7.9 has no such cap).

Verification examples

Verification status: 59 values compared in 14 examples (9 published, 5 hand calculations from the cited equations): 55 within tolerance, 4 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples to ASCE 7-22:

Example	What it checks	Published	ILX Calcs
AISC-III-ROOF	AISC Design Examples v16.0 Part III roof: pg 30 psf, Rroof 40, partially exposed C (pp. III-5, III-6)	Ct 1.17, pf 24.6, pm 30, ps + 8 = 32.6 psf, γ 17.9 pcf, hb 1.37 ft	same
AISC-III-1	Table III-1 drifts: side parapet (lu 121 ft), end parapet (211 ft), both 2 ft, 6 ft screen wall (60.5 ft)	0.630 ft, 11.2 psf, 5.02 ft (both parapets); 2.37 ft, 42.4 psf, 18.9 ft	same (within 0.3 %)
C7-EX1	ASCE 7-22 Commentary Example 1: gable 6 on 12, Rroof 30	Ct 1.14, pf = ps 23.9 psf, hd 2.46 ft, surcharge 31.1 psf over 9.3 ft	same
C7-EX2	Commentary Example 2: fully exposed B, Rroof 20, pg 27.5, slope 41° (the curved-roof distribution is not computed)	Ct 1.07, pf 18.5 psf, Cs 0.72, ps 13.3 psf	same
C7-ROS	Commentary C7.10 rain-on-snow example, 1/4 on 12, 100 ft, risk category IV	ps 24.6, 32.6 with rain-on-snow, pm 32 psf	same
C7-EX3	Commentary Example 3: upper roof (Ct 1.13, pf 28.5) and lower unheated roof with a 10 ft step	pf 27, pm 25, hc 8.6 ft, pd 64.8 psf, peak 92 psf	same; hb, the drift heights and the drift width differ by 4 to 5 % because the example takes the density from the upper roof's ground snow load (deviation ASCE7-C7-EX3-GAMMA)

Hand calculations in closed form:

Example	What it checks	Hand calculation	ILX Calcs
H1	Sliding from a 30° slippery upper roof, W = 20 ft	pf upper 30.8 psf, 246.4 plf, 16.43 psf over 15 ft, total 50.03 psf	same
H2	Adjacent structure 6 ft away, h = 12 ft, upper roof 200 ft, pg 50 psf	leeward 5.83 ft, width 34.98 ft, pd 119.5 psf; windward truncated in the gap 2.68 ft	same
H3	Gable 4 on 12, W = 18 ft, simple rafters	leeward pg = 30 psf, windward 0	same
H4	Table 7.3-3 interpolation (Rroof 45, pg 25) and Fig. 7.4-1(b) slippery line at 30°	Ct 1.1875, Cs 0.667, pf 20.78, ps 13.85 psf	same
H5	3 ft step, 300 ft upper roof, 10 ft lower roof	hc 1.663 ft, drift cut at hc, width tapered to 10 ft, pd 29.76 psf	same

CHAPTER 203

Diaphragm forces

The in-plane seismic design force of a diaphragm, level by level, to ASCE 7-22 12.10, with the unit shear, chord and collector forces for a rectangular diaphragm spanning between two lines of vertical elements. Results are at strength level, with the 0.7 level beside them for allowable stress work. Copy them into the Wood diaphragm or other material calculation, which does the material checks: nothing links the two. Run one calculation per direction, since wpx and the span differ.

Inputs of the diaphragm forces

- **Design method:** 12.10.1 and 12.10.2; the alternative of 12.10.3, which 12.10 requires for precast concrete diaphragms in SDC C to F and permits for others it lists; or 12.10.4, for one-story wood panel or bare steel deck diaphragms on rigid walls within the limitations of 12.10.4.1. Which applies is your decision.
- **Seismic parameters:** SDS, SD1, Ie and the Seismic Design Category; Ω0 (Table 12.2-1); whether the structure is braced entirely by wood light-frame shear walls (the collector exceptions); for 12.10.3, Cs and the system (zs); for 12.10.4, the diaphragm (Rdiaph, Tdiaph).
- **Levels, from the top down:** the height, seismic weight w, weight tributary to the diaphragm wpx (0 means w), and the story force Fx from Seismic base shear or your analysis; any transfer force, with the Type 4 box where Ω0 applies to it (12.10.1.1); and Rs for the alternative method (Table 12.10-1, in the table so that a roof and floors can differ). wpx may leave out the walls parallel to the force; the examples do.
- **Diaphragm:** the span L between the vertical elements, the depth B, the length the collector gathers (0 for none) and a reaction factor over w L / 2 for a share your analysis gives, such as torsion on a rigid diaphragm. It defaults to 1.0.

Checks performed

Result	Basis
Fpx, traditional	Eq. 12.10-1, not less than Eq. 12.10-2, need not exceed Eq. 12.10-3
Fpx, alternative	Eq. 12.10-4 and the minimum of Eq. 12.10-5; Cpx from Fig. 12.10-2, Eqs. 12.10-6 to -14; Rs of Table 12.10-1
Fpx, one story	Eqs. 12.10-15, -16a, -16b; Rdiaph and Tdiaph by the diaphragm
Transfer force	Added at the level; with Ω0 at a Type 4 irregularity (12.10.1.1, 12.10.3.3)
Shear, chord	w = Fpx/L; V = w L / 2 × the reaction factor; v = V/B; M = w L²/8; chord = M/B. With 12.10.4: 1.5 v, over the span if L is under 100 ft, else a boundary zone of at least 0.10 L each end (12.10.4.2.2)
Collector, traditional	12.10.2.1: the greatest of the diaphragm's share of Fx (Fx × wpx / w) with Ω0, Eq. 12.10-1 (held to Eq. 12.10-3) with Ω0, and Eq. 12.10-2 without; no Ω0 in SDC B or with the wood light-frame exception
Collector, alternative	12.10.3.4: 1.5 × inertial and transfer forces; a transfer already with Ω0 is not amplified; none in SDC B or with the light-frame exception

Result	Basis
Collector, one story	12.10.4.2.4: Ω0-diaph = 2, not over Rdiaph; not combined with the 1.5 shear amplification

Collector forces are v × length × the amplification, the way the examples work them. Each result reads **Calculated**. The drawing is a plan of the level with the largest unit shear.

Limitations

- A rectangular simple span with uniform force. Openings, reentrant corners, multi-span diaphragms and force distribution to more than two lines need your analysis: the reaction factor is the only way in here.
- Material strength, deflection and drift are not checked. The limitations of 12.10.4.1 and Exception 2 of 12.10.3.4 (frame strength as a cap) are yours, and so is the capacity-limited effect of 11.3. Collector forces are horizontal only: combine them with gravity and vertical seismic effects.
- The story forces are entered. So is the upper-level share where the analysis is modal.
- The 12.10-4 minimum and limits use the wpx of the row. 12.3.3.5 and other irregularity rules are not applied, except Type 4 on transfers.

Verification examples

Verification status: 75 values compared in 11 examples (10 published, 1 hand calculations from the cited equations): 73 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

All published values are from FEMA P-2192-V1 (2020), Chapter 7, written to ASCE 7-22. (PDF pages.)

Example	Pages	Values
One-story wood hall, 7.4, both directions and both methods	267 to 272	Fpx 19,072 and 17,072 lb; limits; 14,647 and 13,111 lb from the equation; ASD 13,350 and 11,950 lb; Cpn 0.576
Six-story steel building, 7.5, traditional	273 to 278	Six levels of Eq. 12.10-1, design forces, roof limits, V, v, collector 76 and 127 kips
Six-story steel building, 7.5, alternative	278 to 283	Γm1, Γm2, Cp0, Cpi, Cs2, Cpn, Cpx and Fpx at each level, roof v, collector 146 kips
RWFD, 7.6, traditional	286 to 290	Fpx 1,712 and 1,459 kips, v 2.37 klf, moment, chord 356 kips, collector 533 kips
RWFD, 7.6, by 12.10.4, both directions	291 to 294	Tdiaph, Cs-diaph, Fpx 1,266 and 1,296 kips, v 1.76 and 2.64 klf, chord 264 kips, collector 317 kips

One documented deviation: at level 6 of the alternative example, P-2192 gets Cpx = 0.60 by dividing by 12 ft, while Fig. 12.10-2 interpolates over the 14.4 ft from 0.8hn to hn. The program gives 0.58 and so about 457 kips where the example has 471 kips.

A hand calculation covers a transfer force with Ω0. No published example here covers a transfer force or the alternative collector rule for it; those rest on the code text.

CHAPTER 204

Rain load

Rain load on a roof with its ponding head, to ASCE 7-22 Chapter 8: R = 5.2 (ds + dh + dp), with all drainage that 8.2 takes as blocked out of service. The hydraulic head comes from the tables of the commentary or your own data, and the ponding head from your analysis or the amplified method of the commentary. ASCE 7-22 puts ponding inside the rain load, so there is no separate ponding check (AISC 360-22 no longer has the old Appendix 2 method). R is unfactored: copy it into the member calculations as load type R.

Inputs of the rain load

- **Storm and roof area:** the risk category, which sets the return period (Table 8.2-1: 100, 200 or 500 years); the rainfall intensity i of the 15 min storm of that return period, from NOAA Atlas 14; and the area served by one secondary outlet A, the tributary roof plus half of any wall that diverts rain onto it.
- **Secondary drainage system:** the static head ds up to the secondary inlet, at the point you are designing (a low point has the largest). The hydraulic head dh is entered from test data or calculations, or found from the flow Q = 0.0104 A i in Table C8.2-1 (overflow drains), Table C8.2-3 (rectangular scuppers), Table C8.2-5 (circular scuppers) or Eq. C8.2-2 (free overflow along a level roof edge, dh negligible). For a drain you may give another dam diameter (weir flow, Eq. C8.2-3) or bowl depth (orifice flow). Whether the secondary system qualifies at all is your input too: how far its inlet stands above the primary inlet, whether it shares drain lines with the primary, and whether it has controlled flow drains.
- **Ponding:** enter dp from a structural analysis of the roof under unfactored rain and dead load, or use the amplified method with D, E, the spans, the spacing, and I of the primary and secondary members. Enter I as the deflection calls for it: the commentary says a joist or truss modeled as a beam is commonly reduced 15 % for shear deformation. It says too that all deformations that add to the roof's deflection should count.
- **Bay with low slope:** whether this bay holds water up to the secondary system (8.2), or holds none: internal (8.3 condition 1) or next to a free-draining edge with secondary members parallel to it (8.3 condition 2); and the roof slope.

Checks performed

Check	Basis
Flow	Q = 0.0104 A i (Eq. C8.2-1)
Hydraulic head	Tables C8.2-1, C8.2-3, C8.2-5, linear between points and scupper widths as their notes allow, Eq. C8.2-3 and the bowl depth rule, kept to 80 % of the table value; Eq. C8.2-2
Past a table	NOT FEASIBLE: their notes say extrapolation is not appropriate. Under a drain table's first flow, its first head
Secondary system	8.2(b): inlet at least 2 in. above the primary; (c) its own drain lines; (d) no controlled flow drains. Otherwise it reads Not adequate: base ds and dh on the next point of discharge
Ponding stability	Eq. C8.2-4: 1.15 Cp + Cs under 1.0; at 1.0 or more there is no solution, which is ponding instability, and no R is given

Check	Basis
Rain load	R = 5.2 (ds + dh + dp) (Eq. 8.2-1). With the amplified method: Bp (D + R without ponding) − D, Eqs. C8.2-4 to C8.2-6, and dp from R
Dead load with rain	1.2D + 1.6R (2.3.1) and D + R (2.4.1), for reference
Low slope	8.3: under 1/4 in. per ft, or under β = (Ls/S + π)/20 next to a free-draining edge, an advisory asks you to investigate the deflected roof

A result reads **Calculated**, or **Not adequate** where the secondary system or the framing fails. The drawing is a section at the drain with ds, dh and dp dimensioned at their true values; it is not to scale along the roof.

Limitations

- The drainage system is not designed. The tables are for one secondary outlet, and assume free outfall at scuppers (backwater raises dh).
- The amplified method is approximate and applies to rectangular bays of primary members carrying equally spaced secondary members, with sinusoidal shapes behind Cp and Cs. Other framing needs an analysis: enter dp. E is one value for both members.
- The method gives one bay's uniform load. Peak loads over a member's span and the 8.3 investigation of a deflected roof are yours.
- Roofs with internal gutters, large adjacent walls, or high roofs draining onto low ones change the critical duration (C8.2): the 15 min rainfall may not hold. 8.4 on draining onto existing roofs is yours to check.
- No member shape lookup: enter I. That keeps this type independent of the member calculations.
- The SI tables (C8.2-2, C8.2-4, C8.2-6) are not used separately: metric input converts to the inch-pound tables, whose values they round.

Verification examples

Verification status: 22 values compared in 12 examples (9 published, 3 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

All published values come from the ASCE 7-22 commentary. Design Guide 40 (rainwater accumulation) is not on disk; it would be the next check.

Example	Source	Values
Ponding in a 40 × 40 ft steel bay	C8.2, p. 672	Cp 0.155, Cs 0.128, Bp 1.44, 41 psf, 59 psf, R 44 psf, factored 88.4 psf
Dam adjustment	C8.2 Example 1, p. 670	dh 2.4 in., total 4.4 in.
Bowl depth under orifice flow	C8.2 Example 2, p. 670	dh 4.0 in., total 6.5 in.
Slope limit	C8.3, p. 674	β 0.76 in. per ft
Table lookups	Tables C8.2-1, C8.2-3, C8.2-5, pp. 668 to 671	Five cells, one interpolated

Hand calculations cover Eq. 8.2-1, Eq. C8.2-1 and the case with no primary members. The example rounds R to 44 psf before factoring and the program carries 44.1 psf, so its 88.6 psf against the printed 88.4 psf is inside the tolerance. Examples 1 and 2 print one decimal, so they are held to half of that step.

CHAPTER 205

Story drift and P-delta

Takes what an elastic analysis gives, level by level, and checks it: the design story drift against the allowable drift, the stability coefficient against its maximum, and the separation from an adjacent structure or a property line. The analysis stays yours. Use it for results from a frame program, or from the Seismic base shear type, whose drift line has no stability coefficient and no separation. Nothing links the types: you copy the values.

Inputs of the story drift

- **Cd, R, Ω0, Ie, risk category and Seismic Design Category:** from Table 12.2-1, Table 1.5-2 and 11.6. R is used only for the Maximum Considered Earthquake Displacement, Ω0 only for the lower limit of β.
- **Structure (Table 12.12-1):** the row of the table, or your own coefficient. The first row needs interior walls, partitions and ceilings designed for the drift, and four stories or fewer: that first condition is your judgment.
- **Single story, interior walls and partitions designed for the drift:** note a of the table, no drift limit. Off by default. It applies only with one story, and you confirm the condition.
- **The system is only moment frames, ρ:** in SDC D to F the limit is divided by ρ (12.12.1.1). ρ is used for nothing else here.
- **The table's column holds:** elastic displacements of the levels, elastic story drifts, or either already amplified by Cd/Ie. Modal programs combine the drifts of each mode (12.9.1): enter those as drifts, not differences of combined displacements.
- **Stories, from the top down:** the name of the level, hsx (the height of the story below it), the displacement or drift, the story shear Vx of the same analysis, the gravity load added at that level (Px accumulates down), β (0 means 1.0, which the code permits as the conservative value), and δdi if you want diaphragm deformation in δDE and δMCE.
- **Type 1 torsional irregularity:** in SDC C to F, enter the largest difference at the edges with diaphragm rotation (12.8.6.5). The box adds a reminder; it cannot change the numbers you enter.
- **The analysis already includes P-delta:** θ is divided by (1 + θ) before the check of Eq. 12.8-19, and the drifts are not multiplied.
- **Separation:** an adjacent structure on the same property, with its δDE2 at the adjacent edge, or a property line. With a distance entered it is checked; at 0 the required distance is only reported.

The results you enter should come from forces with a load factor of 1.0 on Eh and expected gravity loads of at least 1.0D + 0.5L, with L reduced as 12.8.6.1 says. Eq. 12.8-6 need not be considered and the computed period may be used without the Cu Ta limit (12.8.6.2).

Checks performed

Check or result	Basis
Design Earthquake Displacement	Eq. 12.8-16: δDE = Cd δe / Ie + δdi

Check or result	Basis
Maximum Considered Earthquake Displacement	Eq. 12.8-17: δMCE = 1.5 (R δe / Ie + δdi)
Design story drift against Δa	12.8.6.5, 12.12.1, Table 12.12-1, 12.12.1.1. The governing story is the check; every story is listed
Stability coefficient against θmax	Eq. 12.8-18: θ = (Px / hsx) / (Vx / Δxe). Eq. 12.8-19: θmax = 0.5 / (β Cd), at most 0.25, need not be less than 0.10; β not less than 1.25/Ω0
Factor above 0.10	12.8.7: drifts multiplied by 1/(1 − θ) before the drift check; an advisory says to multiply the member forces too, or use a rational analysis
Separation	Eq. 12.12-2: δSS = √(δDE1² + δDE2²); property line setback δDE; 12.12.2

The check is on the drift and on θ, so a passing calculation reads **Adequate**. There is nothing to size: the system is outside the calculation, so there is no auto-design.

Limitations

- No analysis is run. Displacements, shears and loads are taken as entered.
- Torsion, accidental torsion and the torsional amplification Ax are in your entered values, not added here.
- δdi is taken as entered and left out of the story drift, as 12.8.6.5 permits.
- Separation uses the largest δDE of the structure, which is the usual top of the building. The exception of 12.12.2 (a smaller distance by rational analysis) is your call.
- 12.12.3 (members spanning between structures) and 12.12.4 (deformation compatibility) are not checked.
- Nonbuilding structures and Chapter 16 to 18 displacements are out of scope.

Verification examples

Verification status: 103 values compared in 8 examples (8 published, 0 hand calculations from the cited equations): 101 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
12-story concrete frame, north-south	FEMA P-1051 (2015), pp. 454 to 459, Tables 10-11a, 10-12a	θ per story, accumulated load, drift ratio, Δxe, 0.5/(β Cd)
12-story concrete frame, east-west	FEMA P-1051, p. 460, Table 10-12b	θ per story
One-story industrial frame	FEMA P-1051, Example 9.1, pp. 369 to 375	δx 18 in., allowable 10.28 in., θmax 0.125
23-story steel building	FEMA P-2192-V1 (2020), Table 4-7, p. 151	δx, drift ratios both ways, 2 % limit
18-story coupled composite walls	FEMA P-2192-V1, Table 5-5, pp. 197 to 198	Amplified displacements, largest drift ratio 1.65 %

FEMA P-2192 is written to ASCE 7-22. FEMA P-1051 is written to ASCE 7-10, whose θ equation is the same as Eq. 12.8-18 by algebra. Two published values are documented differences: θmax of 0.091 in P-1051 is 0.10 here, because ASCE 7-22 adds that θmax need not be taken below 0.10 (the value 0.5/(β Cd) itself agrees); and the θ of 0.069 printed in Example 9.1 does not follow from its printed numbers, which give 0.058. P-1051 p. 459 prints Cd = 5.5 and then "(6.5)" in the line for θ: only 5.5 reproduces its value and its table, so 5.5 is used.

No published example checks the 1/(1 − θ) factor, the division by (1 + θ), the MCE displacement or the separation: those are hand calculations in the tests.

CHAPTER 206

Seismic site parameters and spectrum

Gives the seismic design values of a site: SDS, SD1, the design spectrum, the Seismic Design Category and, if you ask, the vertical spectrum and the site class from a velocity profile. ASCE 7-22 has no table of site coefficients any more: section 11.4.3 takes SS, S1, SMS, SM1 and the multi-period spectrum from the USGS Seismic Design Geodatabase. So those values are the input here. Copy the results into Seismic base shear or the other seismic types: nothing links them.

Inputs of the spectrum

- **Latitude, longitude, site class and risk category.** The site class comes from your geotechnical report. If the soil is not known well enough, choose **Not known**: the most critical of Site Classes C, CD and D at each period (11.4.2.1).
- **Get values from USGS:** the button asks the USGS web service for that site and fills in everything below, with the date and model in **Source of the values**. The values are then part of your calculation and are saved with it. **The calculation never asks again**, so it works offline and gives the same answer later. If you change the location, risk category or site class afterward, an advisory tells you to get the values again. If the service does not answer, you get a message and can type the values.
- **SS, S1, SMS, SM1, TL, PGAM:** from USGS or typed. S1 sets SDC E or F at 0.75 or more. With SMS or SM1 at 0, the value comes from your table by the rule of 21.4, which is how USGS gets it too (FEMA P-2192). PGAM is carried for reference.
- **Design spectrum: multi-period** (11.4.5.1), or **two-period** (11.4.5.2), which the code permits only where the multi-period values are not available: that is your call.
- **MCER spectrum table:** period and SaM at up to 30 periods. The second and third values are other site classes, for 11.4.2.1 or 20.3: the largest governs at each period.
- **Period of interest T:** gives Sa at that period, with the rules past 10 s and past TL.
- **Vertical spectrum:** the option of 11.9 in place of 12.4.2.2, by region: west of 105° W, Alaska, Hawaii and the territories by 11.9.2; the rest of the conterminous US at 2/3 of the horizontal spectrum.
- **Site class from a velocity profile:** layers with thickness and shear wave velocity. It reports the average over the upper 100 ft and the class. With **velocities estimated**, it also finds the class at 1.3 times and 1/1.3 times the average (20.3). **Extend the last layer** is your judgment on the geology, for data past 50 ft and short of 100 ft. The profile does not change the values: they stay those of the site class you chose, and an advisory says if the two differ.

Checks performed

Result	Basis
SDS, SD1	Eqs. 11.4-1, 11.4-2

Result	Basis
SMS, SM1 from the table	21.4: 90 % of the largest SaM from 0.2 to 5 s; 90 % of the largest T SaM from 1 to 2 s above 1,450 ft/s or 1 to 5 s below, not less than SaM at 1 s
Multi-period spectrum	11.4.5.1: 2/3 of the MCER values, straight lines between them, 10/T and 10 TL/T² past 10 s
Two-period spectrum	11.4.5.2: Eqs. 11.4-3 to 11.4-5, T0 and TS
MCER spectrum	11.4.6: 1.5 times the design spectrum
Seismic Design Category	11.6, Tables 11.6-1 and 11.6-2, the more severe; E or F at S1 of 0.75 or more
Vertical MCER and design spectra	11.9.2, 11.9.3, Table 11.9-1, Eqs. 11.9-1 to 11.9-8
Average shear wave velocity, site class	Eq. 20.4-1, Table 20.2-1, 20.3

The type gives values and reads **Calculated**. Site Class F stops the calculation: it needs a site response analysis (21.1).

Limitations

- USGS is the source of the values. The app checks only that they are numbers in range, and shows the 21.4 result from the table beside SMS and SM1 as a cross-check. USGS rounds what it returns.
- Site-specific procedures (11.4.7, Chapter 21) are not run. A site-specific spectrum can be typed into the table.
- The four conditions under which 11.6 permits Table 11.6-1 alone depend on the structure, so they are checked in Seismic base shear, not here.
- Site Class F soils (20.2.1) are not detected from velocities: that is your geotechnical engineer's call. Near-fault status (11.4.1) is not determined.
- For Site Class not known, Table 11.9-1 has no column of its own; the column of Site Class D is used, the largest of the three.
- Correlations from blow count or strength to velocity are not built in: enter the velocities your geotechnical engineer gives.
- Chart to 5 s; the calculation lists every period.

Verification examples

Verification status: 64 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 61 within tolerance, 3 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Vertical spectrum, Irvine, Site Class D	FEMA P-2192-V1 (2020), Table 3-2, p. 122	SMS 1.63, Cv 1.426, SaMv at 23 periods, Fmd at 13
Most critical of D, CD and DE, velocities estimated	FEMA P-2192-V1, Table 3-3, p. 124	Final SaM at 23 periods; 850, 1,105 and 654 ft/s and their classes

No published example covers one site from SS and S1 through SDS, the multi-period spectrum and the category with the numbers shown, so those steps are hand calculations in the tests. The button is tested against a stand-in for the service, and the shape of its answer was read from a live call on 2026-11-07 (latitude 33.66, longitude -117.82, Site Class D and the default class). One documented difference: Table 3-2 leaves out the lower limit of Eq. 11.9-4 (0.5 SaM / Fmd) at 1, 1.5 and 2 s, where it governs. This type applies it as the code says, so it gives 0.449, 0.313 and 0.232 g against the printed 0.425, 0.242 and 0.155 g. The other 20 periods agree. In the code copy on file, Table 11.9-1 prints "12.0" for the SMS row of 1.0 and Eq. 11.9-8 prints "1 < Tv ≤ 1.0" for 10 s. The example's Cv of 1.426 and its Fmd values confirm 1.0 and 10.

CHAPTER 207

Wind loads, envelope procedure

Pressures on the main wind force resisting system of a low-rise building by ASCE 7-22 Chapter 28, with flat or gable roofs. For the procedure of Chapter 27, components and cladding, or other roofs, use Wind loads. The two types share only the terrain data, so nothing links their results.

Inputs of the envelope procedure

- **Wind speed, exposure, Kd, Kzt, Ke:** Chapter 26. Kd is applied in Eq. 28.3-1. Kzt is entered: the Wind loads type computes it where you need that.
- **Enclosure:** enclosed, partially enclosed or partially open, with both signs of GCpi. Open buildings use 27.4 and 28.3.7, which this type does not cover.
- **Roof:** flat or gable. The building is a rectangle in plan, with the ridge along L.
- **Eave height, roof angle:** h is the eave height for roofs up to 10 degrees, as the figures say, and the mean roof height above that.
- **Parapet height, overhang:** 28.3.4 and 28.3.5.
- **Stories, light-frame, flexible diaphragms:** only for the exceptions to the torsional load cases (28.3.2).
- **Moment frames resist the wind:** turns off the floor of 28.3.3 (the shear with the roof neglected).

Checks performed

Result	Basis
Velocity pressure qh	Eq. 26.10-1, Table 26.10-1
End zone a	Figures 28.3-1 and 28.3-2, with the exception past 300 ft
GCpf of every zone, Load Cases 1 to 4	Figures 28.3-1 and 28.3-2, straight lines between roof angles
Pressure of every zone, both signs of GCpi	Eq. 28.3-1
Negative Zone 2 and 2E, and Zone 3 and 3E past min(0.5 B, 2.5 he)	28.3.2.1
Shear across and along the ridge, torsion, uplift of the whole building	The zones over their areas, for the reference corner
Shear not less than with the roof neglected	28.3.3
Minimum loads, 16 psf on walls and 8 psf on the roof projection	28.3.6
Parapets, overhangs	Eq. 28.3-2, 28.3.5

The type gives loads and reads **Calculated**. There is nothing to size.

The coefficients are pseudo loadings: each corner takes its turn as the windward corner, and frames and bracing take the zone pressures in their own calculations. The totals here are for one reference corner, to check the pressures and to size a system by hand, and do not replace those cases.

Limitations

- Conditions of 28.1.2 are yours to confirm: a regular shape, no across-wind effects, no channeling or buffeting, no arched or unusual roof.
- **Hip roofs are not here.** Step 6 of Table 28.2-1 gives coefficients for flat and gable roofs, and Commentary Figure C28.3-3 only suggests how to adapt them, as a judgment of the committee.
- 28.3.7 (longitudinal loads on open or partially enclosed frames) and open buildings are not covered.
- Monoslope roofs, multiple ridges and non-rectangular plans are not covered.
- 28.3.2.1 is applied across the ridge. In Load Case 2 this type keeps Zone 2 for the whole depth along the ridge, the conservative reading for uplift. The text is not explicit there: **the ASCE Wind Guide (not on hand) should confirm it.**
- Torsion and shear totals add the wall and roof zones by area. In Load Case 4 only the end walls count, as the slopes run along the wind.
- Overturning is not reported: it depends on what resists it. Components and cladding are in Wind loads.

Verification examples

Verification status: 46 values compared in 6 examples (6 published, 0 hand calculations from the cited equations): 46 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

I found **no free published worked example** of Chapter 28. The ASCE guide to the wind provisions of ASCE 7-22 has them (Chapter 4), and it is on the list to buy after wave 7.

Example	Source	Values
Coefficients at flat, 20 degrees and 30 degrees, both figures	ASCE 7-22 Figures 28.3-1, 28.3-2, pp. 356 to 357, read from the page images	Every printed zone: these are the code's numbers, not a worked example

Everything else is a hand calculation from the code's equations, written in the tests: qh, the end zones, interpolation, the rule for Zone 2, shears, torsion, uplift, the floor of 28.3.3, the minimum loads, the torsion exceptions, parapets and overhangs. **What the Wind Guide should confirm:** the totals of Load Cases 1 to 4, and the reading of 28.3.2.1 along the ridge.

CHAPTER 208

Wind on other structures

Wind forces on building appurtenances and other structures to ASCE 7-22 Chapter 29 (directional procedure): solid freestanding walls and signs, rooftop structures and equipment, chimneys, tanks and similar structures, open signs and single-plane open frames, and trussed towers. The forces are strength level (W in the ASCE 7-22 load combinations). Wind on the building itself (MWFRS and components and cladding) is the Wind loads calculation.

Inputs of the wind on other structures

- **Wind speed and site:** V for the risk category, the exposure category, Ke (Table 26.9-1, or 1.0), Kzt (26.8, entered; the Wind loads calculation computes it), Kd from Table 26.6-1 for the structure (0.85 for walls, signs, rooftop equipment, open frames and triangular, square or rectangular towers; 0.90 square, 0.95 hexagonal, 1.0 octagonal and round chimneys and tanks) or entered (the table's note permits 0.95 for round or octagonal structures with nonaxisymmetric systems), and the gust-effect factor: 0.85 for a rigid structure (n1 ≥ 1 Hz; a lower n1 is not feasible with 0.85) or entered (Gf of 26.11.5 for a flexible structure). Rooftop equipment uses (GCr), which includes the gust effect.
- **Solid freestanding wall or sign:** B, s, h (ground to the top), the solidity ratio ε (more than 0.7; with openings the note 1 reduction 1 − (1 − ε)^1.5 is permitted and is an input), a return corner Lr (Case C) and the double-faced sign options of note 2 (thickness t; Rmin ≤ 0.75 reduces Cf of Cases A and B by 1 − 0.133Rmin, Rmax ≤ 0.4 reduces the Case B eccentricity to (0.2 − 0.25Rmax)B).
- **Rooftop structure or equipment:** the building's mean roof height, width B normal to the wind and length L, the unit's vertical and horizontal projected areas Af and Ar and its height. The linear (GCr) reduction from 0.1Bh to Bh (lateral) and 0.1BL to BL (uplift) is permitted and is an input.
- **Chimney, tank or similar:** the cross section (square normal to a face or along the diagonal, hexagonal, octagonal, round), the surface of a round section (moderately smooth, rough, very rough by D'/D), the diameter or least width D, the height h and the base height above the ground (a roof-mounted stack).
- **Open sign or open frame:** B, s, h, ε ≤ 0.7, flat-sided or rounded members with their typical diameter.
- **Trussed tower:** square or triangular, one segment: height, base height, ε of one face and the gross area of one face; rounded members (note 3) and the wind along the diagonal of a square tower (note 4).

Checks performed

Check	Basis
Velocity pressure	qz = 0.00256 Kz Kzt Ke V² (Eq. 26.10-1), Kz by Table 26.10-1 note 1
Signs and walls, Cases A and B	F = qh Kd G Cf As (Eq. 29.3-1), qh at the top of the sign; Cf of Fig. 29.3-1 by B/s and s/h, bilinear (note 4) and held at the table's edges; Case A through the centre, Case B at 0.2B from it (stated as torsion); for s/h = 1 the resultant 0.05h above the centre; Case B need not be considered for B/s ≥ 2
Signs and walls, Case C	B/s ≥ 2: Cf by region from the windward edge, linear between the tabulated B/s along the sign (note 4); the return-corner factor on the asterisked values (Lr/s from 0.3; below 0.3 no reduction is taken); × (1.8 − s/h) for s/h > 0.8 (note 3); region forces, total and resultant

Check	Basis
Rooftop equipment	Fh = qh Kd (GCr) Af and Fv = qh Kd (GCr) Ar (Eqs. 29.4-2, 29.4-3), qh at the building's mean roof height; the commentary's size note for large units
Chimneys and tanks	F = qz Kd G Cf Af (Eq. 29.4-1) summed over height bands of 5 ft or less with qz at each band's centroid; Cf of Fig. 29.4-1 by h/D (linear, note 2), round sections by D √qz; base moment and resultant height
Open signs and frames	Eq. 29.4-1 with Af = ε B s and Cf of Fig. 29.4-2 by ε and the members
Trussed towers	Eq. 29.4-1 with Af = ε Ag, Cf of Fig. 29.4-3, the rounded member factor (≤ 1.0) and the diagonal factor 1 + 0.75ε ≤ 1.2
Conditions	29.1.2 stated: regular shape, no across-wind, vortex shedding, galloping, flutter, channeling or buffeting

The type is an analysis: a result reads **Calculated**. The drawings show the structure's outline with its dimensions and forces: a sign in elevation and in plan for Case A (and Case B where B/s < 2), and Case C as region pressure bands across the sign in plan; a rooftop unit with Fh and Fv; a chimney or tank with the load per foot by height; a tower segment with its force. Supports, posts, anchorage and foundations are not part of the calculation.

Limitations

- Not covered: circular bins, silos and tanks of 29.4.2 (roof and underside pressures, grouped tanks), rooftop and ground-mounted solar panels (29.4.3 to 29.4.5), solid signs attached to walls (29.3.2: use the Wind loads calculation's wall C&C with GCpi = 0), components and cladding of rooftop equipment and screens (30.11), tower guys, appurtenances and ice (Fig. 29.4-3 notes 5 and 6), the gust-effect factor of flexible structures (enter it), across-wind response.
- One tower segment per calculation; a tower with several segments is one calculation per segment.
- Case C between B/s = 10 and 13 interpolates point by point along the sign between the two columns' region values, each column's last region continuing to the sign's end.

Verification examples

Verification status: 47 values compared in 22 examples (17 published, 5 hand calculations from the cited equations): 47 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

No published ASCE 7-22 worked example of Chapter 29 is on file (the ASCE wind loads guide is on the documents list). The force coefficients are checked against the tabulated values of the standard and the forces by hand:

Example	What it checks	Standard / hand calculation	ILX Calcs
F29.3-1-AB	Cf of Cases A and B at (B/s, s/h) = (1, 1), (10, 0.5), (0.5, 0.2), (45, 0.16)	1.45, 1.70, 1.80, 1.95	same
F29.3-1-C5, -C5LR, -C13	Case C regions at B/s = 5 and 13, return corner Lr/s = 1	3.10 / 2.00 / 1.45 / 1.05; 2.325; 4.00 / 2.60 / 2.00 / 1.50 / 1.35 / 0.90 / 0.55	same
F29.4-1	Square h/D = 7, round smooth h/D = 25, hexagonal h/D = 1, round D √qz ≤ 2.5 at h/D = 25; Kd	1.4, 0.7, 1.0, 1.2; 0.90, 1.0, 0.95	same

Example	What it checks	Standard / hand calculation	ILX Calcs
F29.4-2	Flat ε = 0.2 and 0.05; rounded ε = 0.5 low and high D √qz	1.8, 2.0, 1.5, 1.1	same
F29.4-3	Square ε = 0.3 and the diagonal factor; triangle ε = 0.3 with rounded members	2.59, 1.2; 1.414	same
O1	Sign 30 × 10 ft at 25 ft, V = 115 mph, Exposure C	qh 31.98 psf, Cf 1.75, F 12.13 kips at 20 ft, e 6 ft	same
O2	Rooftop unit on a 40 ft building; a large unit with the (GCr) reduction	qh 35.20 psf, Fh 2.729, Fv 2.873 kips; (GCr) 1.85, Fh 33.21 kips	same
O3	Round stack D = 3 ft, h = 15 ft	Cf 0.567, F 0.625 kips, M 4.68 kip-ft	same
O4	Case C, B = 50 ft, s = h = 10 ft (note 3 factor 0.8)	Cf 2.48 at the edge, region forces 5.16 and 3.50 kips, total 14.41 kips	same

CHAPTER 209

Rigid diaphragm torsion

The distribution of a story shear to the walls and frames below a diaphragm that is not flexible, by ASCE/SEI 7-22 12.8.4: the center of mass and the center of rigidity, the inherent and accidental torsion, the torsional irregularity ratio TIR and Type 1 of Table 12.3-1, the amplification Ax, and the direct and torsional shear of every element in both plan directions. It is an analysis: the result reads *Calculated*.

Inputs of the rigid diaphragm torsion

- **Level and story shear:** the plan dimensions Lx and Ly (coordinates are measured from the lower-left corner; the plan edges x = 0, Lx and y = 0, Ly are where the torsional irregularity ratio is taken and 5 % of the dimension perpendicular to the force is the accidental eccentricity) and the strength-level story shears Vx and Vy below the diaphragm (from the seismic base shear calculation; 0 skips a direction). The **rigid diaphragm basis** records how the diaphragm is idealized: a concrete slab or concrete-filled deck meeting 12.3.1.2 (span-to-depth ratio 3 or less, no horizontal irregularity of Types 2 to 5: confirmed by you), or a semirigid diaphragm you idealize as rigid (12.3.2.1.1 and 12.8.4.3 permit the rigid assumption for TIR and Ax).
- **Walls and frames:** each element with its name, the direction of the force it resists (x or y) and its center in plan. Its rigidity is either entered (kip/in, or relative values used consistently) or computed from the wall length L, thickness t, the story height h and the modulus E as a concrete or masonry pier, cantilever $k = E\,t / (4\,(h/L)^3 + 3\,h/L)$ or fixed at both ends $k = E\,t / ((h/L)^3 + 3\,h/L)$ (flexure and shear, G = 0.4 E).
- **Center of mass:** point weights (the diaphragm, walls, equipment at their centroids) and rectangular area pieces of uniform weight (an L- or T-shaped floor), or the centroid of the rectangular plan for a uniform mass.
- **Torsion (code options):** the seismic design category; whether more than 75 % of the story's lateral strength is at or on one side of the center of mass (the second Type 1 condition of Table 12.3-1, from the element strengths: yours); whether accidental torsion is included in the design forces where 12.8.4.2.1 requires it (prefilled) or always; Ax from this level's rigid-diaphragm displacements or from δmax / δavg entered from your building model; and the option that a torsional shear never reduces the direct shear (an office practice, not an ASCE 7 requirement: off by default).

Checks performed

Check	Basis
Rigidity of the elements	Entered, or pier flexure + shear; Σ kx, Σ ky
Center of mass	Σ w x / W, Σ w y / W of the weights and area pieces (or the plan centroid)
Center of rigidity, J	$x_{cr} = \Sigma\,ky\,x / \Sigma\,ky$, $y_{cr} = \Sigma\,kx\,y / \Sigma\,kx$; $J = \Sigma\,kx\,(y - y_{cr})^2 + \Sigma\,ky\,(x - x_{cr})^2$ over the elements of both directions
Torsional irregularity ratio	TIR = Δmax / Δavg (Eq. 12.3-2) at the two plan edges, from the rigid-body drift V/Σk ± (T/J) × distance, with accidental torsion each way and Ax = 1 (12.3.2.1.1); the larger of the two directions

Check	Basis
Torsional irregularity	Table 12.3-1 Type 1: TIR > 1.2 or the 75 % strength condition; the requirements of Table 12.3-1a (12.7.3, 12.5.4, 12.8.6, 12.3.3.5, 12.2.5.4, 12.3.4.2.1, 12.9.1.5) are listed; an advisory, not a failure
Accidental torsion	12.8.4.2.1: in the design forces for SDC B with Type 1 and TIR > 1.4, and SDC C to F with Type 1 (or always, your option); 5 % of the plan dimension perpendicular to the force each way (12.8.4.2.2)
Amplification Ax	Eq. 12.8-15, Ax = (δmax / 1.2 δavg)², 1.0 to 3.0, SDC C to F with Type 1 (12.8.4.3)
Element forces	Direct shear V k / Σk (parallel elements) plus torsional shear T k d / J (every element), T = V (e ± Ax e_a); each element takes the larger of the accidental torsion cases with the sign of its torsional shear kept (C12.8.4.2), and the larger of the two directions
Redundancy	TIR > 1.4 in both directions in SDC D to F: ρ = 1.3 (12.3.4.2.1), an advisory

The drawing (on screen, in the report and as a PDF / DXF sheet) is the level plan with the elements as entered (walls with their length and thickness, elements entered by rigidity as short bars), each tagged with its name and design shear (or its name only when there are more than six; the sheet notes list every shear), the center of mass and the center of rigidity, the story shears on their line of action and the plan dimensions. A plan too large for the smallest plan scale is compressed and drawn not to scale.

Limitations

- One level at a time: the story shear and the rigidities are this level's. Ax by Eq. 12.8-15 uses displacements relative to the base; this level's own rigid-body displacements are exact for a one-story structure and are the level's drift otherwise: enter δmax / δavg from your building model for a multistory structure.
- Elements parallel to x or y only (no skewed walls); no flexible or semirigid diaphragm analysis: the diaphragm is idealized as rigid.
- The 100 % + 30 % orthogonal combination (12.5) is not formed; the forces are per direction. Under concurrent orthogonal loading 12.8.4.2.2 permits the accidental offset in one direction only.
- The 75 % strength condition of Type 1 is your input (element strengths are not part of the calculation).
- The forces are QE (strength level, before ρ and the load combinations); story drift limits (12.12) and the design of the walls, frames, chords and collectors are separate calculations.
- Table 12.3-1a cites 12.3.3.4 for the collector amplification while the body text is 12.3.3.5, and lists 12.8.4.2 under SDC D to F while 12.8.4.2.1 says SDC C to F; the body text is followed (to confirm against the code text: ASCE 7-22 errata).

Verification examples

Verification status: 62 values compared in 10 examples (7 published, 3 hand calculations from the cited equations): 62 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published worked examples (research/notes/05_lateral_rigid_diaphragm_torsion.md):

Example	What it checks	Published	ILX Calcs
YAW-CM-CR	L. Yaw, Rigid Diaphragm Analysis (Walla Walla University, 2022): one-story concrete wall building, 80 × 50 ft	x_cm 40.592, y_cm 26.016 ft; relative rigidities 1.2898, 1.1119, 0.3754; x_cr 37.037, y_cr 38.729 ft; J 4,548.7	same within 0.01 %

Example	What it checks	Published	ILX Calcs
YAW-VX	Force along x, 82.14 k	direct 63.62 / 18.52 k, inherent ∓3.34 / ±10.97 k, accidental 0.66 / 2.16 k, design 60.94 / 22.51 / 13.12 k	same within 0.01 %
YAW-VY	Force along y, 82.14 k	direct 44.11 / 38.03 k, inherent ∓3.07 k, accidental 3.45 / 1.05 k, design 44.50 / 44.55 / 1.98 k	same within 0.01 %
SEAOC-DE33	SEAOC 2018 IBC Seismic Design Manual Vol. 1, Design Example 33 (via the free Tekla Tedds verification page), e + e_a = 24 ft	CR (20, 20) ft, e_a 4 ft, direct 75 / 25 k, totals 52.5 / 47.5 k	same (the code envelope of the ± e_a cases gives wall A 60.0 k)
FEMA-13.2-B	FEMA P-1051 §13.2, 16 equal masonry walls on plank, accidental torsion only	Σd² 11,520, Mta 7.6 V, wall D 0.125 V + 0.0238 V = 0.149 V	same within 0.2 %
FEMA-13.2-D	Same building in SDC D: δmax / δavg at the plan end, Ax	1.402, Ax 1.365, wall D 67.4 kips	1.401, 1.363, 67.2 kips (FEMA rounds the ratio first)
FEMA-11.2	FEMA P-1051 §11.2, precast building, four transverse walls	95 + 14.25 = 109.25 kips	same

Hand calculations: HC-1 (core walls: J = 400, TIR 2.25, Ax capped at 3.0, shears 87.5 / 37.5 k), HC-2 (eccentric center of rigidity: x_cr = 20 ft, J = 3,200, envelope shears 52.125 / 44.625 / 7.3125 k, TIR 1.326) and HC-3 (L-shaped floor: x_cm 35.714, y_cm 18.571 ft, W 280 k), all exact.

CHAPTER 210

Seismic nonstructural components

The horizontal seismic design force Fp on an architectural, mechanical or electrical component and its attachments by ASCE/SEI 7-22 Chapter 13, with the height factor Hf, the structure ductility factor Rμ, the component factors CAR and Rpo, the limits of Eqs. 13.3-2 and 13.3-3, the vertical force, the overstrength force for anchors, the anchor forces of a rigid component on four supports and the relative displacement DpI. It is an analysis: the result reads *Calculated*.

Inputs of the nonstructural component

- **Seismic hazard and structure:** SDS; the seismic design category of the structure the component is in or on (13.1.2; it drives the Table 13.1-1 exemptions); the risk category of the structure (Ie of Table 1.5-2, used in Rμ and DpI); the seismic force-resisting system of the structure (R and Ω0 of Table 12.2-1 for Rμ, *not specified or not listed* for Rμ = 1.3, or entered; with several systems choose the one that gives the lowest Rμ); the average roof height h; and Hf by Eq. 13.3-4 with the approximate period Ta (Ct hn^x of Table 12.8-2 with hn taken as h, or entered: the lowest Ta of the two directions) or by Eq. 13.3-5 where the period is not known.
- **Component:** the row of Table 13.5-1 (architectural) or Table 13.6-1 (mechanical and electrical components, vibration-isolated components, equipment support structures and platforms, distribution system supports and distribution systems) giving CAR (at or below grade plane, or above grade plane by a structure), Rpo and Ω0p, or entered values. For vibration-isolated rows, the restraint air gap over 0.25 in. doubles the design force (Table 13.6-1 note a). The component importance factor Ip (13.1.3: 1.5 for life-safety components such as sprinklers and egress stairs, hazardous contents, or Risk Category IV operation; yours), the operating weight Wp, where it is supported (above grade plane: Hf and Rμ apply; at or below grade plane: Hf = Rμ = 1.0 and the grade CAR), the height of the point of attachment z, and the CAR of a supporting platform (13.3.1.3: the component's CAR is not less than the platform's).
- **Anchorage:** for a rigid component on four support points at the corners of an a × b rectangle, the spacings a and b, the height of the center of gravity above the attachments hcg and the number of anchors per support point.
- **Relative displacement:** none, Eq. 13.3-10 from the height between attachments and the allowable drift ratio of Table 12.12-1, or Dp entered (Eq. 13.3-9 or 13.3-11).

Checks performed

Check	Basis
Height factor Hf	Eq. 13.3-4, Hf = 1 + a1 (z/h) + a2 (z/h)^10 with a1 = 1/Ta ≤ 2.5 and a2 = 1 − (0.4/Ta)² ≥ 0, or Eq. 13.3-5, Hf = 1 + 2.5 (z/h); z/h ≤ 1.0; 1.0 at or below grade plane (13.3.1.1)
Ductility factor Rμ	Eq. 13.3-6, Rμ = (1.1 R / (Ie Ω0))^½ ≥ 1.3; 1.3 where the system is not specified or not listed; 1.0 at or below grade plane (13.3.1.2)
Component factors	CAR and Rpo (13.3.1.3, 13.3.1.4), Ω0p (13.4.2) from Tables 13.5-1 / 13.6-1; CAR not less than the platform's

Check	Basis
Fp	Eq. 13.3-1, Fp = 0.4 SDS Ip Wp (Hf / Rμ)(CAR / Rpo), not more than 1.6 SDS Ip Wp (Eq. 13.3-2) nor less than 0.3 SDS Ip Wp (Eq. 13.3-3); 2Fp for isolated components with an air gap over 0.25 in.
Vertical force	Ev = 0.2 SDS Wp concurrent with Fp (13.3.1.6, Eq. 12.4-4a); Fp is QE with ρ = 1.0 (13.2.2)
Design forces	Eh, Ω0p Eh (13.4.2, where the overstrength is required for anchors in concrete or masonry), 0.7 Eh for ASD (2.4, 13.1.8)
Anchorage forces	Overturning Eh hcg resisted by the support points: force along either side (M/2b, M/2a) or diagonally ((M/2)(cos θ/b + sin θ/a), θ = $\tan^{-1}$(b/a), the FEMA P-2192 method); tension with (0.9 − 0.2 SDS)D (LRFD) or (0.6 − 0.14 SDS)D (ASD), compression with (1.2 + 0.2 SDS)D or (1.0 + 0.14 SDS)D; shear Eh / 4; with and without Ω0p
Relative displacement	Eq. 13.3-10, Dp = (hx − hy) ΔaA / hsx, or entered; DpI = Dp Ie (Eq. 13.3-8)
Exemptions	The Table 13.1-1 row that may exempt the component (SDC A; SDC B; SDC C to F weight, attachment and Ip conditions), named for you to confirm

The drawings show the building elevation (schematic) with the attachment height z and the roof height h, and the component elevation (schematic outline: its size is not an input) with the center of gravity, the design force and the gravity and vertical force, the support spacing, hcg and the anchor forces.

Limitations

- Components of 20 % or more of the combined effective seismic weight (13.1.1, 13.2.9) and Eq. 13.3-7 (accelerations from a nonlinear response history analysis) are outside this calculation.
- The anchor design itself (ACI 318-19 Chapter 17 with the 17.10 seismic requirements, TMS 402 for masonry, ACI 355.2 qualification of post-installed anchors), prying (13.4.3), friction clips and power-actuated fasteners (13.4.5, 13.4.6) are yours; use the concrete anchor calculation for anchors in concrete.
- The anchorage forces assume a rigid component on four support points; flexible components, more support points, isolator snubbers and their supporting frames are not analysed.
- The design of the component, its support structure or platform, distribution system braces and hangers is not part of the calculation; the force on a brace is Fp of the tributary weight entered as Wp.
- The 13.2.9 expression for a combined system is garbled in the text copy on file (to confirm against the code text); it is not used.

Verification examples

Verification status: 77 values compared in 18 examples (15 published, 3 hand calculations from the cited equations): 77 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

FEMA P-2192-V1 (2020 NEHRP Provisions Design Examples, Chapter 8) works its examples with the ASCE 7-22 equations. FEMA prints Hf, Rμ and the coefficients rounded and multiplies with the unrounded values; periods are compared to 0.01 s.

Example	What it checks	Published	ILX Calcs
8.3-PANEL	Precast spandrel panel, steel SMF building, z/h = 40.5/67.5, SDS 1.487 (pp. 339 to 349)	Ta 0.81 s, a1 1.23, a2 0.76, Hf 1.74 (2.50 by Eq. 13.3-5), Rμ 1.71, 0.403 Wp, max 2.379 / min 0.446 Wp, Fp 4,570 lb, Ev 3,047 lb	same
8.3-FASTENERS	Fasteners of the connecting system (CAR 2.8, Ω0p 1.0) (p. 350)	1.129 Wp, Fp 11,568 lb	same
8.4	Egress stair, Ip = Ie = 1.5, Ta of the braced frame and Rμ of the moment frame (pp. 370 to 378)	Ta 0.484 s, Hf 2.03, Rμ 1.40 (1.48 SCBF), 0.582 Wp; flight 410 / landing 298 lb, Ev 141 / 103 lb; fasteners 1.280 Wp, 903 / 657 lb; DpI 2.52 in.	same
8.5-FAN	HVAC fan on the roof, ordinary masonry walls, SDS 0.474 (pp. 383 to 388)	Ta 0.29 s, a1 2.5, a2 0, Hf 3.5, Rμ 1.3, 0.357 Wp, Fp 1,072 lb, Ev 284 lb, Vu 268 lb, bolt force −409 lb (compression)	same
8.5-ISOLATED	Same fan on spring isolators, 2Fp (pp. 390 to 393)	0.864 Wp, Fp = Fp,max 2,275 lb, 2Fp 4,550 lb, tension 448 lb, compression 2,023 lb, Vu 1,138 lb; with Ω0p 1,237 / 1,991 lb	same
8.6	Threaded piping and hot-rolled bracing on a BRBF roof, Ip 1.5 (pp. 405 to 416)	Ta 0.38 s, Hf 3.5, Rμ 1.53, 1.508 Wp (24.7 / 47.8 plf), supports 0.914 Wp, longitudinal brace 2,318 lb	same
8.7	Elevated vessel and its platform, CAR raised to 1.4 (pp. 432 to 444)	Ta 0.353 s, Hf 2.52, Rμ 1.48, 0.762 Wp, Fp 3,808 lb, Ev 1,200 lb; platform 762 / 240 lb	same

FEMA's tension case "LC6" uses 1.2D − Ev, which is not an ASCE 7 combination; the program uses (0.9 − 0.2 SDS)D for uplift, and the LC7 tensions and LC6 compressions match. Hand calculations: a component at grade (Hf = Rμ = 1.0, grade CAR, Fp 4,800 lb), a structure system not specified with Eq. 13.3-5 (Hf 2.25, Rμ 1.3, Fp 775.4 lb) and ASD anchor forces (minimum Fp 1,200 lb, −145 lb, 210 lb), all exact.

CHAPTER 211

Seismic wall anchorage

The anchorage of a concrete, masonry or other structural wall to a flexible or rigid diaphragm by ASCE/SEI 7-22 12.11: the out-of-plane wall force, the anchorage force with ka and its minimums, the optional reduction below the roof, the force per anchor with the 1.4 factor for steel elements, the continuous crosstie force and the subdiaphragm ratio and shear, each checked against the design strength you enter.

Inputs of the wall anchorage

- **Seismic hazard:** SDS, the risk category (Ie of Table 1.5-2) and the seismic design category (SDC A: only the general anchorage of 1.4.4, 0.2 times the wall weight but not less than 5 psf; SDC C to F: the additional requirements of 12.11.2.2 for concrete and masonry walls).
- **Wall:** concrete or masonry, or other (wood, cold-formed steel); its weight Ww (psf); the wall height from the support below to the anchor and from the anchor to the support above (0 at the roof) and the parapet. The weight tributary to the anchor is Ww × (half the height below + half the height above + the parapet) per foot of wall.
- **Diaphragm and anchor level:** flexible (12.3.1.1 or 12.3.1.3) with its span Lf between the vertical elements that support it laterally in the direction considered, or rigid / semirigid (ka = 1.0); the diaphragm material (wood and metal deck have their own rules); the anchor at the roof or below it; and, below the roof of a structure with no flexible diaphragms, the option to apply (1 + 2z/h)/3 with the anchor height z and the roof height h (your choice: C12.11.2 notes that 1.0 may be more appropriate where the wall cantilevers above its highest support).
- **Anchors:** the spacing s, the element checked (a steel element of the anchorage system such as a strap, hold-down or bracket takes the 1.4 factor of 12.11.2.2.2; an anchor bolt or reinforcing bar does not) and the design strength of one anchor (LRFD φRn or ASD Rn/Ω, from the product report, ACI 318 Chapter 17 or the NDS).
- **Crossties and subdiaphragm:** the continuous crosstie spacing (0 = not calculated), whether the crosstie elements are steel (× 1.4), its design strength (0 = force reported only), the subdiaphragm depth perpendicular to the wall (0 = none) and its design unit shear (0 = shear reported only).

Auto-design (editor toolbar) varies the anchor spacing (8 ft down to 1 ft, framing multiples) and, where crossties are calculated, the subdiaphragm depth (8 to 40 ft). Objective: fewest anchors. The anchor strength, the crosstie spacing, the wall and the diaphragm are kept; see Auto-design.

Checks performed

Check	Basis
Out-of-plane wall force	Fp = 0.4 SDS Ie Ww, not less than 0.10 Ww (12.11.1)
Anchorage force	Fp = 0.4 SDS ka Ie Wp (Eq. 12.11-1), ka = 1.0 + Lf/100 ≤ 2.0 at a flexible diaphragm, 1.0 otherwise (Eq. 12.11-2), not less than 0.2 ka Ie Wp nor 5 psf × the tributary area (12.11.2.1); with the reduction (1 + 2z/h)/3, not less than the 12.11.1 force, 0.2 Wp or 5 psf × area; SDC A: 0.2 Wp ≥ 5 psf (1.4.4)

Check	Basis
Anchor	Fp × s (× 1.4 for a steel element in SDC C to F at a concrete or masonry wall, 12.11.2.2.2; × 0.7 for ASD) against the design strength entered
Anchor spacing	Over 4 ft: the wall is designed to bend between the anchors (12.11.2.1), an advisory
Continuous crosstie	Fp × crosstie spacing (× 1.4 steel, × 0.7 ASD) against the strength entered, or reported (12.11.2.2.1)
Subdiaphragm	Length-to-width ratio = crosstie spacing / depth ≤ 2.5 (12.11.2.2.1, SDC C to F, concrete or masonry walls); unit shear Fp (spacing / 2) / depth against the strength entered, or reported
Additional requirements	12.11.2.2.1 to 12.11.2.2.7 named for SDC C to F (wood: no toenails, nails in withdrawal or cross-grain bending; metal deck not the crosstie; embedded straps; eccentric anchorage; pilasters)

The drawings show the wall section at the anchor (heights, parapet, tributary height, the diaphragm, the anchorage force and the anchor spacing and force) and, where crossties are calculated, the partial diaphragm plan along the wall (anchors, continuous crossties, the subdiaphragm with its ratio and shear). The wall thickness is schematic.

Limitations

- The anchor, crosstie and subdiaphragm strengths are entered: the device, its embedment, the splices and the sheathing and nailing are designed elsewhere (the concrete anchor and wood diaphragm calculations, product data).
- The wall spanning between anchors over 4 ft apart, eccentric or skewed anchorage (12.11.2.2.6) and pilasters (12.11.2.2.7) are named, not calculated.
- The forces are strength level (E) with ASD at 0.7 E; the simplified procedure of 12.14.7.5 (no Ie) is not offered.
- Nonstructural walls are designed by Chapter 13 (use the nonstructural components calculation), except the anchorage of concrete or masonry walls to flexible diaphragms, which Table 13.5-1 note b sends back to 12.11.2.

Verification examples

Verification status: 28 values compared in 12 examples (8 published, 4 hand calculations from the cited equations): 27 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

FEMA P-1051 (2015 NEHRP Provisions Design Examples) is written to ASCE 7-10, whose 12.11.1 and Eqs. 12.11-1 / 12.11-2 are unchanged in 7-22. FEMA P-1051 does not apply the 1.4 steel factor to its tie device; the values below are compared before that factor.

Example	What it checks	Published	ILX Calcs
14.2-SIDE	Masonry warehouse, 8 in. CMU side walls (65 psf), wood roof, Lf = 200 ft, SDS 1.43 (pp. 14-47 to 14-51)	ka 3.0 → 2.0, 1.14 Wp, 1.19 klf, 4.76 kips per anchor at 4 ft, crosstie 23.8 kips at 20 ft, subdiaphragm 0.595 klf	same
14.2-END	End walls, 12 in. CMU (103 psf), Lf = 100 ft (pp. 14-53 to 14-55)	1.89 klf, 7.56 kips, crosstie 37.8 kips, minimum subdiaphragm depth 8 ft, 2.36 klf (8 ft) and 1.18 klf (16 ft)	same
11.1-PLANK-D	Five-story masonry, topped plank (rigid), SDC D (p. 11-31)	208 plf, 0.83 kips at 4 ft	same

Example	What it checks	Published	ILX Calcs
11.1-PLANK-C	Untopped plank, SDC C, SDS 0.37 (p. 11-15)	61.6 plf	61.6 plf by Eq. 12.11-1; the 12.11.2.1 minimum 0.2 ka Ie Wp governs at 83.2 plf (a documented deviation)
13.1 / 13.2	Out-of-plane wall forces (pp. 13-10, 13-73, 13-85)	37 psf; 4.3 psf with the 4.5 psf minimum governing; 7.1 psf	same

Hand calculations: the 0.2 ka Ie Wp minimum (83.2 plf), the reduction below the roof with the 12.11.1 floor (factor 0.467, 0.24 klf), a steel strap in ASD (0.7 × 1.4 × 0.48 klf × 6 ft = 2.82 kips) and SDC A (30 plf by the 5 psf minimum), all exact.

CHAPTER 212

Live load reduction

The reduced uniform live load on one member by ASCE/SEI 7-22 4.7, with the roof live load reduction of 4.8: the live load element factor KLL, the tributary area of the floors the member supports, Eq. 4.7-1 with its 0.50 / 0.40 limits, heavy live loads, garages, assembly areas and one-way slabs, and the roof by Eq. 4.8-1 (R1, R2) or as an occupiable roof. It is an analysis: the result reads *Calculated*. The reduced loads are service loads for a member calculation.

Inputs of the live load reduction

- **Live load element factor KLL:** a row of Table 4.7-1 (interior and exterior columns 4, edge columns with cantilever slabs 3, corner columns with cantilever slabs 2, edge beams without cantilever slabs and interior beams 2, all other members 1), or a calculated KLL (the table footnote permits it; Commentary C4.7.1 defines KLL as the influence area over the tributary area).
- **Tributary area per floor and floors supported:** AT is the area per floor times the floors (C4.7.1: areas of a member supporting more than one floor are summed). Enter 0 floors for a member that carries only the roof.
- **Unreduced live load Lo** (Table 4.3-1) and the **use**: reducible, assembly area (4.7.5, not reduced), passenger vehicle garage (4.7.4) or a use Table 4.3-1 marks as not reducible. Live loads over 100 psf are treated by 4.7.3 automatically.
- **Reduce the floor live load:** 4.7.1 permits the reduction; it is your choice. **One-way slab** limits AT to the span times 1.5 times the span (4.7.6); a one-way slab carries one floor.
- **Roof:** an ordinary flat, pitched or curved roof, awning or canopy (4.8.2: Lr = Lo R1 R2, with the reduction on or off), a fabric awning or canopy on a skeleton structure (outside 4.8.2, not reduced), an occupiable roof (4.8.3: a floor-type live load reduced by 4.7, with its use) or a roof for other special purposes (loads approved by the authority having jurisdiction, not reduced). The roof slope F is in inches of rise per foot; for an arch or dome, enter the rise-to-span ratio (F = 32 × rise / span).

Checks performed

Check	Basis
Reduced live load	L = Lo (0.25 + 15/√(KLL AT)) where KLL AT ≥ 400 ft² (Eq. 4.7-1, 4.7.2)
Lower limit	L ≥ 0.50 Lo for one floor, 0.40 Lo for two or more (4.7.2); heavy live loads and garages: not reduced for one floor, at most 20 % for two or more (4.7.3, 4.7.4); assembly not reduced (4.7.5)
One-way slab	AT ≤ 1.5 ℓ² (4.7.6)
Roof live load	Lr = Lo R1 R2, 12 ≤ Lr ≤ 20 psf; R1 = 1, 1.2 − 0.001 At, 0.6; R2 = 1, 1.2 − 0.05 F, 0.6 (Eq. 4.8-1, 4.8.2)
Occupiable roof	Reduced by 4.7 with the roof counted as one more floor of the member and its area added to AT (4.8.3 with C4.8.3 and C4.7.1: our reading)

Check	Basis
Loads on the member	L × n × A and Lr × At (service loads)

The drawings show the tributary plan (the areas as squares of the same area) and a schematic stack of the levels the member supports with their unreduced and reduced loads.

Limitations

- One member and one live load per floor. Members carrying floors of different loads or areas: use the load chasedown.
- Pattern (partial) loading of 4.3.3, concentrated loads, the load combinations and the member design are the member calculation's.
- The occupiable roof as an extra floor of the member is our reading of 4.8.3 with the Commentary.
- No auto-design: nothing is sized.

Verification examples

Verification status: 38 values compared in 23 examples (16 published, 7 hand calculations from the cited equations): 38 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

AISC Design Examples v16.0, Part III, designs a building to ASCE/SEI 7-22; its beams, girders and columns reduce the live load by 4.7 and the roof live load by 4.8.2. All values agree within the printed digits.

Example	What it checks	Published	ILX Calcs
III-26, III-31	Interior beams, AT 450 and 300 ft²	60.0, 69.0 psf	same
III-40	Edge girder with a calculated KLL = 1.98, AT 690 ft²	52.5 psf	same
III-43	Interior girder, AT 1,130 ft²	45.2 psf	same
III-50 to III-53	Columns over three floors (KLL = 4) and the roof	L 33.2, 51.1, 40.2, 35.2 psf, 32.0 psf (0.40 Lo over 30.3); R1 0.6, 0.882, 0.658; Lr 12.0, 20.0, 17.6, 13.2 psf	same

Hand calculations (closed form, no published example on disk): heavy live load over two floors (120 psf) and one floor (not reduced), a garage over three floors (0.8 Lo), an assembly area, a one-way slab with AT capped at 600 ft², an arch roof (F = 8, R1 = R2 = 0.8, Lr = 12.8 psf) and an occupiable roof counted as a floor; Table 4.7-1 values.

CHAPTER 213

Atmospheric ice loads

The ice load from freezing rain on a member, guy, cable, chimney, plate or dome, and the wind on the ice-covered element, by ASCE/SEI 7-22 Chapter 10: the design ice thickness with the height and topographic factors, the ice area or volume and weight, and the wind force on the iced projected area with the Chapter 29 force coefficients. It is an analysis: the result reads *Calculated*. Di and Wi go into the load combinations of 2.3.3 and 2.4.3.

Inputs of the ice loads

- **Nominal ice thickness t and concurrent gust speed Vc:** read from Figures 10.4-2 to 10.4-5 (for the risk category) and 10.5-1, 10.5-2, or the ASCE Hazard Tool (Atmospheric Ice Geodatabase). ASCE 7-22 has no ice importance factor: the risk category is in the map you read (Table 1.5-2 now lists the earthquake factor only; C10.4.2).
- **Source, special icing regions, exclusions:** a site in a special icing region, in complex terrain or where snow or in-cloud icing governs needs a site-specific study (10.1.1); the electric, communication tower and mast structures with their own standards are excluded (10.1.3). Either makes the calculation NOT FEASIBLE until resolved.
- **Height z** of the element (for fz and qz; one segment per calculation for a tall structure, 10.8 step 3), **Kzt** (Eq. 26.8-1), exposure, Ke, Kd and G for the wind (26.6, 26.9 to 26.11).
- **Ice density:** not less than 56 pcf (10.4.1).
- **Element:** a member of an open frame or lattice (round, rectangular with Dc = $\sqrt{(b^2 + d^2)}$, or Dc and the projected width entered from Fig. 10.4-1) with its length and the frame's solidity ratio with ice; a guy or cable; a round chimney or tank (diameter, height); a flat plate (area of one side, vertical or horizontal, the permitted 0.8 / 0.6 reduction off by default); a dome or sphere (radius).

Checks performed

Check	Basis
Height factor	fz = (z/33)^0.10 for 0 < z ≤ 900 ft, 1.4 above (Eq. 10.4-4)
Design ice thickness	td = t fz Kzt^0.35 (Eq. 10.4-5, 10.4.4)
Ice area and weight	Ai = π td (Dc + td) (Eq. 10.4-1), Di = γ Ai; plates and domes Vi = π td As (Eqs. 10.4-2, 10.4-3), × 0.8 / 0.6 for plates where chosen
Ice density	Not less than 56 pcf (10.4.1)
Velocity pressure	qz = 0.00256 Kz Kzt Ke Vc^2 (Eq. 26.10-1) at the concurrent gust
Wind on ice	Projected width + 2 td (10.5); members: Fig. 29.4-2 flat on the bare width, rounded D√qz ≤ 2.5 on the ice and on round members (10.5.3); guys and cables Cf = 1.2 (10.5.5); round chimneys Fig. 29.4-1 round D√qz ≤ 2.5 by h/D (10.5.1); Wi = qz Kd G Cf × width
Combinations	2.3.3 and 2.4.3 with Di and Wi (stated)

The drawings show the iced section (the circle Dc around the section and the radial ice) and, for members and cables, the elevation with the ice weight; for plates and domes a schematic section with the equivalent ice layer.

Limitations

- Wind on iced plates, walls and signs (10.5.2, Fig. 29.3-1 with the iced dimensions) is not computed: use the wind-on-other-structures calculation with the dimensions increased by td. Trussed towers as a whole (10.5.4) are not covered.
- Snow and in-cloud icing loads (site-specific), galloping, ice shedding and aeolian vibration (10.1.2), and the design temperatures of 10.6 are noted, not computed. Partial ice loading (10.7) is the structure's analysis.
- Kd of guys and cables: Table 26.6-1 does not list them; 1.0 is taken (enter another value if you choose).
- Fig. 10.4-1 is a figure in the code: for shapes other than round and rectangular, enter Dc as it shows.
- No auto-design: nothing is sized.

Verification examples

Verification status: 20 values compared in 6 examples (0 published, 6 hand calculations from the cited equations): 20 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

No published ASCE 7-22 Chapter 10 worked example is on disk (the Commentary has none). Every value is a hand calculation (closed form) from the equations of Chapter 10 and 26.10 and the Chapter 29 coefficients: a round pipe in an open frame at 50 ft (fz, td, Ai, Di, the ice weight, qz, Wi), a square tube on a hill at 1,000 ft (fz = 1.4, Kzt^0.35, Dc = √32, flat and rounded coefficients), a guy cable at 33 ft (Cf = 1.2), a round chimney (Cf by h/D), a vertical plate with the 0.8 reduction and a dome. An ASCE ice example is listed among the documents needed.

CHAPTER 214

Simplified seismic design

The seismic base shear and level forces of a simple bearing wall or building frame building by the simplified alternative procedure of ASCE/SEI 7-22 12.14: the eligibility conditions of 12.14.1.1, SDS of 12.14.8.1, the seismic design category, R and the system limitations of Table 12.14-1, V = F SDS W / R, the level forces in proportion to the level weights, story shears and overturning. It is an analysis: the result reads *Calculated*.

Inputs of the simplified seismic design

- **SDS:** entered (2/3 SMS from the USGS values by 11.4.4), or 2/3 Fa Ss with Fa = 1.0 for a rock site (not more than 10 ft of soil between the rock and the bottom of the footing or mat) or 1.4 for a soil site (12.14.8.1). Ss need not be taken above 1.5 (permitted; on by default, yours to clear).
- **S1:** only for the seismic design category: S1 ≥ 0.75 g makes it E (12.14.1.1).
- **Risk category** and **site class:** the procedure needs risk category I or II and a site class other than E or F.
- **System:** a row of Table 12.14-1 (bearing wall systems A.1 to A.18, building frame systems B.1 to B.26) with R and the limitation in SDC B, C and D / E.
- **Levels:** one row per story above grade plane (at most three): height above the base and the seismic weight (12.14.8.1 items 1 to 5: storage, partitions, permanent equipment, snow over 30 psf, roof gardens).
- **Limitations 5 to 12 of 12.14.1.1:** lines of resistance, center of weight, diaphragm overhangs, line spacing for diaphragms that are not flexible, orientation, use in both directions, no offsets, no weak story. Each is a check box you confirm; all are on, as the code requires.

Checks performed

Check	Basis
Eligibility	12.14.1.1 items 1 to 4 checked (risk category, site class, three stories, Table 12.14-1 system); items 5 to 12 as confirmed; any item not met: NOT FEASIBLE
SDS	12.14.8.1 (Fa 1.0 / 1.4, Ss ≤ 1.5) or 11.4.4
Seismic design category	Table 11.6-1 with the simplified SDS; E where S1 ≥ 0.75 g (12.14.1.1)
System limitation	Table 12.14-1: P / NP by SDC, the 40 ft entry of intermediate precast walls (footnote c) and the 35 ft limit in SDC D of light-frame walls of other materials (footnote d)
Base shear	V = F SDS W / R, F = 1.0, 1.1, 1.2 for one, two, three stories (Eq. 12.14-12)
Vertical distribution	Fx = wx V / W (Eq. 12.14-13), story shears (Eq. 12.14-14), overturning (12.14.8.4; foundations may use 75 %)
Load effects	Eh = QE (no redundancy factor), Ev = 0.2 SDS D (zero where SDS ≤ 0.125), Emh = 2.5 QE (12.14.3)
Drift	Not calculated; 1 % of hn where a value is needed (12.14.8.5)

The drawings show the building elevation (story heights to scale, the width schematic) with the level forces, and the story shear and overturning moment diagrams.

Limitations

- One direction per calculation (12.14.1.1 item 10 requires the procedure in both: run each direction).
- The horizontal distribution (12.14.8.3), diaphragm, collector, wall anchorage and connection forces of 12.14.7 and the design of the elements are not part of the calculation.
- Footnote c of Table 12.14-1 (the 40 ft entry of intermediate precast shear walls) is read from the text copy, where the footnote text does not match the entry (to confirm against the code text).
- No auto-design: nothing is sized.

Verification examples

Verification status: 32 values compared in 20 examples (17 published, 3 hand calculations from the cited equations): 32 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

FEMA P-1051 Example 14.1 designs a three-story wood light-frame building by the simplified procedure (ASCE 7-10; the 12.14 equations are the same in 7-22, renumbered). FEMA P-2192 has no 12.14 example.

Example	What it checks	Published	ILX Calcs
P-1051 14.1	SDS 0.89, F 1.2, R 6.5, W 751 kips (pp. 726 to 730)	V 123.4 kips; F roof 30.0, 3rd 46.7, 2nd 46.7 kips	same

Hand calculations (closed form): SDS with Fa = 1.4 (1.2507 g), the Ss cap on a rock site, a two-story building (F = 1.1, base overturning, 1 % drift, Ev), and the R of Table 12.14-1 rows.

CHAPTER 215

Nonbuilding structures (seismic)

The seismic base shear of a nonbuilding structure by ASCE/SEI 7-22 Chapter 15: the system of Table 15.4-1 (similar to buildings) or Table 15.4-2 (not similar) with R, Ω0, Cd and the height limits, the importance factor of 15.4.1.1, the period of 15.4.4, rigid structures (15.4.2) and nonrigid ones by 12.8 with the Chapter 15 minimums, the vertical distribution, and the basics of tanks and vessels. It is an analysis: the result reads *Calculated*.

Inputs of the nonbuilding structure

- **Seismic hazard:** SDS, SD1, S1 and TL (the ASCE Hazard Tool); the risk category by the hazard of the contents and the function (Table 1.5-1), and Ie of a reference document or of Chapter 15 where it is larger (15.4.1.1, for example 1.5 for storage racks open to the public). Ie is the largest.
- **System:** a row of Table 15.4-1 or 15.4-2 (the table is named in the list), or Other with R, Ω0 and Cd entered (a Table 12.2-1 system for a structure similar to buildings, 15.4.1 item 1a). Where the code offers a choice of rows (an ordinary braced frame with its limited height, a permitted height increase or unlimited height), the row is yours.
- **Reference document:** for tanks and vessels designed to AWWA D100, D103, API 650 Appendix E or API 620 Appendix L, and stacks and chimneys designed to ACI 307, the minimums of Eqs. 15.4-3 and 15.4-4 replace 15.4-1 and 15.4-2.
- **Liquid tanks:** a ground-supported liquid tank has an impulsive and a convective component (15.7.2, 15.7.6); the calculation gives the impulsive base shear. Confirm that the weight and period entered are the impulsive ones.
- **Period:** from a substantiated analysis (entered) or Eq. 15.4-6 from level forces fi and the elastic deflections δi they cause. The approximate period formulas of 12.8.2.1 are not used for nonbuilding structures (15.4.4).
- **Cs method:** Method 2 (SDS, SD1) or Method 1 with Sa at T from the multi-period design spectrum (12.8.1.1).
- **Levels:** height above the base and the operating weight with normal contents (15.4.3) of each level; for Eq. 15.4-6 also fi and δi.

Checks performed

Check	Basis
System and Ie	Table 15.4-1 or 15.4-2 (R, Ω0, Cd), Ie by 15.4.1.1
Height limit	Table 15.4-1 / 15.4-2 by SDC; limits a footnote can relax (12.2.5.4, pipe racks) are advisories
Period	Entered or Eq. 15.4-6, $T = 2\pi \sqrt{(\Sigma\, w_i \delta_i^2 / (g \Sigma\, f_i \delta_i))}$
Rigid structure	$T < 0.06$ s: $V = 0.30\, S_{DS}\, W\, I_e$ (Eq. 15.4-5)

Check	Basis
Nonrigid structure	Cs = SDS/(R/Ie) not above SD1/(T R/Ie) or SD1 TL/(T² R/Ie) (Eqs. 12.8-3 to 12.8-5) or Sa/(R/Ie) (Eq. 12.8-2); minimum 0.044 SDS Ie ≥ 0.01 and 0.5 S1/(R/Ie) for structures similar to buildings (12.8), 0.044 SDS Ie ≥ 0.03 and 0.8 S1/(R/Ie) for Table 15.4-2 systems (Eqs. 15.4-1, 15.4-2), 0.044 SDS Ie ≥ 0.01 and 0.5 S1/(R/Ie) with the reference document (Eqs. 15.4-3, 15.4-4); V = Cs W
Vertical distribution	12.8.3 (k by T; k = 1 for rigid structures), shears and overturning
Load effects	Ev = 0.2 SDS D, Emh = Ω0 QE; redundancy and P-delta by 15.4.6; vertical ground motion of 15.1.4 noted

The drawings show a schematic elevation (a frame outline for structures similar to buildings, a line for stacks, vessels and towers) with the level forces, and the shear and overturning moment diagrams.

Limitations

- Nonbuilding structures supported by other structures (15.3: Chapter 13 below 20 % of the combined weight, the combined analysis above), the special provisions of 15.5 and 15.6, and the designs of the reference documents are outside this calculation.
- Liquid tanks: the convective (sloshing) component, freeboard, hydrodynamic pressures and the vertical acceleration of 15.7 and 15.1.4 are not computed.
- Tables 15.4-1 and 15.4-2 are subsets (the cantilever storage rack and reinforced concrete tabletop rows are not listed: use Other). The text copy of ASCE 7-22 loses the R, Ω0, Cd of the distributed mass cantilever row; 3, 2, 2.5 are taken from FEMA P-1051 (to confirm against the code text). The text of 15.4.1 item 2 and 15.4.4 refers to the 12.8 equations by their ASCE 7-16 numbers; the minimums and period formulas meant are applied (to confirm against the code text).
- One direction per calculation. No auto-design: nothing is sized.

Verification examples

Verification status: 67 values compared in 24 examples (19 published, 5 hand calculations from the cited equations): 67 within tolerance, 0 recorded deviations. Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

FEMA P-1051 Chapter 17 works nonbuilding structures to ASCE 7-10; the equations used are the same in ASCE 7-22 (renumbered). FEMA prints Cs to three digits.

Example	What it checks	Published	ILX Calcs
17.2 transverse	Pipe rack, steel OMF of Table 15.4-1, Ie 1.5, T 0.42 s, W 36 kips (pp. 883 to 885)	Cs 0.171, Cs,max 0.184, Cs,min 0.0264, V 6.2 kips	same
17.2 longitudinal	Steel OCBF, T 0.24 s, W 432 kips (pp. 886 to 888)	Cs 0.185, Cs,max 0.346, V 79.9 kips	same
17.4	Boiler building, OCBF with unlimited height (R 1.5), Ie 1.25, T 1.90 s (pp. 894 to 895)	Cs 0.143, Cs,min 0.0366, V 6,907 kips	6,927 kips (FEMA multiplies the rounded Cs; within 0.3 %)
17.7	Skirt-supported vertical vessel, Table 15.4-2 R 3, T 0.762 s, W 300 kips (pp. 908 to 911)	Cs 0.518, Cs,min 0.123 (Eq. 15.4-1), V 155.4 kips, k 1.131, M 10,583 kip-ft	same (the weight entered as 50 levels)

Hand calculations (closed form): a rigid structure (Eq. 15.4-5), Eq. 15.4-2 and Eq. 15.4-4 for a steel stack with S1 = 0.75 g, the 0.03 minimum of Eq. 15.4-1, the Rayleigh period of Eq. 15.4-6, and Table 15.4-1 / 15.4-2 rows.

CHAPTER 216

Modal response spectrum analysis

The modal response spectrum analysis of ASCE/SEI 7-22 12.9.1 on a planar stick model: one weight per level and one shear stiffness per story. The calculation finds the natural periods and mode shapes, the participation factors and the effective modal masses. It combines the modal responses by SRSS or CQC and scales the forces to the ELF base shear (12.9.1.4). It reports the level forces, story shears, overturning moments and drifts, and checks the design story drift against Table 12.12-1. It is an analysis: the result reads *Calculated*.

Inputs of the modal analysis

- **Design spectrum:** the multi-period design spectrum of 11.4.5.1, entered as Sa = 2/3 of the USGS multi-period MCER values at the 22 periods (Seismic site parameters gives them). The sample is FEMA P-2192 Table 3-2 × 2/3, so enter your site's values. The two-period spectrum (11.4.5.2, from SDS, SD1 and TL) is offered for where Exception 2 of 11.4.5 applies.
- **SDS, SD1, S1, TL:** used for the ELF base shear of 12.9.1.4 (Eqs. 12.8-2 to 12.8-7), Cu and the seismic design category.
- **Risk category:** Ie from Table 1.5-2.
- **System:** a Table 12.2-1 system (R, Ω0, Cd, the Ta structure type) or the values entered. The approximate period is Ct hn^x or 0.1N. The redundancy factor ρ is used for Δa/ρ.
- **Stick model:** for each level, its height above the base, its seismic weight wx (12.7.2) and the lateral shear stiffness of the story below it (story shear over story drift, from a model of the structure). Up to 60 levels.
- **Modes:** all modes (100 % of the mass, 12.9.1.1), or the fewest that reach 90 % (the exception). This is your choice.
- **Combination:** CQC (the default) or SRSS (12.9.1.3). For CQC, the modal damping is 5 % by default, the damping of the 11.4 spectra (C12.9.1.3).
- **Allowable drift Δa / hsx:** the Table 12.12-1 coefficient, which you enter.

Checks performed

Check	Basis
Natural modes	K φ = ω² M φ (shear building, lumped masses), periods, Γ, effective modal mass, cumulative participation (12.9.1.1)
Modal responses	Sa(T) of 11.4.5 divided by R/Ie (12.9.1.2); displacements and drifts × Cd/Ie
Combination	SRSS or CQC with ρij = 8ζ²(1 + r) r^1.5 / ((1 − r²)² + 4ζ² r (1 + r)²), each quantity combined on its own (12.9.1.3)
ELF base shear	T = min(T1, Cu Ta) (12.9.1.4.1); Cs by Method 1 (multi-period) or Method 2 (two-period) with Eqs. 12.8-6 and 12.8-7; V = Cs W
Scaling	Forces × V/Vt where Vt < V (12.9.1.4.1); drifts × Cs W/Vt where Eq. 12.8-7 governs and Vt < Cs W (12.9.1.4.2)

Check	Basis
Story drift	Δ = Cd Δe / Ie × the drift scale, against Δa (Table 12.12-1; Δa/ρ for moment frames in SDC D to F, 12.12.1.1)
3-D model	12.9.1.8 requires a 3-D model: stated as an advisory

The drawings show the stick elevation with the combined, scaled level forces, and the story shear and overturning moment diagrams.

Limitations

- **Planar model.** The model is planar, one direction at a time, with a fixed base and shear-building stories. 12.9.1.8 requires a 3-D model, so use this for preliminary design, to check a 3-D model, or where you judge that it represents the structure.
- **Not covered.** Accidental torsion (12.9.1.5), P-delta (12.9.1.6; Story drift takes these results), soil-structure interaction (12.9.1.7), orthogonal effects (12.5), CQC-4, and the residual rigid mode of 12.9.1.1 (a stick model has all its modes).
- **Combined values do not balance.** Level forces, shears and moments are each combined separately, so they are not in equilibrium.
- **No auto-design.** Nothing is sized.

Verification examples

Verification status: 26 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 26 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published stick-model example is on disk. The FEMA P-1051 and P-2192 modal examples are 3-D models from analysis programs. The values are therefore checked against closed forms:

Example	What it checks	Value
Two equal masses and stiffnesses	$\omega^2 = (k/m)(3 \mp \sqrt{5})/2$, Γ1, effective masses 94.72 % and 5.28 %, ρ12, SRSS and CQC base shears, ELF V and V/Vt	T1 1.0166 s, T2 0.3883 s, Vt (CQC) 54.24 kips
Uniform five-story building	$\omega j = 2\sqrt{(k/m)}\sin((2j - 1)\pi/22)$	T1 2.2075 s, T2 0.7563 s, T3 0.4797 s
One story in the plateau	Vt = W SDS/R, $\delta = Cd\ Sa\ g/(R\ \omega^2)$	48.26 kips, 0.2654 in.
S1 = 0.8 g	Cs by Eq. 12.8-7, drift scale Cs W/Vt	1.910

CHAPTER 217

Gust-effect factor

The gust-effect factor of ASCE/SEI 7-22 26.11. The calculation takes the natural frequency (from your analysis, or the approximate value of 26.11.3 where 26.11.2.1 permits it) and classifies the structure as rigid or flexible. It gives G = 0.85 and the computed G for rigid structures, and Gf for flexible ones (n1 < 1 Hz), with the damping ratio you enter. As an option it also gives the along-wind displacement and acceleration at the top by Commentary C26.11. It is an analysis: the result reads *Calculated*. Wind loads (`wind_loads`) covers only rigid buildings: use the factor from here in its place for a flexible one.

Inputs of the gust-effect factor

- **Wind and building:**
  - The basic wind speed V (26.5) and the exposure (26.7).
  - The mean roof height h.
  - The width B normal to the wind and the depth L along it.
- **Natural frequency n1:**
  - From an analysis (26.11.2), with the stiffness and mass expected under the design wind (C26.11).
  - Or the approximate lower-bound na of 26.11.3 for:
    - steel moment frames (Eq. 26.11-2);
    - concrete moment frames (Eq. 26.11-3);
    - other steel or concrete systems (Eq. 26.11-4);
    - concrete or masonry shear walls (Eq. 26.11-5, with Cw entered).
  - The approximate na is permitted only for h ≤ 300 ft and h < 4 Leff (26.11.2.1). Outside those limits the result is NOT FEASIBLE.
  - Low-rise buildings may be taken as rigid (26.11.2).
- **Damping ratio β:** your judgment of the inherent damping under the design wind (C26.11, Structural Damping). The commentary example uses 1 %.
- **Rigid structures:** 0.85 (26.11.1), the computed G (26.11.4), or Gf, which C26.11 permits for every case.
- **Along-wind response (C26.11):**
  - Cfx, the mode shape exponent ξ and the building density (weight per volume).
  - The air density ratio.
  - A serviceability wind speed and averaging time for the acceleration.
  - An acceleration limit, if you want one: ASCE 7 sets none. A limit shows as a recommendation.

Checks performed

Item	Basis
Frequency	26.11.2; Eqs. 26.11-2 to 26.11-5 with the 26.11.2.1 limits (h ≤ 300 ft, h < 4 Leff, Eq. 26.11-1)
Classification	26.2: rigid where n1 ≥ 1 Hz, flexible below; low-rise permitted rigid
Rigid G	z̄ = 0.6h ≥ zmin, Iz, Lz, Q, G = 0.925(1 + 1.7 gQ Iz Q)/(1 + 1.7 gv Iz) (Eqs. 26.11-6 to -9, Table 26.11-1)
Flexible Gf	V̄z, N1, Rn, ηh, ηB, ηL, Rh, RB, RL, R, gR, Gf (Eqs. 26.11-10 to -16)
Along-wind response	Xmax (Eq. C26.11-1), σẍ (C26.11-2, R at the service speed), gẍ (C26.11-5), ẍmax (C26.11-4)

The drawing shows a schematic elevation with the wind profile, the equivalent height 0.6h and the factor.

Limitations

- Along-wind only. Across-wind loading, vortex shedding, galloping, flutter and torsional response are not covered (C26.11): use the literature or wind tunnel tests (26.11.6, Chapter 31).
- One wind direction per calculation, and the fundamental mode only.
- The acceleration limit has no basis in the code. The 1.5 to 2.5 %g in the hint is from knowledge (to confirm against the code text).
- No auto-design: nothing is sized.

Verification examples

Verification status: 28 values compared in 5 examples (2 published, 3 hand calculations from the cited equations): 25 within tolerance, 3 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Example	Published	ILX Calcs
Table C26.11-1 (h 600 ft, Exposure B, n1 0.2 Hz, β 1 %)	Iz 0.201, Lz 710 ft, Q² 0.616, V̄z 135 ft/s, N1 1.053, Rn 0.128, Rh 0.214, RB 0.666, RL 0.343, R² 1.261, gR 3.787, Gf 1.162	same
Table C26.11-1, rigid G	0.860	0.818: Eq. 26.11-6 with the table's own Iz and Q² (recorded deviation C26.11-1-G)
Table C26.11-2	K 0.501, m1 745,400 slug, V̂ 195 ft/s, Xmax 1.76 ft, V̄z 89.1 ft/s, gẍ 3.79	same
Table C26.11-2, accelerations	σẍ 0.176, ẍmax 0.668 ft/s²	about 0.126 and 0.48 ft/s²: the table uses R of the 115 mph case (deviation C26.11-2-SIGMA)

Hand calculations check the approximate frequencies of Eqs. 26.11-2 to 26.11-4.

CHAPTER 218

Impact and machinery loads

The live load with impact of one source, by ASCE/SEI 7-22 4.6 and 4.9. The sources are:

- **Elevators:** the ASME A17.1 pointer of 4.6.2.
- **Machinery:** 20 % or 50 % impact (4.6.3).
- **Hoists for facade access and building maintenance:** 2.5 times the rated load or the stall load (4.6.4).
- **Crane runways:** vertical impact by crane type and service class (4.9.3), and the lateral and longitudinal forces (4.9.4, 4.9.5).

The calculation gives the loads as live loads L with their factored values. It is an analysis: the result reads *Calculated*. Fall-arrest anchorages (4.6.5) are their own type, Fall-protection anchorage.

Inputs of the impact loads

- **Source:**
  - Elevator.
  - Light machinery, shaft- or motor-driven.
  - Reciprocating machinery or power-driven units.
  - Hoist.
  - Powered monorail crane.
  - Bridge crane of service class D to F, or of A to C.
  - Hand-geared crane.

  Assign the bridge crane service class from the actual service (4.9.3.1).
- **Static load P:** depends on the source:
  - elevator: the supplier's static reaction;
  - machinery: the weight of the machinery and moving loads;
  - hoist: the rated load;
  - crane: the maximum wheel load of 4.9.2.
- **Elevator:** whether the reactions already include the A17.1 impact. If they do not, they are doubled (C4.6: A17.1 includes the 100 % impact factor).
- **Larger impact allowance:**
  - Machinery: the manufacturer's percentage where it exceeds the code's (4.6.3).
  - Cranes: a larger value you choose.

  A value below the code's is never used.
- **Hoist:** the stall load.
- **Crane:**
  - The rated capacity and the hoist and trolley weight.
  - Whether the trolley is electrically powered (4.9.4).

- The number of wheels.
- The share of the lateral force taken by the runway considered (4.9.4: by the runways' lateral stiffness, 50 % for equal runways).
- Whether the bridge is hand-geared (no longitudinal force, 4.9.5).

Checks performed

Item	Basis
Elevator	ASME A17.1 through 4.6.2; 100 % impact per C4.6 unless included
Machinery	1.20 P or 1.50 P, or more where the manufacturer specifies (4.6.3)
Hoist	max(2.5 rated load, stall load) (4.6.4; C4.6.4 relates 1.6 × 2.5 to OSHA's 4 × rated load)
Crane vertical	Maximum wheel load × (1 + 25 %, 25 %, 10 % or 0 %) by type and service class (4.9.3)
Crane lateral	0.20 (rated capacity + hoist and trolley), electrically powered trolleys, by the share entered, per wheel (4.9.4)
Crane longitudinal	0.10 × the maximum wheel loads, except hand-geared bridges (4.9.5)
Factored	1.6L (2.3.1) or 1.0L (2.4.1)

The drawing shows a schematic supporting member with the load and, for cranes, the lateral force.

Limitations

- **Elevators.** ASME A17.1 is not on disk. The buffer, safety and rail load cases and the deflection limits are the supplier's and A17.1's, entered as reactions (to confirm against the code text).
- **One source per calculation.** The supporting member, its connections and its deflection are not designed here. The crane runway beam has its own type.
- **Load cases.** The crane forces are given separately. Combining the vertical impact, lateral and longitudinal forces in load cases (AISC Design Guide 7) is the engineer's work.
- **No auto-design.** Nothing is sized.

Verification examples

Verification status: 16 values compared in 9 examples (1 published, 8 hand calculations from the cited equations): 16 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Example	Published	ILX Calcs
AISC DG7 Example 14.1.1 (20 ton crane, 25 % entered)	wheel load with impact 1.25 × 38.1 kips; lateral 2.53 kips per wheel	47.6 kips; 2.53 kips

Hand calculations check:

- The same crane under ASCE 7-22 class B (10 %), with its longitudinal forces.
- Monorail and hand-geared cranes.
- Machinery at 20 %, at 50 % and with a manufacturer's 60 %.

- A hoist governed by its stall load.
- An elevator reaction with 100 % impact.

CHAPTER 219

Fall-protection anchorage

The anchorage of a personal fall arrest system:

- the anchor device;
- the bolts that fix its post or bracket to the host;
- the host member in bending.

It is checked for two forces:

- **ASCE/SEI 7-22 4.6.5:** a live load of 3,100 lb for each attached line, in any direction, factored by the load combinations.
- **OSHA:** 29 CFR 1926.502(d)(15) for construction or 1910.140(c)(13) for general industry. Either 5,000 lb per person attached, or twice the system's maximum arrest force under a qualified person. ANSI/ASSP Z359.6 is offered as an option.

An end anchorage of a horizontal lifeline takes the line tension. Commentary C4.6.5 explains the link between the two: 1.6 × 3,100 lb = 4,960 lb, which matches OSHA's 5,000 lb.

Inputs of the fall-protection anchorage

- **OSHA standard:** construction (1926.502) or general industry (1910.140).
- **Persons attached:** one line each (4.6.5).
- **OSHA basis:** one of
  - 5,000 lb per person;
  - qualified person, a safety factor of 2 on the maximum arrest force;
  - the ANSI/ASSP Z359.6 engineered system, 2 × the maximum arrest force.

  For the last two, enter the maximum arrest force from the system's manufacturer (OSHA limits it to 1,800 lb with a body harness).
- **OSHA strength compared with:** the available strength (φRn or Rn/Ω), as C4.6.5 relates the two rules. This is the default. The other choice is the nominal strength ("without failure"). This is your judgment.
- **System:** a single anchorage, or the end anchorage of a horizontal lifeline. For a lifeline, enter its span and the sag at arrest from the lifeline manufacturer. The arrest load acts at midspan and $T = P / (2 \sin \theta)$.
- **Anchor and connection:**
  - The device's rated (ultimate) strength.
  - The attachment height e above the host.
  - The base bolts: count, grade, diameter, threads.
  - The lever arm from the compression edge to the tension bolts. Half the bolts are in tension.
- **Host member:** a simple span with the anchor at a distance a, its nominal flexural strength Mn (from its own calculation), and the dead load moment at the anchor.

Auto-design varies the bolt count (2 to 8) and diameter (1/2 to 1-1/4 in.) for the least bolt area. The persons, the arrest basis, the device, the height, the lever arm and the host are kept.

Checks performed

Check	Basis
Anchorage forces	3,100 lb per line (4.6.5), × 1.6 (2.3.1) or 1.0 (2.4.1); OSHA force unfactored; lifeline tension P / (2 sin θ)
Anchor device	Rated strength ≥ max(OSHA force, 1.6 × 3,100 lb per line)
Bolt tension	Horizontal pull F e / (nt d) or vertical pull F / n; F'nt by Eq. J3-3a / J3-3b; Rn = F'nt Ab (AISC 360-22 J3.6, J3.7)
Bolt shear	F / n against Fnv Ab (J3.6)
Host bending	1.2 MD + 1.6 M(L) (or MD + M(L)) and MD + M(OSHA) against φMn or Mn/Ω (φ 0.90, Ω 1.67)
Horizontal lifeline	Advisory: OSHA requires a qualified person's design (1926.502(d)(8))

The drawings show the anchor elevation and, for a lifeline, its schematic with the end tension.

Limitations

- **Written from knowledge.** The OSHA sections and ANSI/ASSP Z359.6 are written from knowledge (to confirm against the code text). The regulations are free at osha.gov and are listed in research/DOCUMENTS_NEEDED.md. ASCE 7-22 4.6.5 and C4.6.5 are read.
- **Not designed here.** Prying of the base plate, the plate itself, welds, the post or bracket, fall clearance, swing fall and rescue.
- **Host member.** The host is checked in bending only, as a simple span. Lifeline hosts are designed in their own calculation.

Verification examples

Verification status: 17 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published worked example is on disk. Hand calculations check:

- The 4.6.5 and OSHA forces.
- The factored ASCE force.
- Bolt tension and shear with their AISC strengths, in LRFD and ASD.
- The host moments.
- The qualified-person force for two persons.
- The lifeline angle and tension.

CHAPTER 220

Fluid and hydrostatic pressure

Loads on a rectangular tank or pit:

- the liquid inside as the fluid load F;
- the backfill and groundwater outside as H;
- the uplift under the base slab.

The calculation gives the wall strip moments and shears in the governing load combinations of ASCE/SEI 7-22 2.3.1 (or the 2.3.7 alternative) and 2.4.1. It also checks flotation with the tank empty: the factored uplift against 0.9D (0.6D in ASD), and your safety factor.

Inputs of the fluid pressure

- **Tank or pit:**
  - The inside length and width, the wall height and thickness.
  - The base slab thickness, and its projection past the walls (it engages the soil above against flotation).
  - Open or covered (top slab thickness).
  - The concrete unit weight.
  - Any other permanent dead load that is always present when the tank is empty.
- **Liquid (F):** water (62.4 pcf), seawater (64.0 pcf) or a unit weight you enter, and the maximum depth (2.2: a fluid of well-defined maximum height). The wall top is free or pinned by the top slab.
- **Backfill and groundwater (H):**
  - The backfill height above the base slab.
  - The lateral earth pressure coefficient K, from the geotechnical report (3.2.1; at rest where the wall top is held).
  - The moist and saturated soil unit weights.
  - The highest groundwater below grade, which sets the uplift (3.2.2).
  - Whether the backfill is permanent, so that it may resist the liquid (2.3.1 and 2.4.1: 0.9 or 0.6 where permanent, else 0).
  - The groundwater load factors: H with 1.6 (2.3.1), or the 2.3.7 / 3.3 alternative with the groundwater Hw at 1.0.
- **Flotation:**
  - Whether the soil over the projection counts.
  - A safety factor on the unfactored loads, beside the load combination. This is your criterion; 0 means none.

Auto-design varies the base slab thickness (12 to 48 in.) and the projection (0 to 8 ft) for the least slab concrete. The tank, the liquid, the soil, the groundwater and the factors are kept.

Checks performed

Item	Basis
Liquid pressure	p = γ d, triangular, F (2.2)
Soil and groundwater	K γ z above the water table; K (γ zw + (γsat − γw)(z − zw)) + γw (z − zw) below (3.2.1)
Wall strip	One-way vertical strip fixed at the base, free or pinned at the top (beam analysis), per foot
Combinations (LRFD)	1.4F; 1.4F + 0.9H with H at its minimum (no groundwater) where the backfill is permanent; 1.6H (or 1.6Heb + 1.0Hw by 2.3.7), tank empty
Combinations (ASD)	1.0F; 1.0F + 0.6H; 1.0H (2.4.1)
Uplift	γw × depth of the slab underside below the highest groundwater, over the full area (3.2.2)
Flotation	1.6U (2.3.1) or 1.0U (2.3.7) ≤ 0.9D; ASD U ≤ 0.6D; optional FS U ≤ D
Base slab	Net factored uplift pressure for its design

The drawing shows a section with the liquid, soil and groundwater pressures and the uplift.

Limitations

- **Wall model.** One-way vertical strips only. Plate action of walls with a low aspect ratio (the PCA rectangular tank coefficients), circular tanks, wall reinforcement, crack control, joints and waterstops are not covered: see the liquid-containing tank wall type.
- **Resisting case.** The resisting backfill is taken without groundwater (H at its minimum).
- **Load cases not covered.** Seismic sloshing (ASCE 7 15.7) and the base slab's bending are not covered.
- **Flotation weight.** Only the walls, the slabs, the soil over the projection and the permanent load you enter count. Side friction is not counted.
- **Safety factor.** The usual 1.1 to 1.5 is from knowledge (to confirm against the code text).

Verification examples

Verification status: 17 values compared in 5 examples (0 published, 5 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: the provisions it uses were read in the target edition.

No published flotation or tank-wall example is on disk. Hand calculations check:

- A cantilever wall: $\gamma d^3/6$ and $\gamma d^2/2$, in LRFD and ASD.
- A propped wall: $w0\ L^2/15$, $0.0298\ w0\ L^2$, $w0\ L/10$ and $2\ w0\ L/5$.
- The soil-only case: $K\ \gamma\ H^3/6$.
- The default pit's uplift and dead load, with the flotation demands of 2.3.1, 2.3.7 and 2.4.1.

CHAPTER 221

Continuous beam analysis

A linear elastic analysis of a straight beam of any material: 1 to 6 spans with optional overhangs, pins or fixed supports, line loads, point loads and concentrated moments per load type, the ASCE 7-22 load combinations or one load set, and pattern live load. It gives the shear, moment and deflection envelopes and the reactions; size the member in its material calculation (steel beam, wood beam, concrete beam) with these forces.

Inputs of the beam analysis

- **Stiffness:** an AISC shape (Shapes Database v16.0, Ix about the strong axis) or E and I entered. E = 29,000 ksi is prefilled (steel, AISC 360-22); enter the modulus of your material (wood: the NDS E or Emin you choose; concrete: Ec of ACI 318-19 19.2.2) and the gross, effective or cracked I you judge appropriate. EI is constant along the beam. With a shape, its self-weight can be added as dead load.
- **Spans and supports:** the left support and each span with the support at its right end; a support is a pin (roller), a fixed end, or none (a free end, for example a cantilever: left fixed, one span with no right support). The left and right overhangs are free ends beyond the outer supports (0 = none).
- **Loads:** line loads (uniform, partial or linearly varying), point loads and concentrated moments, each with its load type (D, L, Lr, S, R, W, E) and where it acts: a span, an overhang, or the beam measured from its left end. *From* and *To* are measured from the start of that span or overhang; *From* = *To* = 0 loads all of it. Loads act downward when positive; moments are clockwise positive.
- **Load combinations:** the ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combinations of the load types present, chosen by the method switch (W and E in both directions), or one load set: the unfactored sum of the loads entered.
- **Pattern live load:** ASCE 7-22 4.3.3 partial loading. When on, the live load (L) is placed on all segments (spans and overhangs), on each segment alone, on alternate segments (1st, 3rd, ... and 2nd, 4th, ...) and on each pair of adjacent segments, and every pattern is enveloped with the other loads at full value.
- **Deflection limits:** span / n for live load and for total load; IBC 2021 Table 1604.3 floor values (360, 240) are prefilled and 0 turns a check off. For an overhang or cantilever the span is twice its length.

Results and checks of the beam analysis

Item	Basis
Analysis basis	Euler-Bernoulli beam elements with consistent load vectors (exact for these load shapes); first-order; no shear deformation
Pattern live load	ASCE 7-22 4.3.3; a note states the patterns used, or that pattern loading is off
Reactions and envelope forces	Largest and smallest reaction at each support; largest positive and negative moment and largest shear, with their location and combination
Live load deflection	Each of L, Lr, S and R alone (service, with the live load patterns), against span / n
Total load deflection	The ASD gravity combinations (or the load set), service, against span / n

The drawings (on screen, in the report and as a PDF / DXF drawing sheet) show the beam elevation (spans and overhangs, pin / roller and fixed supports, the loads by load type with their values, span and load-position dimensions, the shape's mark; with E and I entered the member depth is schematic) and, at the elevation's scale and aligned under it, the envelopes of shear, moment (drawn on the tension side) and service deflection over every case analysed, with the largest values of each span labelled. Moments are positive sagging; shears are positive with the left part pushed up. The sheet notes give the stiffness, the analysis basis and the combinations; member design, connections and bearing are not part of the calculation. A long beam on a letter sheet is drawn at 1/16" = 1'-0" at the smallest; a beam much over 65 ft reads best on an 11 × 17 sheet.

Limitations

- Constant EI along the beam; no shear deformation, no second-order effects, no support settlement or springs.
- No member strength checks: the analysis gives forces and deflections only.
- Pattern loading applies to the live load L only (not to Lr, S or R), with the patterns listed above rather than every possible arrangement.
- Ev, the overstrength factor Ω0 and the 0.5L option of ASCE 7-22 2.3.1 are not modeled.

Verification examples

Verification status: 92 values compared in 22 examples (14 published, 8 hand calculations from the cited equations): 92 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Closed-form beam formulas (hand calculation), one unfactored load set, E = 29,000 ksi, I = 1,000 in^4, w = 1 kip/ft, L = 20 ft unless stated. The closed forms below are exact and the app matches them within 0.1 %. The published beam cases of the AISC Steel Construction Manual, 16th Ed., Table 3-22 and the continuous-beam coefficients of Table 3-21c follow.

Example	What it checks	Closed form	ILX Calcs
BA-1	Simple span	$wL^2/8$ = 50.0 kip-ft; $5wL^4/384EI$ = 0.1241 in	50.0; 0.1241
BA-2	Fixed-fixed beam	$wL^2/12$ = 33.33, $wL^2/24$ = 16.67 kip-ft; $wL^4/384EI$ = 0.0248 in	33.33, 16.67; 0.0248
BA-3	Propped cantilever	$wL^2/8$ = 50.0 kip-ft; 3wL/8 = 7.5 kips	50.0; 7.5
BA-4	Two equal spans	$wL^2/8$ = 50.0 kip-ft; 1.25wL = 25.0 kips; $9wL^2/128$ = 28.13 kip-ft	50.0; 25.0; 28.13
BA-5	Two spans, pattern live load	$49wL^2/512$ = 38.28 kip-ft; R = 7wL/16 = 8.75 and −wL/16 = −1.25 kips	38.28; 8.75, −1.25
BA-6	Cantilever, tip load 10 kips, 10 ft	PL = 100 kip-ft; $PL^3/3EI$ = 0.1986 in	100.0; 0.1986
BA-7	20 ft span with a 5 ft overhang	$-wa^2/2$ = −12.5 kip-ft; R = 15.625, 9.375 kips	−12.5; 15.625, 9.375
BA-8	End moment 10 kip-ft (sign convention)	R = ±M0/L = ±0.5 kips; M = +10 kip-ft	±0.5; 10.0

Published: AISC Steel Construction Manual, 16th Ed., Table 3-22 "Shears, Moments, and Deflections" (P = 10 kips, w = 1 kip/ft, l = 20 ft per span, same E and I). The continuous-beam cases print coefficients to three significant figures (deflections to two), which explains the small differences.

Example	Table 3-22 case	Published	ILX Calcs
AISC-7	7, simple beam, load at center (p. 3-217)	M = Pl/4 = 50.0 kip-ft; Δ = Pl³/48EI = 0.0993 in	50.0; 0.0993
AISC-8	8, load at any point, a = 15 ft (p. 3-217)	M = Pab/l = 37.5 kip-ft; R = 2.5 / 7.5 kips; Δmax = 0.0694 in	37.5; 2.5 / 7.5; 0.0694
AISC-9	9, two symmetric loads, a = 5 ft (p. 3-217)	M = Pa = 50.0 kip-ft; Δ = 0.1366 in	50.0; 0.1366
AISC-16	16, fixed both ends, load at center (p. 3-220)	±Pl/8 = 25.0 kip-ft; Δ = Pl³/192EI = 0.0248 in	±25.0; 0.0248
AISC-17	17, fixed both ends, a = 5 ft (p. 3-220)	M1 = −28.13, M2 = −9.38, Ma = 14.06 kip-ft; R1 = 8.44 kips	same
AISC-37	37, three spans, one end span unloaded (p. 3-229)	R = 7.66 / 24.0 / 9.0 / −0.66 kips; MB = −46.8, MC = −13.32, M+ = 29.4 kip-ft; Δ = 0.0563 in	7.67 / 24.0 / 9.0 / −0.67; −46.67, −13.33, 29.39; 0.0563
AISC-39	39, three spans, all loaded (p. 3-229)	R = 8.0 / 22.0 kips; MB = −40.0, M+ = 32.0 / 10.0 kip-ft; Δ = 0.0658 in	8.0 / 22.0; −40.0, 32.0 / 10.0; 0.0656
AISC-41	41, four spans, first and third loaded (p. 3-230)	R = 8.92 / 11.44 / 9.28 / −1.08 kips; MB = −21.44, MC = −14.28, M+ = 39.84 kip-ft; Δ = 0.0925 in	8.93 / 11.43 / 9.29 / −1.07; −21.43, −14.29, 39.86; 0.0924
AISC-42	42, four spans, all loaded (p. 3-230)	R = 7.86 / 22.8 / 18.56 kips; MB = −42.8, MC = −28.56, M+ = 30.88 / 14.56 kip-ft; Δ = 0.0620 in	7.86 / 22.86 / 18.57; −42.86, −28.57, 30.86 / 14.54; 0.0616

AISC Steel Construction Manual, 16th Ed., Table 3-21c (continuous beams, equal spans, equally loaded, p. 3-214), spans of 20 ft, w = 1 kip/ft or P = 10 kips per load point; the coefficients are printed to two or three digits:

Case	Published coefficients	Published	ILX Calcs
Three spans, uniform	−0.10 wL², +0.08 wL², R = 0.4 / 1.1 wL	−40.0, 32.0 kip-ft; 8.0, 22.0 kips	−40.0, 32.0; 8.0, 22.0
Four spans, uniform	−0.107 / −0.071 wL², +0.077 wL², R = 11/28, 26/28 wL	−42.8 / −28.4, 30.8 kip-ft; 7.86, 18.57 kips	−42.86 / −28.57, 30.86; 7.86, 18.57
Two spans, load at each center	−0.188 PL, +0.156 PL, R = 0.31 / 1.38 P	−37.6, 31.2 kip-ft; 3.1, 13.8 kips	−37.5, 31.25; 3.125, 13.75
Three spans, load at each center	−0.15 PL, +0.175 PL, R = 0.35 / 1.15 P	−30.0, 35.0 kip-ft; 3.5, 11.5 kips	−30.0, 35.0; 3.5, 11.5
Two spans, loads at the third points	−0.333 PL, +0.222 PL, R = 0.67 / 2.66 P	−66.6, 44.4 kip-ft; 6.7, 26.6 kips	−66.67, 44.44; 6.67, 26.67

CHAPTER 222

2-D frame analysis

A plane frame of any shape (portal frames, multi-bay frames, gable frames, braced frames, trusses) analysed by the direct stiffness method for each ASCE 7-22 load combination: nodal displacements, support reactions, member end forces and the axial force, shear and moment along every member.

Analysis is an input, and the default is first-order. Saved calculations do not change: a calculation with no choice of analysis runs the first-order path with exactly its old numbers. The other choices add second-order analysis (P-Δ and P-δ), the three stability methods of AISC 360-22 (direct analysis, effective length, first-order analysis) and, with a stability method, the design of steel members to Chapters E, F, G and H. Off, the calculation reads "Calculated"; with every member designed it reads "Adequate" or "Not adequate".

Inputs of the frame analysis

- **Sections:** up to three AISC shapes (Shapes Database v16.0); a member uses its area A and strong-axis Ix. A member can instead use an A and I entered in its row (any material).
- **Nodes:** numbered 1, 2, ... in table order, at (x, y) with x to the right and y up. A node's support is free, fixed, pin, a roller holding Y (on a horizontal surface) or a roller holding X (on a vertical surface).
- **Members:** from node i to node j, the section, E (29,000 ksi for steel, AISC 360-22 B4, prefilled; enter the modulus of your material) and the end conditions: rigid, pinned at i, pinned at j or pinned both ends (a truss member). A node where every member is pinned is a pin joint; it cannot carry a nodal moment.
- **Nodal loads:** Fx (right positive), Fy (up positive) and Mz (counter-clockwise positive) per load type.
- **Member loads:** a uniform load over the whole member or a point load at a distance a from end i, acting by gravity (downward; per unit length of the member for an inclined member), along global X, or perpendicular to the member along its local y axis (90° counter-clockwise from the direction i to j).
- **Load combinations:** the ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combinations of the load types present, chosen by the method switch (W and E in both directions), or one load set: the unfactored sum of the loads.
- **Analysis:**
  - *First-order (no stability method):* the default, as above. Second-order effects are not included, and the calculation says so.
  - *Second-order, nominal stiffness:* P-Δ and P-δ, no imperfections, no stiffness reduction. The forces are second-order but this is not a stability method: AISC 360-22 C1 asks for imperfections and stiffness reductions too.
  - *Direct analysis method (C2):* second-order with 0.8 on all stiffnesses, τb in addition on flexural stiffness, and initial imperfections. K = 1 in member design.
  - *Effective length method (Appendix 7.2):* second-order at nominal stiffness with notional loads; permitted where Δ2nd/Δ1st ≤ 1.5.
  - *First-order analysis method (Appendix 7.3):* first-order at nominal stiffness with the additional lateral load Ni = 2.1α(Δ/L)Yi ≥ 0.0042Yi and B1 on the total member moments; K = 1.

- **Segments per member** (2 to 10, prefilled 4): each member is divided and each segment has its geometric stiffness. Four segments reproduce the Commentary benchmarks. Moments and deflections between segment ends are approximate.
- **Include shear deformation** (on; C2.1(a) and 7.3.2 ask for it): Av = d tw for I-shapes, channels and tees, 2 h t for rectangular HSS, A/2 for round HSS and pipe; for an entered section from **Av** in the member design inputs (blank: none in that member). Off reproduces the bracketed values of the Commentary.
- **Initial imperfections:** notional loads (C2.2b) or a modelled lean (C2.2a), at the **out-of-plumbness ratio** (0.002 = 1/500, the Code of Standard Practice tolerance, C2.2b(c)). A notional load acts wherever a D, L, Lr, S or R load acts, and in the same way along the member: Yi is at the analysis level, so αYi holds for LRFD and ASD. Both directions run in a combination with no lateral load; with one, the notional load takes the direction of its resultant (C2.2b(b)). A combination counted twice shows as "… + Nx" and "… − Nx". **Imperfections applied in:** all combinations (C2.2b(a)), or gravity-only (C2.2b(d), C2.2a), which is checked against Δ2nd/Δ1st ≤ 1.7.
- **Stiffness parameter τb** (direct analysis): iterated per member and per combination from αPr/Pns (Eq. C2-2), or 1.0 with an additional notional load of 0.001 Yi in all combinations (C2.3(c)). An entered section has no Pns, so its τb stays 1.0 and the check says so.
- **Member design** (with a stability method): the **steel of each section** (A992 for W shapes, A500 or A1085 for HSS, A53 for pipe), and the **member design inputs**: your decisions, one row per member, and a member without a row takes the defaults. Lb (blank = member length), Lcx and Lcy (blank = member length, C3; under the effective length method a blank Lcx uses K from the story stiffness), Cb (1.0 is conservative; Eq. F1-1 for your own diagram). Lcz = Lcy. The bracing that defines these lengths is not checked. E must be 29,000 ksi for a designed member.

Auto-design does not run on this type: changing a section changes the stiffness and every force, and sections are shared between members. Size by hand with member design on, or take the forces to a member's own calculation.

Results and checks of the frame analysis

Item	Basis
Equilibrium of the frame	ΣFx, ΣFy and ΣM about the origin of the applied loads and the reactions, for every combination; the largest residual is reported and must be below 10^{-6} of the applied load
Analysis basis	Direct stiffness method: 6 × 6 member stiffness with axial and bending deformation, member loads as fixed-end forces, end releases by static condensation
Second-order effects	First-order: a note that P-Δ and P-δ are not included (AISC 360-22 C1, C2, Appendix 8). Otherwise the analysis below
Second-order analysis	Each member in equal segments, with an elastic and a consistent geometric stiffness (a Timoshenko stiffness when shear deformation is on); axial forces iterated for each combination, never superposed; ASD at 1.6 times the combination, the results divided by 1.6 (C1, C2.1(d))
A frame that buckles or does not converge	NOT FEASIBLE with the reason, and no forces. Buckled: the tangent stiffness is no longer positive definite, including a member between its ends. Not converged: 30 iterations. Also refused: a member chord rotating over 0.10 rad, where the small-displacement basis stops, and αPr ≥ Pns
Amplification	Δ2nd/Δ1st per story, the two runs at the same stiffness, the drift being the columns' average weighted by vertical load (Eq. A-8-7); the elastic critical load factor of the combination with the largest ratio; RM, Pe story and B2 by Eqs. A-8-6 to A-8-8 as a cross-check. A story is the nominally vertical members between two levels; a frame with no such members has no story ratio

Item	Basis
Limits	Direct analysis: ≤ 1.7 when imperfections are in gravity-only combinations (C2.2b(d)), and the recommended ≤ 2.5 (Commentary C1, a flag, not a failure). Effective length: ≤ 1.5 (7.2.1(b)), K = 1 at ≤ 1.1 (7.2.3(b)). First-order method: B2 ≤ 1.5 (read from B2 as Example C.1C does), Eq. A-7-1 on beams with L the larger of span and story height, Eq. A-7-2
Effective length factor (Appendix 7.2)	K of each story's moment-frame columns by the story stiffness approach (Commentary Eq. C-A-7-5), the larger of all combinations, at least 1.0; pinned-both-ends columns and members outside a story 1.0. Enter Lcx to use your own
Compression, per member	Chapter E through the steel library: E3, E4, E7, with Lcx, Lcy and Lcz = Lcy
Tensile yielding, per member	D2(a). Rupture and block shear are not checked (below)
Flexure, strong axis, per member	Chapter F with Lb, Cb: W, S, M, HP with compact webs; channels if compact; rectangular and round HSS, pipe; tees with the lesser strength (stem in compression), since the stem's direction is not an input
Shear, per member	Chapter G, in the plane of the frame
Combined force, per member	H1-1a, H1-1b (H1.2 in tension) for every combination, with the largest axial force and moment along the member in that combination; the largest ratio is reported with its combination. First-order method: M includes B1
Member not designed	An entered A and I; E other than 29,000 ksi; a web or D/t the library's flexure does not cover. Said per member; nothing is checked for it, and the calculation stays "Calculated"
Reactions and member forces	Largest and smallest reaction components at each support, and the axial force, shear and moment of each member, over all combinations
Largest nodal displacement	The largest x or y displacement and its combination

The drawings (on screen, in the report and as a PDF / DXF drawing sheet) show the frame elevation on the member centre lines with node numbers, member marks (M1, M2, ...; the sections are listed by mark in the sheet notes), supports, moment releases (small circles), the nodal and member loads by load type with their values, and the bay and story dimensions; and, for the combination with the largest moment, the moment (on the tension side), shear and axial force (tension +) diagrams drawn on the frame with the largest and smallest value of each member labelled (the units are in the scale line). Uniform loads stacked on a sloping member share one label, listed from the outer band in. The largest displacement is in the design summary; the deformed shape is not drawn. Connections and bases are not part of the calculation. Notional loads are not drawn; the sheet notes state the analysis used. Member moments are positive with tension on the member's local −y side (the bottom of a member drawn left to right). Under ASD a second-order displacement is 1.6 times the combination divided by 1.6: not a service displacement. For serviceability use a first-order or one-load-set run.

Limitations

- First-order analysis (the default): second-order effects, geometric and material nonlinearity are out of scope, and no shear deformation. Choose another analysis for stability design.
- Second-order analysis: elastic and in the plane of the frame only, on the undeformed geometry with a tangent stiffness. Member shortening by its own bending, large displacement and inelastic behaviour are out. Out-of-plane buckling of the frame is not analysed: member design takes Lcy and Lb as your inputs.

- The Commentary recommends checking problems with design load to elastic buckling load near 0.6 to 0.7; AISC benchmark Cases 1 and 2 run up to about 0.4 and 0.65. Above that, Δ2nd/Δ1st passes 2.5 and the check flags it.
- Appendix 8 B1 and B2 are not an analysis choice. The nt and lt split needs a choice of restraints that a general plane frame does not give, and the module has the rigorous answer anyway. B2 is reported as a cross-check, and B1 applies in the first-order analysis method, on the total moment as 7.3.2(b) says.
- Member design covers strong-axis bending and axial force in the plane of the frame. Not checked: connections and bases; tensile rupture and block shear at member ends (D2(b), J4); biaxial bending and out-of-plane loads; bracing strength and stiffness (Appendix 6); concentrated-load limit states (J10); seismic provisions (AISC 341); the Chapter H alternatives H1.3 and H2. Cb under axial tension is not increased.
- The columns of a braced frame with rigid ends get K from the story stiffness formula under the effective length method, which assumes they resist sway, and 7.2.3(a) allows K = 1 there: enter Lcx. RM treats every column with a rigid end as a moment-frame column (0.85 at the lowest), on the safe side.
- A pinned link or leaning column buckles at its own Euler load: give it a real I, not a nearly zero one.
- No rigid end zones, semi-rigid connections, springs, support settlement, temperature, inclined supports or partial uniform loads (use point loads or split the member).
- No pattern live load: enter the live load arrangements as separate load cases or calculations if they govern.
- Auto-design does not apply (see above).

Verification examples

Verification status: 188 values compared in 49 examples (40 published, 9 hand calculations from the cited equations): 164 within tolerance, 24 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Closed-form solutions (hand calculation), one unfactored load set, E = 29,000 ksi. The portal frame is solved by slope-deflection with axially rigid members (A = 10,000 in² in the model); the derivation is in the test file (`backend/tests/test_verify_frame_2d.py`).

Example	What it checks	Closed form	ILX Calcs
FR-1	Fixed-fixed beam, w = 1 kip/ft, 20 ft	$wL^2/12$ = 33.33, $wL^2/24$ = 16.67 kip-ft	33.33, 16.67
FR-2	Propped cantilever	$wL^2/8$ = 50.0 kip-ft; 3wL/8 = 7.5 kips; $9wL^2/128$ = 28.13 kip-ft	50.0; 7.5; 28.13
FR-3	Two equal continuous spans	$wL^2/8$ = 50.0 kip-ft; 1.25wL = 25.0 kips; 0.375wL = 7.5 kips	50.0; 25.0; 7.5
FR-4	Fixed-base portal, h = L = 12 ft, H = 10 kips	base 0.2857Hh = 34.29, top 0.2143Hh = 25.71 kip-ft; $\Delta = Hh^3/(16.8EI)$ = 0.0613 in	34.29, 25.71; 0.0613
FR-5	Three-bar truss, P = 10 kips at the apex	diagonals $-P/(2 \sin \alpha) = -8.33$ kips; chord +6.67 kips	−8.33; +6.67

Published examples (`backend/tests/test_verify_frame_2d.py`): F. Udoeyo, *Structural Analysis* (Temple University Press, 2020, open textbook), Chapters 11 (slope-deflection) and 12 (moment distribution), and AISC Steel Construction Manual, 16th Ed., Table 3-22 beam cases modelled as frames. End moments are compared as magnitudes.

Example	What it checks	Published	ILX Calcs
UD-11.5	Fixed column with 2 kips/ft lateral load, fixed-end beam with 20 kips (Example 11.5)	M = 17.09, 15.83, 14.58 kip-ft; Ax = 10.13, Ay = 10.20, Cy = 9.80, Cx = 9.87 kips	17.08, 15.83, 14.58; 10.13, 10.21, 9.79, 9.88

Example	What it checks	Published	ILX Calcs
UD-12.3	Beam over a column with a propped end, moment distribution (Example 12.3)	M = 12.48, 60.37, 75.31, 14.94, 7.47 kip-ft	12.53, 60.27, 75.36, 15.09*, 7.55*
UD-12.5	Column fixed at both ends with a pinned 65 kN/m beam, metric input (Example 12.5)	M = 13.17, 26.33, 53.39 kN·m	13.30, 26.59, 53.18
AISC-17F	Table 3-22 case 17 (p. 3-220)	M1 = −28.13, M2 = −9.38, Ma = 14.06 kip-ft; R1 = 8.44 kips	same
AISC-39F	Table 3-22 case 39 (p. 3-229)	R = 8.0 / 22.0 kips; MB = −40.0; M+ = 32.0 / 10.0 kip-ft	8.0 / 22.0; −40.0; 31.94 / 10.0

* Documented deviation `UD0EY0-12.3-MD`: the textbook stops the moment distribution after two cycles with distribution factors rounded to 0.01; the app gives the exact elastic solution (1 % apart). In Example 11.5 the textbook writes every slope-deflection equation with the same K = I/L for the 10 ft column and the 6 ft beam; the test gives the column I × 10/6 so both members have that K (with equal I the app gives 16.98 / 16.04 / 14.48 kip-ft). Examples 11.7 and 12.6 of the textbook are not used: their printed moments violate the zero column-top shear of a frame whose beam ends on a roller.

Second-order analysis and the stability methods (`backend/tests/test_verify_frame_2d_stability.py`, LRFD and ASD where the source has both). The solver has its own tests (`backend/tests/test_frame_second_order.py`): it finds the Euler load of a pinned column and a cantilever within 0.2 % and the closed-form amplification.

Example	What it checks	Published	ILX Calcs
Commentary C2 Case 1, Fig. C-C2.2	W14X48, 28 ft, pinned, w = 0.2 kip/ft, P = 0, 150, 300, 450 kips, 4 segments	Mmid 235, 270, 316, 380 kip-in.; Δ 0.202, 0.230, 0.269, 0.322 in. (shear off: 235, 269, 313, 375; 0.197, 0.224, 0.261, 0.311)	within 0.3 %
Commentary C2 Case 2, Fig. C-C2.3	W14X48, 28 ft, cantilever, H = 1 kip, P = 0, 100, 150, 200 kips	Mbase 336, 470, 601, 856 kip-in.; Δ 0.907, 1.34, 1.77, 2.60 in. (shear off: 336, 469, 598, 848; 0.901, 1.33, 1.75, 2.56)	within 0.4 %
DG28 Fig. D-4	W10X60, 15 ft, direct analysis, Pu = 440, H = 2.33 kips	Δ1st 0.580, Δ2nd 2.22 in.; Mu 1,394 kip-in.; H1-1a 1.00	0.583, 2.23; 1,400; 0.996
C.1A (Design Examples)	Direct analysis, a frame with a leaning column	Ni 0.576, 0.614; Δ1st 0.181, 0.193; M 109, 127 and 72.2, 84.8 kip-ft; H 5.52, 6.26 and 3.68, 4.18; P 71.4, 72.6 and 47.6, 48.4 kips; αPr/Pns 0.0760, 0.0811	within 0.4 %
C.1B	Effective length	Δ1st 0.145, 0.155; M 110, 126 and 73.1, 84.1 kip-ft; RM 0.925; Kx 3.45, 3.46	within 0.3 %; Kx within 0.3 %
C.1C	First-order analysis method	Ni 1.21, 0.806; Δ 0.304, 0.203; M 131 and 70.9, 87.1 kip-ft; B2 1.48, 1.54 (over 1.5: not permitted); 0.5 Pns 478; 0.08 Pe 108 kips	within 1 %; reads NG under ASD, as the example says
E.1A, F.1-3, G.1A, D.1	One member each through member design	φPn 893, Pn/Ω 594; Cb φMn 289, 192; φVn 306, 204; φPn 277, 184	within 0.4 %
DG28 Example 3.2	Warehouse moment frame, direct analysis, Comb 7 to 16, drifts Comb 3 to 19, from **our reconstruction** of one line of the building	Pr 91.6, Mr 10,800 kip-in.; Δ2nd 0.890, 4.56 in.	91.1, 10,550; 0.890, 4.52; some values outside 1 % (below)

Hand calculations, marked as such in the test file: the closed-form cantilever and pinned beam-column of DG28 Figs. D-2 and D-3, within 0.05 %; the H1 equations and B1 by hand.

Documented deviations: `C1-DRIFT2`: the second-order drifts printed in C.1A and C.1B are 2.5 to 4 % below ours, with the printed forces matching; the printed 1.41 is below what sway alone gives, and the same solver reproduces the Commentary benchmarks, so the difference seems to be in the printed figure. It is within the Commentary's 5 % for deflections and on the conservative side. `C1C-108`: Figure C.1C-3 prints 108 kip-ft with 5.32 kips over 20 ft, which is 106.4. `C1C-875`: the printed story buckling strength under ASD rounds the story shear. `DG32-MODEL`: the guide does not print its model, so a miss in the warehouse may be our reconstruction: the governing column moment is 2.3 % low, drifts under notional load alone 4 % high, and small moments up to 7 %.

Not verified against a worked example: H1 of a member under a full frame analysis (DG28 Fig. D-4 is the closest), tees and channels in member design, modelled imperfections (checked here against notional loads), Eq. C2-2b with τb below 1.0 in a frame (by hand only).

CHAPTER 223

2-D truss analysis

A plane pin-jointed truss of any shape analysed by the direct stiffness method for each ASCE 7-22 load combination: member axial forces (tension positive), support reactions and joint displacements. It is an analysis tool: its status reads "Calculated", and the members and joints are designed in their own calculations.

Inputs of the truss analysis

- **Sections:** up to three AISC shapes (Shapes Database v16.0, including double angles); a member takes its area A and radius of gyration r from its section. A member can instead use an A (and optionally an r) entered in its row.
- **Joints:** numbered 1, 2, ... in table order, at (x, y) with x to the right and y up, up to 300 joints. A support is free, a pin, a roller holding Y (level surface), a roller holding X (vertical surface) or a roller on a surface at an angle from the horizontal (it holds the direction normal to the surface). A joint no member reaches is ignored.
- **Members:** from joint i to joint j, up to 600 members, the section and E (29,000 ksi for steel, AISC 360-22 B4, prefilled; enter the modulus of your material). Every member is pinned at both ends.
- **Joint loads:** Fx (right positive) and Fy (up positive) by load type, at the joints only.
- **Self-weight of the members:** optional, lumped half to each end joint as dead load, from the shape's weight or, for an entered A, from the unit weight you enter.
- **Effective length factor K:** used only in the slenderness note; 1.0 is prefilled. Choosing it is your judgment.
- **Load combinations:** the ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) combinations of the load types present, chosen by the method switch (W and E in both directions), or one load set: the unfactored sum of the loads.

There is nothing to size, so this type has no auto-design.

Results and checks of the truss analysis

Item	Basis
Determinacy	The count of members and reactions against twice the joints: determinate, indeterminate to a stated degree, or too few (NOT FEASIBLE)
Stability	The free stiffness matrix: a mechanism is NOT FEASIBLE, and its smallest eigenvalue names the joint and the direction that move
Equilibrium of the truss	The applied loads and the reactions, for every combination
Member forces and reactions	The largest tension and compression of each member, and the extremes of each reaction component, over the combinations
Largest joint displacement	The resultant displacement and its combination
Slenderness of compression members	Advisory: KL/r preferably not over 200 (AISC 360-22 E2 user note). Single angles use rz; other shapes the smaller of rx and ry
Slenderness of tension members	Advisory: L/r preferably not over 300 (AISC 360-22 D1 user note; not for rods or hangers)

The drawings (on screen, in the report and as a PDF / DXF drawing sheet) show the truss elevation with joint numbers, member marks, supports, joint loads and chain dimensions, and the member forces of the governing combination on the truss (for example "12.3 T", "8.1 C"). A large truss is drawn in parts across several views.

Limitations

- First-order linear elastic analysis. A truss that is part of a lateral system needs a stability analysis (AISC 360-22 C1, C2).
- Loads act at the joints only. Bending of a chord loaded between joints, joint eccentricity and the secondary moments of real gusseted joints are not included.
- No member design or connection design: use the forces in the member's own calculation.
- No support settlement, temperature, fabrication error or spring supports, although the source textbook treats some.
- Pattern live load is not generated: enter the arrangements as separate calculations if they govern.

Verification examples

Verification status: 67 values compared in 12 examples (12 published, 0 hand calculations from the cited equations): 67 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Compared with published worked examples, one unfactored load set. The test file holds every value and its page.

Source	Example	Compared
McGuire, Gallagher, Ziemian, *Matrix Structural Analysis*, 2nd Ed.	2.1, 3.2, 3.3 (indeterminate), 3.5, 3.6 (indeterminate)	Displacements, reactions and member forces
Udoeyo, *Structural Analysis* (open textbook, 2020)	5.2, 5.3, 8.6, 8.7, 8.11	Member forces and reactions (determinate)
Udoeyo	10.8, 10.9 (force method, indeterminate)	Member forces

For Example 2.1 the book prints the horizontal components of the bar forces at joint a, not the bar forces, so those components are compared.

Udoeyo's book has printing errors, which are left out of the comparison and explained in each example's note: Example 5.2's force BC (5.42 kN printed, 9.17 kN by statics), Example 8.6's force DE (60 kips printed, 45 kips by statics) and its displacement, and the displacements of Examples 8.7 and 8.11. The behaviour tests compare the corrected displacements as hand calculations. Inclined rollers, self-weight lumping, the load combinations and the slenderness notes have no published example; they are checked by hand calculation.

CHAPTER 224

Section properties

The section builder: every geometric property of a cross-section, from a standard shape, a parametric shape, an outline you draw as points, or a section built up from parts of different materials. The tool has no design code and no LRFD / ASD choice; it reads "Calculated", and its values feed any member calculation.

Inputs of the section properties

Shape picks one of four ways to describe the section.

Standard shape. Region, then family, then designation. The list holds only what the server's source setting allows, so a family whose only source is a publisher's or vendor's table may be absent until the owner decides on its use. If the server has no shape database, the list is empty and says so; enter the dimensions as a parametric shape. A calculation saved with a shape the server no longer offers reports Not feasible and says why; nothing is replaced silently.

Parametric shapes. Only the dimensions of the chosen shape are shown.

- *I-shape:* depth d, flange width bf and thickness tf, web thickness tw, root fillet radius, toe radius. *Equal flanges* off gives a different bottom flange. *Sloped flanges* on gives the flange slope (rise over run: 0.1667 is 1:6, AISC S; 0.14 is 14 %, IPN) and where tf is measured, as a fraction of the outstand from the tip (0.5 is mid-outstand). For a rolled W-shape the fillet radius is kdes − tf from the AISC Shapes Database.
- *Channel:* the same, on one side of the web.
- *T-shape:* the same, plus a stem toe radius and, when the flange is sloped, a taper of the stem faces.
- *Angle:* vertical leg h, horizontal leg b, thickness t (and a different thickness of the vertical leg if you need one), root radius, and toes with a radius or a 45 degree bevel.
- *Hollow rectangle (RHS, HSS):* width, height, wall thickness, outside corner radius (0 is sharp; the AISC Manual HSS tables use twice the design thickness, and the design thickness is 0.93 times the nominal for ERW HSS, AISC 360-22 B4.2), and the inside radius, by default the outside radius less the wall.
- *Hollow circle (CHS, pipe):* outside diameter D and wall t.
- *Rectangle:* b and h, with square corners, a corner radius or a corner bevel. *Round bar:* D.
- *Lipped C, track, Z with lips, hat:* depth, width, thickness and inside bend radius, all out to out; the lip (a Z's may be at another angle and its lower flange another width); the hat's outstanding flange.
- *Rail:* overall depth, head width and depth, web thickness, base width, base depth at the web and at the tips, and two radii. Straight flanks: a published rail profile with head arcs goes in as a custom outline.

Custom outline. A table of points in order around the outline: x, y, a corner radius r (a tangent arc at that point, at any angle) and a bulge (the side to the next point is an arc; bulge = tan(sweep / 4), the DXF convention; positive is counterclockwise; two points with a bulge of 1 each make a circle). A point with a radius cannot touch a curved side. The *Loop* column separates outlines: the lowest number is the outline, each other number a hole. Up to 200 points and 20 loops.

Built up from parts. A table of parts, each a plate, a parametric shape or a named database shape (W14X90, or EU-IPE:IPE 200 to name the family too). The columns d, b, tw, tf, r and lip hold whatever the kind uses: a plate is b wide and d high; an angle has its thickness in tw; an RHS its outside radius in r; a hat its flange in lip. Each part is mirrored about its own y axis if asked, rotated, then moved by dx and dy from its own origin: plate, I, tube and bar at the center; channel and C at the back of the web at mid-depth; angle at the heel; tee at the flange top; Z on the web; hat at the flange's centre line. Parts may touch, which welds them into one section, but may not overlap. Up to 30 parts. Each part has its own E, Fy and unit weight.

Built up from rectangles. The older table of solid rectangles and holes, each by width, height and center; holes lie inside one solid rectangle.

Material. The unit weight for the weight per length (steel 490 pcf, concrete 150 pcf). Poisson's ratio for the shear areas only.

Results and checks of the section properties

The x axis is horizontal and the y axis vertical, both through the centroid; the centroid is measured from the left and bottom edges of the bounding box. The step-by-step text names the method of each property:

Result	Method	Does it depend on the mesh?
A, $\bar{x}$, $\bar{y}$, Ix, Iy, Ixy, I1, I2, θ	Exact: closed-form integrals around the outline over lines and true arcs (Green's theorem). No chords, no numerical integration	No
S top, bottom, left, right, and S1, S2 about the principal axes	Exact: I / c to the extreme fibre	No
r about x, y and the principal axes	Exact: √(I / A)	No
Zx, Zy, Z1, Z2 and the plastic axes	Closed form for a given axis; the axis is the root of the area balance to 1e-10 of the size	No
Weight per length, surface area per length	Exact: area times unit weight; the length of the boundary, holes included. Faces shared by touching parts are removed, which alone does use the mesh's boundary	Only for touching parts
J	Finite elements on the true outline (quadratic triangles, Saint-Venant warping, Pilkey 2002): fillets, tapers, toes and closed cells are all in it	**Yes**
Cw	Finite elements: the warping function about the shear center	**Yes**
Shear center, xo, yo, ro, H	Finite elements (Trefftz's definition: warping orthogonal to x and y); ro and H in the form of AISC 360-22 Eq. E4-9, E4-8	**Yes**
Shear deformation areas Asx, Asy	Finite elements: the energy of the elastic shear stress solution. They depend on Poisson's ratio	**Yes**
J thin-walled, Cw thin-walled	Classical thin-walled theory on the centre line, the method behind the AISC tables; fillets ignored. Shown whenever they differ from the finite element value	No
ho, rts, Wno, Sw, Qf, Qw, eo, C	The family's own formulas on the centre line (I, channel, RHS)	No
Mesh check	The area and Ix of the mesh against the exact values, with the element and node counts. Beyond 0.5 % the mesh is not trusted and the result is Not feasible	

Result	Method	Does it depend on the mesh?
Section dimensions	The outline closes, does not cross itself, holes are inside and do not overlap, parts do not overlap; otherwise Not feasible with the reason and a hint	

Both bases. The finite element J and Cw are the true values of the outline. AISC's Cw, ro and H, and the equations of AISC 360 that use them, rest on the thin-walled values: a W14X90 has Cw = 15,830 $in.^6$ on the true outline and 15,940 $in.^6$ thin-walled (the table says 16,000); a C10X20 has 44.2 $in.^6$ and 56.7 $in.^6$ (the table says 56.9). Use the basis your equation assumes. The J of an HSS is tabulated by Bredt's thin-walled formula (HSS8X4X1/2: 61.1 $in.^4$), a little under the true J of the solid wall (61.9 $in.^4$).

The circle's shear area. At a Poisson's ratio of 0.3 a solid circle has Asy = 0.851 A, the energy value $6(1+v)^2/(7+14v+8v^2)$ of the elastic solution. It is not Cowper's 0.886, a different definition; compare like with like.

Several materials. The elastic values are transformed by E, and the plastic values by Fy, to the first part's material. A is then the transformed area; A gross is shown as well, and the weight is each part's own. Parts that do not touch have J = the sum of the pieces and no shear center or Cw of their own, and the result says so.

A standard shape shows its family, standard, source and source kind, its source flag, and how the family compares with published tables, when that record is on hand. The numbers are calculated here, from the row's nominal dimensions, by the same engine as every other option.

The drawing (on screen, in the report and on the PDF / DXF drawing sheet) shows the true outline to scale, arcs as fine chords of at most 3 degrees, with the holes and the parts, the centroid, the shear center when it is apart from the centroid, the centroidal x and y axes, the principal axes 1 and 2 when Ixy is not zero, the overall dimensions with the centroid located from the left and bottom edges, and the callouts of the shape. Parts are numbered up to 8 where a leader can reach them; the input summary lists every part.

Limitations

- **Exact properties:** none on the shape beyond what can be drawn: straight sides and circular arcs. An ellipse or a spline goes in as a series of arcs.
- **Finite element properties:** the mesh has a vertex limit, and the size of the elements has a floor of 1/5,000 of the section's diagonal. The density falls for thin walls, and the mesh check states the density used: full density up to a size-to-wall ratio of 120, half to 500, a quarter beyond, which kept the time to about 12 s in the worst outlines tried (a tube of 1,000 in. diameter and 0.01 in. wall). Walls too thin for the mesh (around one millionth of the size) are Not feasible, never an approximate number.
- **Thin-walled values** ignore fillets and tapers; Cw of a tee or an angle is its own small formula; a custom outline, a built-up section, a rail or a solid has none.
- **Family constants** are given for equal-flange I, channels, tees, equal-thickness angles, RHS and CHS. An unequal-flange I and cold-formed shapes give J and Cw thin-walled only.
- **Toe radii** larger than fit are reduced to the largest that fits, and the result says so.
- **Gross section only.** No effective (reduced) section for slender elements, no bolt holes unless you draw them, no residual stress.
- **Plastic moduli** assume the whole section yields, each material at its Fy.
- **Database rows:** where a table gives no toe radius, the row's radius is one constant per family, chosen against published values and stated with the source.
- **Tables:** 200 points, 20 loops, 30 parts, 30 rectangles, and at most 4,000 chords after the curves are divided.

- **Older calculations:** every older input still loads and calculates, on this engine. J is now the true J: it no longer ignores fillets or sums rectangles, so saved J values change. Closed-cell built-up sections now give the torsion stiffness of a closed cell.

Verification examples

Verification status: 138 values compared in 33 examples (21 published, 12 hand calculations from the cited equations): 129 within tolerance, 9 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Closed-form hand calculations and the AISC Shapes Database v16.0 (tests/test_verify_section_properties.py). Tolerance 1 %.

Example	What it checks	Expected	ILX Calcs
SP-1	Rectangle 4 × 12 in	A = 48 in², Ix = 576 in⁴, Zx = 144 in³, J = 202.3 in⁴	48.0, 576.0, 144.0, 202.3
SP-2	Circle D = 10 in	I = 490.87 in⁴, Z = 166.67 in³	490.87, 166.67
SP-3	Tube 10 × 0.5 in	I = 168.81 in⁴, Z = 45.17 in³	168.81, 45.17
SP-4	Plate I-shape (W14X90 dimensions, no fillets)	A = 26.13 in², Ix = 983.0 in⁴, Zx = 154.2 in³	26.13, 983.0, 154.2
SP-5	T-shape 6 × 8 in	ȳ = 5.75 in, Ix = 38.13 in⁴, Zx = 12.0 in³	5.75, 38.13, 12.0
SP-6	Angle 4 × 4 × 1/2 in (no fillet)	Ix = 5.562 in⁴, Ixy = −3.267 in⁴, I2 = 2.295 in⁴	5.562, −3.267, 2.295
SP-7	W14X90 from plates, fillet kdes − tf	AISC: A = 26.5, Ix = 999, Iy = 362, Sx = 143, Zx = 157, Zy = 75.6	26.43, 994.8, 360.9, 142.1, 156.1, 75.4
SP-8	HSS8X4X1/2 (tdes = 0.465 in, corner radius 2 tdes)	AISC: A = 9.74, Ix = 71.8, Iy = 23.6, Zx = 23.5, J = 61.1 (Bredt)	9.74, 71.7, 23.6, 23.5, 61.1
SP-20 to 22	J of rectangles, Timoshenko: 4 × 4, 4 × 8, 1 × 10 in	36.0, 117.1, 3.12 in⁴	35.99, 117.1, 3.12
SP-23	Circle: J and the shear area at a Poisson's ratio of 0.3	J = 981.7 in⁴, Asy = 0.851 A = 66.8 in²	981.7, 66.8
SP-24	Ring D = 10, t = 1 in	A = 28.27, I = 289.8, Z = 81.33, J = 579.6	28.27, 289.8, 81.33, 579.6
SP-25	Channel d = 10, bf = 4, tf = tw = 0.25 in, thin-walled closed forms	J = 0.0911 in⁴, Cw = 108.7 in⁶, e = 1.365 in	0.0911, 108.7, 1.365

Published section properties of the AISC Shapes Database v16.0, each shape built from its tabulated dimensions. The J of a rolled shape now comes from the true outline; the HSS rows compare the thin-walled value, AISC's basis:

Example	Shape	AISC Shapes Database	ILX Calcs
SP-9	W24X68	A = 20.1, Ix = 1830, Iy = 70.4, Sx = 154, Zx = 177, Zy = 24.5, J = 1.87	20.06, 1825, 70.5, 154.0, 176.4, 24.6, 1.875
SP-10	W8X31	A = 9.13, Ix = 110, Iy = 37.1, Sx = 27.5, Zx = 30.4, Zy = 14.1	9.13, 109.9, 37.1, 27.5, 30.4, 14.1

Example	Shape	AISC Shapes Database	ILX Calcs
SP-11	W36X150	A = 44.3, Ix = 9040, Iy = 270, Sx = 504, Zx = 581, Zy = 70.9	44.3, 9083, 271.5, 506, 583, 71.2
SP-12	HSS6X6X3/8	A = 7.58, Ix = 39.5, Sx = 13.2, Zx = 15.8, J = 64.6	7.58, 39.45, 13.15, 15.82, 64.6
SP-13	HSS12X8X5/8	A = 21.0, Ix = 397, Iy = 210, Sx = 66.1, Zx = 82.1, J = 454	21.02, 396.4, 210.1, 66.07, 82.08, 453.9
SP-14	HSS10.000X0.500	A = 13.9, I = 159, S = 31.7, Z = 42.3, J = 317	13.93, 158.7, 31.73, 42.31, 317.3
SP-15	HSS6.625X0.280	A = 5.20, I = 26.4, S = 7.96, Z = 10.5, J = 52.7	5.20, 26.37, 7.96, 10.54, 52.75
SP-16	WT7X45	A = 13.2, Ix = 36.5, Iy = 181, Zx = 11.5, y = 1.09 (from the flange)	13.22, 36.44, 180.4, 11.47, 1.096
SP-17	C10X20, sloped flanges	A = 5.87, Ix = 78.9, Iy = 2.80, Sx = 15.8, Zx = 19.4, Zy = 2.70, J = 0.368, ho = 9.56, rts = 0.894, Wno = 8.25, eo = 0.636, Qf = 4.92, Qw = 9.66	5.87, 78.78, 2.80, 15.76, 19.38, 2.70, 0.368, 9.56, 0.894, 8.25, 0.637, 4.92, 9.65
SP-18	L4X4X1/2, toe radius r / 2	A = 3.75, Ix = 5.52, y = 1.18, J = 0.322, Cw = 0.366, Iz = 2.25	3.77, 5.51, 1.17, 0.321, 0.366, 2.28*
SP-19	W14X90	A = 26.5, Ix = 999, Iy = 362, Sx = 143, Zx = 157, Zy = 75.6, J = 4.06, Cw = 16,000, ho = 13.3, rts = 4.10, Wno = 48.2, Sw1 = 124, Qf = 33.2, Qw = 77.1	26.43, 994.8, 360.9, 142.1, 156.1, 75.4, 4.06, 15,940, 13.29, 4.11, 48.2, 124.0, 33.2, 77.1

* Documented deviation `AISC-L-TOES`: the database does not list an angle's toe radius; with the assumed r / 2 the minor principal Iz is 1.5 % high, and Zx is 3.54 against 3.50 in³ (1.0 %). The former deviations for J of rolled shapes and the weak axis of channels are retired: the true outline now matches both tables. The same three shapes are also run through the standard-shape option on a test copy of their database rows.

CHAPTER 225

Load chasedown

Adds up gravity loads level by level down a column or a bearing wall, each level with its own area and loads, and gives the cumulative load of each type below every level, with the governing gravity combination. Live loads reduce by what has been carried so far. The Load takeoff type does one member with one set of floors; use this one when levels differ. The results are numbers to copy into a footing, column or wall calculation: nothing links them, so changing this calculation does not change those.

Inputs of the chasedown

- **Member:** a column (loads in kips) or a bearing wall (loads per foot of wall).
- **Live load element factor KLL** (Table 4.7-1). A bearing wall has no row of its own: "all other members" is 1, and its tributary area is the level's width times the **wall length** you enter, which is your choice.
- **Reduce live loads (4.7)** and **Reduce roof live load (4.8.2):** on by default as the usual practice; both are permitted, not required, so turn them off to carry the full loads. The roof slope F sets R2.
- **Levels, from the top down:** a name; the tributary area (column) or width (wall); D, L, Lr, S and R in psf; the use of a floor with a live load (assembly under 4.7.5, passenger vehicle garage under 4.7.4, or other); the member's own weight added at that level; and **added** D and L for an offset or a transfer from above, in kips (or per foot of wall). Work out an offset yourself and enter its force.

Checks performed

Result	Basis
Cumulative D, S, R	The sum of area × load, own weight and added loads
Cumulative L	4.7: at each level AT is the sum of the floors carried so far, and every floor takes L = Lo (0.25 + 15/√(KLL AT)), no less than 0.50 Lo (one floor) or 0.40 Lo (two or more), with its own Lo and use; under KLL AT = 400 ft², not reduced; Lo over 100 psf and garages by 4.7.3 and 4.7.4
Cumulative Lr	4.8.2: Lr = Lo R1 R2 on that level's own area, between 12 and 20 psf
Combination	The governing gravity combination at each level, LRFD or ASD (`loads.py`), for D, L, Lr, S and R

The type adds loads and reads **Calculated**. The results are a table per level in the report. It has no drawing: no story height or other geometry is part of the calculation.

Limitations

- Gravity only. Add wind or seismic axial forces in the receiving calculation, or use Load combinations.
- Added loads are not reduced, since they have no tributary area here. If a transfer carries reducible floors, reduce it yourself first.

- Different KLL per level is not available: one member, one factor. A one-way slab rule (4.7.6) is not applied.
- Own weight is entered, not computed. No story height is part of the calculation.

Verification examples

Verification status: 17 values compared in 7 examples (1 published, 6 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

I found no published chasedown that shows the reduction level by level. The one published check is at the base of a column:

Example	Source	Values
Interior column 2C	AISC Design Examples v16.0, Part III, Tables III-2 to III-4, pp. III-52 to III-55	PD 276 kips, PL 108 kips, PS 27.7 kips

Hand calculations from 4.7 and 4.8 cover the levels: the reduction as floors add up, the 0.50 and 0.40 limits, heavy live loads, assembly floors, walls per foot, added loads and the combinations.

CHAPTER 226

Load combinations

Takes the unfactored effect of each load (an axial force, a shear, a moment, a reaction, any number) through the ASCE 7-22 Chapter 2 combinations and gives the value of each combination and the largest and smallest result. Use it where a member calculation is done by hand or in another program, or to see which combination governs. Each effect keeps your units; the type does no conversion, so it has no units switch.

The design calculations carry their own combinations for D, L, Lr, S, R, W and E (`loads.py`). This type adds flood, ice, tornado, fluid, earth pressure, self-straining effects, and seismic combinations with ρ, Ev and Ω0. Copy the results into a calculation as values: nothing links the two.

Inputs of the combinations

- **Load effects:** up to four named effects, and a row per load type with its effect and sign: D, L, Lr, S, R, W, WT (tornado), E (entered as QE, the effect of the horizontal seismic forces), Di, Wi, Fa, F, H, T. Rows of one type add up.
- **Design method:** LRFD (2.3) or ASD (2.4). 2.1 says an element is designed by one or the other.
- **Seismic:** the redundancy factor ρ (12.3.4); whether Ev = 0.2 SDS D applies, on by default (12.4.2.2 excepts SDC B and some foundation checks, and it is your call that an exception holds); SDS; and whether to add the overstrength combinations with Ω0 (Table 12.2-1) where the standard requires them.
- **Code options:** the factor 0.5 on L (2.3.1 and 2.3.6 exception 1), off by default because it depends on the occupancy; the flood zone (2.3.2, 2.4.2); whether H is permanent; and the factor on T, which 2.3.4 leaves to you at no less than 1.0 and 2.4.4 at no less than 0.75.

Checks performed

Result	Basis
Basic LRFD	2.3.1: 1a to 5a; (Lr or S or R) and (L or 0.5W) written out one by one; W or WT
Basic ASD	2.4.1: 1a to 7a, the same way
Flood	2.3.2 (4b, 5b), 2.4.2 (5b, 6b, 7b), by zone, used when Fa is entered
Ice	2.3.3 (2b, 4c, 4d, 5c), 2.4.3 (1b, 2b, 3b, 7c), used when Di is entered
Seismic	2.3.6 (6, 7), 2.4.5 (8, 9, 10) with Eh = ρ QE (12.4.2.1), Ev (12.4.2.2), and Emh = Ω0 QE (12.4.3) as separate combinations
F	The factor of D in 1a to 4a, 6, 7 (LRFD) and 1a to 6a, 8, 9, 10 (ASD)
H	1.6 (LRFD) or 1.0 (ASD) where it has the sign of the rest of the combination; otherwise 0.9 or 0.6 if permanent, 0 if not
T	The entered factor in every combination

Result	Basis
Largest and smallest	W, WT, E and Wi in both directions. A variable load that helps is left out, for "effects of one or more loads not acting": D and F stay

The type is a calculation of numbers: it reads **Calculated**, and it has no drawing, as the unit converter does.

Limitations

- Extraordinary events (2.5), the alternative for water in soil (2.3.7) and the capacity limit on Emh (12.4.3.1: enter a lower effect yourself) are not covered, nor exceptions 2 and 3 of 2.3.1, those of 2.4.1, or the 0.9D of 2.4.5 exception 2. The allowable stress increase of 2.4.5 belongs to the member check.
- The effects add linearly. Where the analysis is not linear (second-order effects), combine the loads in the analysis itself.
- A row is one effect at one location. Two effects from one combination must be read together from its rows, not from the largest and smallest values, which can come from different combinations.
- ρ, Ω0 and the seismic design category are your inputs.
- The H factor follows the sign against the rest of that combination, for each effect on its own. "Principal load effect" is a judgment in the code: where it reads differently for your case, check that combination by hand.
- T is added to every combination with one factor; 2.4.4 lets you lower it where its peak is unlikely to coincide, not per combination here.

Verification examples

Verification status: 53 values compared in 13 examples (3 published, 10 hand calculations from the cited equations): 52 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

The combinations are the code's own text, so most of the checks are hand calculations of each combination group. Few published examples apply these on their own. Three do use them:

Example	Source	Values
Beam moment	AISC Design Examples v16.0, F.1-1A, pp. F-3 to F-4	266 kip-ft (LRFD), 184 kip-ft (ASD)
Shear wall axial force, with Ev	FEMA P-2192-V1 Table 4-8, p. 151	10,015 kips, 6,460 kips; and −378 kips, a documented deviation
Collector at ASD	FEMA P-2192-V1 Section 7.4, pp. 267 to 272	13,350 lb

In FEMA's table only results are printed: D and L were worked back from its first two rows, so those rows confirm that step and not the program. With them and QE = 3,385 kips from the text, the program gets 10,011 and 6,462 kips for rows 3 and 4. Row 5 prints −378 kips where the same inputs give −308 kips: rows 4 and 5 differ by 2 QE, and their printed difference is 6,838 kips, not 6,770. The program keeps its own value.

Hand calculations cover the LRFD and ASD basic groups, overstrength, both flood zones, ice, F, H adding and resisting, loads not acting, the 0.5 factor on L and T. The test file also checks that the combination set equals the one in `loads.py` for every mix of D, L, Lr, S, R, W and E tried.

CHAPTER 227

Wall and pier rigidity

Finds the lateral stiffness of a concrete or masonry wall with openings, or of solid piers on a line, and each pier's share of a force. Copy the rigidity into Rigid diaphragm torsion or your building model: nothing links them. The type reads **Calculated**; it has no pass or fail.

Inputs of the wall rigidity

- **What to analyze:** one wall with openings, or up to 12 solid piers sharing one displacement.
- **Force V at the top:** shared by rigidity. The rigidities do not depend on it.
- **Material:** concrete (Ec by ACI 318-19 19.2.2.1, from f'c and wc), concrete masonry (Em = 900 f'm, TMS 402-22 4.2.2.2), clay masonry (Em = 700 f'm), or E entered.
- **G/E:** 0.4 prefilled (TMS 402-22 4.2.2.2 and the usual value for concrete).
- **Factors on I and on the shear area:** 1.0 is the gross section. ACI 318-19 6.6.3.1.1 gives 0.70 uncracked and 0.35 cracked on I for walls; whether the wall cracks is your call. They apply to every part.
- **Effective thickness t:** the actual thickness of concrete or fully grouted masonry, an equivalent solid thickness for partial grout. Rigidity k uses it; relative R does not.
- **Top of the wall:** cantilever, or fixed against rotation by a stiff level above. Which it is, is your call: the sources tell you to judge where the inflection point sits.
- **Strip at the base, deducted as:** where the openings reach the base of a cantilever wall, the strip taken out of the solid wall can be a cantilever (TM 5-809-3 Example 2, a stiffer wall) or fixed at both ends (CMACN 2002, softer). In TM 5-809-3 Example 2 the difference is about 4 %.
- **Openings:** up to 12, each by its distance from the wall's left end, its sill above the base, width and height. A door has sill 0. An opening reaching the top of a cantilever wall leaves free-standing piers.
- **Piers:** length, height, thickness, ends (cantilever or fixed), and corner. A corner pier is an L or T: I × 1.5 and a shape factor of 1.0 in shear (TM 5-809-3 7-8).

Checks performed

Result	Basis
E, G	ACI 318-19 19.2.2.1; TMS 402-22 4.2.2.2; or entered
Deflection of a rectangular part: V/(E t) [c (h/L)³ + 1.2/(G/E) (h/L)], c = 4 cantilever, 1 fixed	TM 5-809-3 Eq. 7-11, 7-12; flexure plus shear
Wall: solid wall, less the strip with the openings, plus its piers	TM 5-809-3 7-9; CMACN Summer 2002
Piers in a strip: side by side, rigidities add; stacked, deflections add	TM 5-809-3 Example 2
Shear in each pier by rigidity; moment V h for a cantilever, V h / 2 for fixed ends	Statics

Result	Basis
Advisory: stiffer than the solid wall	The cantilever strip at the base can do this; change it to fixed
Advisory: staggered openings	Openings overlapping in height with no clean line between them: the tallest counts over the full band, a softer wall than drawn
Advisory: shallow spandrel	Under half the height of the band, the piers are softer than fixed

The wall is read as a stack of bands: each group of openings overlapping in height is one strip. Inside it, an opening running the band's full height cuts it into piers; a shorter one, such as a window above a sill, stacks a solid strip in series with the piers beside it.

Limitations

- The method is approximate, and its sources say so: TM 5-809-3 7-8 warns that any simplified method can give inconsistent results and asks for judgment. It ignores the rotation of the parts and assumes where they are fixed. Finite elements are the check for a wall that matters.
- Linear elastic. No cracking beyond the factors you enter, no foundation rotation, no flanges on a wall with openings. No strength check: design the piers in their own calculation.
- Openings are rectangles lined up with the wall.

Verification examples

Verification status: 34 values compared in 10 examples (10 published, 0 hand calculations from the cited equations): 34 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
12 × 20 ft wall, one window	TM 5-809-3 Design Example 1, pp. 7-10, 7-11	Solid 0.3461 in., total 0.3726 in., k 2.68
12 × 20 ft wall, door and window	TM 5-809-3 Design Example 2, pp. 7-12, 7-13	Solid 0.4829 in., total 0.6469 in., four piers, shares 7.6, 24, 68.4 %
Chart equations	TM 5-809-3 Figure 7-5a	All four curves at h/d = 1
Two walls with a door	CMACN Masonry Chronicles Summer 2002	0.28, 0.315 in.; 1.57, 1.71 in.; R 0.585
Solid walls	Yaw (2022) Table 2	R 1.2898, 1.1119, 0.3754
Five piers	CMACN Masonry Chronicles Spring 2002 Table 1	R 1.58 to 0.04

TM 5-809-3 reads its deflections off a chart, so its values carry a chart reading, up to about 1 % here. The app computes the chart's equations. CMACN prints Em = 1.5 ksi, evidently 1,500 ksi with the thousands dropped: its deflections are run as printed. None of these is written to TMS 402-22 or ACI 318-19: the method is older than both, and only E and G come from the current codes. No published example has staggered openings, a fixed top or an opening reaching the top; hand calculations cover the last two.

CHAPTER 228

Plate bending

A thin rectangular plate with each edge simply supported, fixed or free, under uniform and patch pressures: moments, deflection and first yield. Use it for floor plate, hatches, covers, tank panels and stair treads. No code sets this method: it is plate theory, solved by finite elements.

Inputs of the plate

- **Spans a and b, thickness t.**
- **Edges:** simply supported (no deflection, free to rotate), fixed (no deflection and no slope), or free. A real bolted or tack-welded plate is usually between the first two, and which applies is your call. Two simply supported edges or one fixed edge are needed to hold the plate.
- **E, ν, Fy:** steel prefilled. For another metal, enter its values; the φ and Ω used are AISC's, so choose an Fy that fits your code.
- **Own weight:** on by default, as load type D.
- **Pressures:** up to 8, each with a load type, uniform or on a patch. A wheel or post is its force over its contact area. A patch needs to be at least 1 % of the span in each direction.
- **Deflection limit:** shorter span over the value you enter. No code sets it for a plate; 120 is prefilled.

Auto-design varies the thickness. Spans, edges, loads, material and the deflection limit are kept.

Checks performed

Check	Basis
D, Mx, My, Mxy	Thin-plate theory; finite elements with cubic Hermite functions, at least 24 per direction, closer around patches
First yield, von Mises at the worst point	6 M / t² per stress; AISC 360-22 Chapter F factors φ 0.90, Ω 1.67. Load combinations ASCE 7-22 2.3.1 or 2.4.1
Deflection	Largest of the ASD combinations, against your limit
Advisory: span under 10 × thickness	Shear deformation, ignored here, starts to count
Advisory: deflection over half the thickness	Membrane action stiffens the plate; linear theory overstates this
Advisory: patch smaller than the thickness	Thin-plate theory overstates the moment

First yield is a conservative limit on purpose: a plate has reserve past it by redistribution and membrane action, which you may count in your own calculation.

Limitations

- Linear, thin, flat, of constant thickness, on continuous edges. No stiffeners, openings, interior supports or discrete fasteners. A simply supported plate's corners lift in reality; here the edges hold against uplift.
- No point load: a true point load has an infinite moment in this theory. Enter its contact patch.
- A long span over 20 × the short span stops: use a strip of unit width.
- No fatigue, no tread pattern, no plate-to-support design. Tread plate: use the base thickness.

Verification examples

Verification status: 37 values compared in 9 examples (9 published, 0 hand calculations from the cited equations): 37 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
All edges simply supported, b/a 1, 2, 5	Batista, arXiv 1001.3016 (2010), Table 2a	Deflection, Mx, My at the center
x edges simply supported, y edges fixed, b/a 1, 2	Tables 3a, 3b	With My at the fixed edge
x edges simply supported, y edges free, b/a 1, 2	Tables 4a, 4b	With deflection and Mx at the free edge
All edges fixed, b/a 1, 2	Tables 5a, 5b	With moments at the fixed edges

These are series solutions printed to eight places, and the paper compares them with Timoshenko and Woinowsky-Krieger. Roark and Timoshenko themselves are not on hand. Patch loads have no published example here: they are checked against the Navier double series, a hand method for simply supported edges. Down to the smallest patch tried, 1/16 of the span, the moment here is within 2 % of that series. Patch loads on fixed or free edges are not checked against anything else. First yield is a plain limit with no published plate example.

CHAPTER 229

Flood loads

Nominal flood loads Fa on a wall or a pile by ASCE 7-22 Chapter 5: hydrostatic, hydrodynamic, breaking waves, debris impact, local scour, flotation and breakaway walls. Copy the loads into the design of the element: nothing links them. The type reads **Calculated**, though flotation and the breakaway limit give advisories.

Inputs of the flood loads

- **Flood zone:** V-Zone, Coastal A-Zone, noncoastal A-Zone. It sets whether breaking waves apply, the depth coefficient and the factors on Fa.
- **Risk Category:** sets Cp (Table 5.4-1) and CI (Table C5.4-1).
- **Water:** saltwater 64.0 pcf, freshwater 62.4 pcf.
- **G:** the eroded ground, after storm erosion and before local scour (5.3.2).
- **DFE:** BFE plus any freeboard your community sets. The hydrostatic and hydrodynamic loads go to here.
- **Stillwater depth:** 0.65 (BFE − G) by Eq. 5.4-3, or a stillwater elevation from the Flood Insurance Study. FEMA P-55 uses the study's elevation.
- **Element:** a wall, per foot, or a square or round pile of width D. Eq. 5.4-4 takes 1.4 D for a square. **Piles in the row** only multiplies the breaking wave on a row.
- **Surface exposed to free water:** a foot more of depth (5.4.2). On by default on a wall; a pile takes no extra foot here.
- **Flow velocity:** upper bound √(g ds), lower bound ds / 1 s, or entered. C5.4.3 gives the range as interim guidance for coastal areas, and tells you to use a better source if you have one.
- **Shape factor a:** not less than 1.25 (Eq. 5.4-1). **Cd:** 0 uses 2.0 square, 1.2 round, 1.25 wall; FEMA P-55 Table 8-2 runs from 1.25 to 2.0.
- **Waves:** dry behind the wall or water on both sides; surface angle from horizontal; wave approach angle.
- **Debris:** weight W (1,000 lb prefilled, a reasonable average in C5.4.5), by Eq. C5.4-3 with the natural period, duration 0.03 s and CO 0.8, or by FEMA P-55 Eq. 8.9 with Cstr. Upstream screening sets CB.
- **Flotation:** the displaced volume and dead weight. 0 leaves it out.
- **Breakaway wall:** wind and earthquake on it, and its intended collapse pressure.

The design method sets the factors on Fa only.

Two depths are used on purpose. Hydrostatic pressure, and the surcharge and drag on a wall, go to the DFE (ASCE 7-22 5.4.2, 5.4.3). Drag on a pile uses D × ds, the stillwater depth, as FEMA P-55 Eq. 8.8 and its examples do, while ASCE 7-22 5.4.3 leaves moving water to fluid mechanics. Breaking waves use ds (5.4.4).

Checks performed

Load	Basis
ds, Hb	Eq. 5.4-3, 5.4-2

Load	Basis
Velocity	Eq. C5.4-1, C5.4-2, or entered
Hydrostatic, on a wall	5.4.2
Hydrodynamic, wall up to 10 ft/s	Eq. 5.4-1, uniform over the projected area
Hydrodynamic, wall over 10 ft/s and any pile	½ Cd ρ V^2 A, FEMA P-55 Eq. 8.8
Breaking wave, pile	Eq. 5.4-4, at the stillwater level
Breaking wave, wall	Eq. 5.4-5, -6 or -7, then Eq. 5.4-8, -9. Holds the hydrostatic part: do not add 5.4.2 to it
Debris impact	5.4.5, Eq. C5.4-3 or FEMA P-55 Eq. 8.9
Local scour, pile	FEMA P-55 Eq. 8.10, 8.11
Flotation (advisory)	5.3.1, combinations 5b or 7b: 0.9 D or 0.6 D against the factored force
Breakaway wall	5.3.3: largest of wind, earthquake and 10 psf; collapse at no more than 20 psf (advisory)

Limitations

- Loads only: the element, its embedment and anchorage are designed elsewhere.
- Wave slam, nonbreaking wave analysis, wave run-up and uplift under a floor (5.4.4 names them) are out. Tsunami (Chapter 6) is out. The breaking wave on a wall is depth limited, so it is the code's method and no wave model.
- The depth is one value: no erosion analysis, no dune loss. Enter G after that study.
- Local scour at a wall comes from a coastal or geotechnical engineer: FEMA P-55's method for walls is not built.
- Scour is reported and not carried into the depths: FEMA P-55's example does the same.

Verification examples

Verification status: 24 values compared in 11 examples (6 published, 5 hand calculations from the cited equations): 24 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
8 in. square pile, oceanfront	FEMA P-55 Vol. II Example 8.4, pp. 8-40 to 8-43	ds 4.6, V 4.6 and 12.2, Hb 3.6 ft; drag 909, wave 868 lb, row of 7 6,076 lb, debris 2,440 lb; scour 1.76, 5.28 ft
10 in. round pile	Example 8.3, pp. 8-30, 8-31	Drag 509, 2,037 lb; wave 1,816 lb

FEMA P-55 is to ASCE 7-10; the equations used are the same in 7-22. Its row total reads "(625 lb)(7)", a leftover: 868 × 7 is the 6,076 it prints. Its scour diagonal uses a 7.5 in. dressed pile though the 8 in. goes into the load equations, so the scour values run at 7.5 in.

No published example was found for a wall. Eq. 5.4-5 to -9, 5.4.2, Eq. 5.4-1 and Eq. C5.4-3 are hand calculations from the code equations, which only guards against a typing error. The tables are read from the code. ASCE 7-22 is the only source for Table C5.4-2's depth coefficient at 2 ft and 3 ft, used here as a straight line, as Figure C5.4-1 shows.

CHAPTER 230

Lateral pressure from surface loads

The lateral pressure on a wall from surface loads behind it, added to the soil load of ASCE 7-22 3.2.1: wide surcharges, point loads, line loads along or toward the wall, strips and rectangles. Copy the pressure into the design of the wall as the load H of Chapter 2: nothing links them. The type reads **Calculated**.

Inputs of the surface loads

- **Wall height H:** from the ground surface to the base of the part looked at.
- **Wall movement:** unyielding, or yielding to active conditions, which takes 75 % of the limited loads (DM7.2 4-4.2). A wide uniform surcharge is not reduced, as K already sets it. Your call.
- **Coefficient K:** for a wide uniform surcharge, at rest for a restrained wall or active. From your soil values.
- **Backfill:** a row of ASCE 7-22 Table 3.2-1, the values of a geotechnical report entered, or not included. Table 3.2-1 is the code's minimum where no report gives the values. MH, CH, OL, OH are unsuitable as backfill and are not listed.
- **Soil pressure:** at rest where the top is restrained, active where the wall may move (3.2.1).
- **Surface loads:** up to 10 by kind. Enter the value that fits: q for uniform, strip and rectangle, Q for a point, Q line for a line.
  - **x:** from the wall to the load, or to its near edge. **x2:** to its far edge.
  - **y:** along the wall from the section looked at, to a point or a perpendicular line, or the two ends of a rectangle. Line and strip loads along the wall act the same at every section.

Checks performed

Result	Basis
Uniform: K q	DM7.2 4-4.2.1
Point	DM7.2 Figure 4-18, with fall-off by $\cos^2(1.1\,\theta)$ along the wall
Line along the wall	DM7.2 Figure 4-19
Line toward the wall	DM7.2 Figure 4-20
Rectangle	Doubled elastic stress for a rigid wall, integrated: Figure 4-17
Strip along the wall	The limit of that rectangle: $(2\,q / \pi)(\beta - \sin\beta \cos 2\alpha)$
Soil load	ASCE 7-22 3.2.1, Table 3.2-1
Total, resultant, height, base moment	Sum at 49 depths, by the trapezoid rule

Limitations

- Rigid wall basis: conservative for active pressure, possibly unconservative for passive. A flexible wall may see up to half as much (the elastic value not doubled).
- Level ground, no water. Below free water, 3.2.1 takes buoyant soil plus full hydrostatic pressure: not here. Expansive soil is not here. Compaction pressures (DM7.2 4-4.3) are not here. Surcharge below a wall's base is not carried into a toe.
- Deep footing loads: the equations are for loads on the surface.
- No wall design.

Verification examples

Verification status: 37 values compared in 24 examples (24 published, 0 hand calculations from the cited equations): 35 within tolerance, 2 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Point load, six distances	UFS 3-220-20 Figure 4-18, p. 206	PH and y/H of the printed table
Line load, seven distances	Figure 4-19, p. 207	PH by the printed formula, y/H of the printed table
Rectangle at its corner	Figure 4-17, p. 205	Influence factors at five values of m, read off the chart
Soil loads	ASCE 7-22 Table 3.2-1, p. 72	Six cells

No published worked example was found with a whole wall and its loads added up. Those checks test the method against the printed curves, tables and formulas. The Figure 4-17 values are read off a chart by eye, good to about 0.005; the reading at m = 2.0 misses by a hair and is a documented deviation. Figure 4-20 has no printed table: it is checked against a thin rectangle by hand. Other classic sources (Caltrans Trenching and Shoring Manual, NAVFAC DM-7.2 of 1986, AASHTO) use the same equations; only Caltrans's Chapter 4 was read and it does not print them.

CHAPTER 231

Tornado loads

Tornado pressures on a rectangular building by ASCE 7-22 Chapter 32, for Risk Category III and IV in the tornado-prone region: whether design is required, the main wind force resisting system, and components and cladding. The same building's wind pressures stand beside them, since the structure takes the greater. The type reads **Calculated**.

Inputs of the tornado loads

- **Risk Category, tornado-prone region, Essential Facility.** Essential Facilities take KdT 1.0 on C&C and need protected glazing.
- **Tornado speed VT:** from Figures 32.5-1 and 32.5-2 or the ASCE 7 Hazard Tool, for your site and Ae. Interpolation by the logarithm of Ae is permitted, and it can decide whether design is required. **The type does not read the maps.**
- **Effective plan area Ae:** a record of what VT was read for (32.5.4).
- **Basic wind speed V and exposure:** for the 32.5.2 threshold and the wind pressures. Tornado pressures use neither exposure nor topography (32.7, 32.8).
- **Ground elevation factor Ke:** Table 26.9-1, or off for 1.0.
- **Plan dimensions, h, roof, θ.** A pitched roof's ridge runs along y.
- **Tornado enclosure classification:** enclosed, partially enclosed, partially open. 32.12 counts unprotected glazing as openings on each assumed windward wall: this often makes a building partially enclosed for tornado and not for wind. You decide and enter it.
- **zop, Ri:** partially enclosed only.
- **GT:** 0.85 prefilled (32.11).
- **C&C zones:** each with its effective wind area.

Checks performed

Result	Basis
Design required	32.1.1, 32.5.2
KzTor, qhT	Table 32.10-1, Eq. 32.10-1
MWFRS, both directions, both signs of internal pressure	Eq. 32.15-1, Figure 27.3-1, KdT 0.80, KvT 1.1 on roof uplift
C&C, h up to 60 ft	Eq. 32.17-1, Figures 30.3-1, 30.3-2
C&C, h over 60 ft	Eq. 32.17-2, Figure 30.4-1
Wind pressures beside them	Chapter 27 and 30 with Kd 0.85, G 0.85, the wind GCpi, and 16 psf at least on C&C

Which load governs may differ by surface: wind often governs the lateral system and tornado the interior of the roof. Tornado loads act as WT in Chapter 2 (the Load combinations type has it).

Limitations

- Rectangular, rigid buildings. No open buildings (Eq. 32.15-2), elevated buildings, parapets, overhangs, canopies, rooftop equipment, solar panels, bins, tanks or other structures (32.16, 32.17.3 to 32.17.5). No Appendix G longer return periods. No wind tunnel.
- Over 60 ft the C&C here covers roofs of 7 degrees or less.
- For a building over 200 ft, the pressure is at h: walls lower down have a larger KzTor up to 1.0 and the table does not list each level.
- The wind pressures are a side-by-side for the same simple building: use the Wind loads type for design wind.
- VT is your input: see the first line above.

Verification examples

Verification status: 55 values compared in 6 examples (6 published, 0 hand calculations from the cited equations): 54 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
1-story school, Evansville, VT 80 mph, partially enclosed	NCSEA Wind Engineering Committee, Tornado Loads on Schools and Hospitals (11/2023), pp. 3 to 8	qhT 16.2 psf; MWFRS tables; C&C at 10 and 100 ft²
4-story hospital, Oklahoma City, VT 104 mph, enclosed, Essential Facility	The same, pp. 14 to 21	qhT 26.5 psf; MWFRS tables; C&C at 10 and 100 ft²

One recorded deviation: Zone 1' of the school at 10 sq ft, where the source uses a directionality factor of 0.75 against the table's 0.90, so the source prints 12.5 and the program reads 13.3.

The building dimensions are read through their L/B ratios, as the figures of the example are not used. The wind pressures of the NCSEA tables print no derivation (they do not follow from the stated data by a simple check), so they are not compared. The wind columns here are hand checked only. The NCSEA equation lines above its tables have typing errors ("18.0 Cp" where 11.0 is meant in the school); the tables themselves match Eq. 32.15-1.

CHAPTER 232

Sheet pile wall

A steel sheet pile wall, cantilever or with one row of anchors, in layered soil with water on both sides and a uniform surcharge: the penetration, the anchor force, the moment and first yield of the section, by UFS 3-220-20 (DM7.2, 2026) 4-7.

Inputs of the sheet pile

- **Retained height H, anchor depth, surcharge.** DM7.2 4-4.2: about 300 psf is common for storage and equipment.
- **Water behind and in front:** depths below the top. The difference is a net water pressure, carried to the tip with no drop for seepage, as in DM7.2's example. A depth past the tip means dry.
- **Soil layers:** up to 6, top down, the last running on. Moist and saturated unit weights, φ, c or su. For clay in undrained terms, φ 0 and c the undrained strength. **Ka, Kp:** 0 takes Rankine. Enter a log spiral Kp with wall friction if your geotechnical engineer gives one.
- **Factor of safety on passive:** divides passive pressure. Enter 1.0 if your Kp is already factored. DM7.2 uses 1.5 to 2.
- **Factor on the depth:** 1.2 for scour and future dredging.
- **S, I, Fy, E per foot of wall:** from the manufacturer. 18.1 in^3/ft and 84.4 in^4/ft are PZ-22 (DM7.2 Table B-9), prefilled with its 38.5 ksi. **There is no sheet pile table in the app, so auto-design is not offered.**
- **Rowe's reduction:** anchored walls only, for penetration in medium dense or better sand. None is the safe side.
- **Loss to corrosion:** a share of S. Default 0, with an advisory, as the service life and exposure are yours.

The design method sets the steel check only: H with its load factor, 1.6, against φ Mn; or H against Mn / Ω. Earth pressures stay unfactored with the factor on passive.

Checks performed

Result	Basis
Net pressure	DM7.2 4-7: active plus water behind, factored passive in front
Anchored: D from moments about the anchor, T from equilibrium	DM7.2 Figure 4-37, free earth support
Cantilever: rotation point and D from force and moment equilibrium	DM7.2 Figure 4-41
Design penetration	D × your factor, rounded up to a foot
Rowe	DM7.2 Eq. 4-31, Figure 4-36
Flexure, first yield	Mn = Fy S, AISC 360-22 Chapter F factors
Advisory: corrosion	Left at 0

Limitations

- Not checked: global stability, seepage and piping, basal heave, interlock strength, deflection, axial load, and the driving of the pile.
- The anchor rod, wale, connections and deadman are not designed. One row only.
- Level ground, no sloped backfill. No tapering of net water below the dredge line. No limited surcharges: use the Lateral pressure from surface loads type and judge how to carry it over.
- Rowe reduces the anchored moment only as DM7.2 states it.

Verification examples

Verification status: 14 values compared in 3 examples (3 published, 0 hand calculations from the cited equations): 13 within tolerance, 1 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Anchored bulkhead, 24 ft	UFS 3-220-20 Appendix B-2, pp. 606 to 614	D 14.45 ft, design 18 ft, T 9,568 lb/ft, connections 11,400 lb/ft, zero shear 4.57 ft, M 63.4 kip-ft/ft, ρ 18.52, factor 0.62, design M 39.3 kip-ft/ft, S 12.25 in³/ft
Cantilever in sand, 15 ft	Figure 4-43, p. 245	D 14.25 ft and M 31,590 lb-ft/ft read off a chart
Cantilever, sand over clay, 20 ft	Figure 4-45, p. 247	D 17.2 ft and M 51,840 lb-ft/ft read off a chart

The cantilevers come from DM7.2's charts, not from a calculation shown line by line, so they compare at 5 %. The Figure 4-43 moment, run on the example's stated soil, is 8 % below the printed value, a documented deviation on the less conservative side. Caltrans Trenching and Shoring Chapter 7 has only soldier pile examples. The steel check has no published value here: DM7.2's example divides by Fy with no margin. USACE EM 1110-2-2504 and USS Steel Sheet Piling Design Manual would add examples and a section table, and they were not on hand.

CHAPTER 233

Beam-column joint

The shear in a reinforced concrete beam-column joint and its detailing, by ACI 318-19 Chapter 15 or, for a special moment frame, 18.8: Aj and the table coefficient, column shear, strong column, joint depth for bars passing through, hooks and hoops. One direction of the frame.

Inputs of the joint

- **Frame:** special moment frame (bars at 1.25 fy, φ 0.85, Table 18.8.4.3), or other (bars at fy, φ 0.75, Table 15.4.2.3).
- **Joint:** interior, a beam each side; or exterior, one beam.
- **Column continues, beam continues, confined:** the three questions of the table. Yes where the column meets 15.2.6, the beam 15.2.7, and two transverse beams meet 15.2.8. These are your reading of the frame.
- **Column:** h along the beams, b, the distance between its mid-heights. The inflection points are taken there.
- **Sum of the columns' nominal strengths:** from your column calculation, at the axial force giving the least strength (18.7.3.2).
- **Beams:** bw, h, d, the width for the top moment, the offset from the column axis, and top and bottom bars on each side. Add slab bars within the flange to the top bars: SP-17 warns that ignoring the slab is unconservative for the strong column check.
- **Column shear:** statics from the beam bars with no beam shear; or from entered moments and beam shears (Mpr and Ve for a special frame, 18.8.4.1); or entered.
- **Joint hoops:** bar, legs along b, spacing, cover, hx, the smallest column bar, Pu.
- **Beams on four sides:** with each beam at least 3/4 of the column's width, halves the hoops and allows 6 in. (18.8.3.2).

Auto-design varies the column depth and width and the hoop spacing. Materials, beams, axial force and the frame's choices are kept.

Checks performed

Check	Basis
Joint shear Vu ≤ φ Vn	18.8.2.1, 18.8.4 or 15.4; Aj by 15.4.2.4
Strong column, weak beam	18.7.3.2, special frames
Joint depth for bars through	18.8.2.3, interior joints of special frames
Hooked bars	18.8.5.1 or 25.4.3, exterior joints

Check	Basis
Joint hoops, amount	18.8.3, Table 18.7.5.4
Joint hoops, spacing	18.7.5.3, 18.8.3.2
Joint hoops, other frames	15.3.1.3, 15.3.1.4: at least two layers, at most 8 in. Skipped where confined; 15.3.1.1 also needs the joint to be outside the seismic system and outside SDC D, E, F, which you confirm

Limitations

- One direction. Enter a second calculation for a joint framed both ways.
- A beam moment computed here is a singly reinforced rectangle: enter it from your beam calculation when compression bars or a T section matter.
- The column, the beams, wide beams with bars outside the core (18.8.3.3), headed bars and splices are out. So is a joint of a column on a wall or a transfer.

Verification examples

Verification status: 47 values compared in 23 examples (7 published, 16 hand calculations from the cited equations): 42 within tolerance, 5 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Intermediate moment frame	ACI SP-17(14) Column Example 3, p. 380	T 252, C 108, Vj 308 kips, Aj 576 in²
Special moment frame	ACI SP-17(14) Column Example 4, p. 392	T 315, C 180, Vj 424 kips, Vn 611 kips
Vn, all cells	ACI 318-19 Tables 15.4.2.3, 18.8.4.3	Hand calculation

SP-17(14) is to 318-14, the only published example on disk: ACI MNL-17(21) would be to 318-19. Two values differ, both documented. Under 318-19 Example 3's joint takes 20 in place of 15 in Vn. Its column shear of 52 kips does not follow from the moments and shears it prints, which give 56 kips; the source's number is kept when the shear is entered, and statics when you enter moments. Hoops, strong column and hooks are hand calculations. The auto-design is a simple size search, untested against a published design.

CHAPTER 234

Post-tensioned slab strip

A post-tensioned one-way slab, or one strip of a two-way slab, by load balancing on continuous spans with overhangs, to ACI 318-19: uplift, hyperstatic moments, service stresses, transfer, strength with fps, minimum steel, shear and deflection. F is entered: you supply the losses, or the tendon supplier does.

Inputs of the slab

- **Slab:** one-way, or a strip of a two-way slab. For two-way: Class U at 6 √f'c, 125 psi of precompression, and the bars of 8.6.2.3. The strip's width and its share of the panel come from your equivalent frame: this type does not distribute moment across the panel.
- **Spans and tendon:** up to 8. e at each support above mid-depth, e at midspan below it. A tendon is a parabola in each span. Reverse curves over supports are not modeled: the small downward kick is left out.
- **Overhangs.**
- **Design section at the supports:** the support centerline, prefilled, or the face of support with its width. ACI 318-19 7.4.2.1 permits the face for a slab built integrally with its supports, and does not require it, so the centerline is the safe side. The sheet and the report say which is in use. The example below labels its moments as at the face, yet they agree with centerline values; until that is resolved the lower number is not the default.
- **Loads:** own weight on by default, superimposed dead, live. Live load runs on all spans, alternate spans and adjacent spans.
- **Tendons:** unbonded or bonded, strand area, spacing, fse after losses (175 ksi is common for unbonded strand), fpu, γp. **f'ci, force at transfer:** your estimate of what the losses take.
- **Bars:** top and bottom per foot at one depth d.
- **Deflection limit:** span over the value you enter.

Auto-design varies the thickness and strand spacing. Tendon positions are kept.

Checks performed

Check	Basis
wp = 8 F a / L²	Load balancing
M2 = Mpt − F e, linear between supports	5.3.11
Service tension and class	Table 24.5.2.1: U, T, C one-way; U two-way
Service compression	24.5.4 at 0.60 f'c, all load
Transfer	24.5.3: 0.60 f'ci; 3 √f'ci as an advisory
Precompression	8.6.2.1 two-way; a practice guide (advisory) one-way
φMn	22.2, 20.3.2.3 or 20.3.2.4.1; tendons and bars as one couple
Mu	1.2D + 1.6L or 1.4D, pattern, plus 1.0 M2

Check	Basis
Minimum bars	7.6.2.3 unbonded one-way; 1.2 Mcr bonded (7.6.2.1); 8.6.2.3 two-way
One-way shear	2 λ √f'c b d, simple and on the safe side
Deflection	Unbalanced load, uncracked, immediate

Limitations

- **Losses are not calculated.** Enter fse. No friction, seating or long-term loss analysis.
- Class T and C slabs are flagged but their crack control (24.3) and cracked deflection are not checked.
- Not designed: anchorage zones (25.9), tendon spacing, bar lengths, punching (use Punching shear), long-term deflection.
- Uniform loads only, on prismatic spans. No point loads, no drop panels. One-way slabs per strip.
- 0.45 f'c under sustained load is not separated out: the check uses all load at 0.60 f'c.

Verification examples

Verification status: 23 values compared in 2 examples (2 published, 0 hand calculations from the cited equations): 20 within tolerance, 3 recorded deviations. **Thin: only 2 published examples.** Code basis: partial. Only some provisions were read in the target edition; the rest rests on older editions, examples or summaries.

Example	Source	Values
Four 20 ft spans, 6 in. slab	ACI SP-17(14), One-way Slab Example 3, pp. 65 to 73	Aps, F, F / A, wp; M2; fps, a, φMn; Mu with M2; Vc

To ACI 318-14, with moments from a program and rounded by the source, compared at 5 %. The text says 0.25 in. at the exterior support but uses a drape of 3.25 in., which needs 0.5 in.: run at 0.5 in. It runs as stated, with 12 in. supports and moments at the faces (the option, not the default), and the negative moments at the second and middle supports are then about 15 % below the printed values, a documented deviation on the less conservative side: the printed negative moments agree with this type at the centerline, though the source says at the face. **Only this one example was run.** SP-17 Two-way Example 3 and the 318-19 examples of ACI MNL-17(21) have not been: the two-way route is hand checked only. PTI Technical Notes in the library need OCR.

CHAPTER 235

Effective length factor

Finds the effective length factor K of a column (k in ACI 318), solved from the equations behind the alignment charts, so no chart is read by eye. It also gives the idealized values of Commentary Table C-A-7.1 and K2 by the two story approaches. The result is a number to enter in a column calculation: nothing links them, so changing this calculation does not change that one.

Inputs of the effective length

- **Code:** AISC 360-22 (G), or ACI 318-19 (ψ). The equations are the same. ACI 318-19 6.6.4.4.3 uses Ec and the inertias of 6.6.3.1.1, so the factor on I defaults to 0.70 for columns and 0.35 for beams; enter another factor where Table 6.6.3.1.1(a) or an analysis allows it.
- **Way of finding K:** alignment chart equations; idealized end conditions; K2 by story stiffness; K2 by story buckling. Story approaches are AISC only.
- **Frame:** sidesway uninhibited or inhibited. Which it is, is your call.
- **Column:** E, I about the axis perpendicular to the plane of buckling, unsupported length L.
- **Column inelasticity τb** (AISC): the Commentary replaces E I with τb E I for all columns in G, using Eq. C2-2 with the required strength Pr and Pns = Fy Ag, α = 1.0 for LRFD and 1.6 for ASD. On by default. It may be conservatively ignored, which gives a larger K. Pns is taken as Fy Ag: reduce the area for a slender section.
- **Top and base:** a joint with its members, or G entered. A base may be pinned (G = 10 prefilled, the Commentary's value unless it is a true frictionless pin) or rigidly attached to a footing (G = 1.0 prefilled; smaller if justified by analysis).
- **Members at a joint:** other columns and girders in the plane of buckling. The column under consideration counts already. For each girder:
  - **Far end:** as the chart assumes; rotation prevented (× 2 braced, × 2/3 sway); pinned (× 1.5 braced, × 1/2 sway); or MF/MN from a first-order lateral analysis, which sets Lg' = Lg (2 − MF/MN), Eq. C-A-7-4.
  - **Q/Qcr:** axial load in the girder over its in-plane buckling load at K = 1.0. Multiplies its E I / L by (1 − Q/Qcr).
  - **Connection:** a shear-only connection gives no restraint and is left out of G. Partially restrained connections: adjust the factor on I yourself from a documented moment-rotation response.
- **Story** (story approaches): Pstory for all columns of the story including gravity-only ones; Pmf, ΔH, H and Hcol for story stiffness; the sum of $\pi^2 EI/(Kn2\ L)^2$ for the moment frame columns, and Kn2, for story buckling.

The design method only sets α in τb.

Checks performed

Result	Basis
G or ψ at each end	AISC 360-22 Commentary Eq. C-A-7-3, with the adjustments of Commentary 7.2; ACI 318-19 Fig. R6.2.5.1

Result	Basis
τb	AISC 360-22 Eq. C2-2a, C2-2b
K, sidesway inhibited	Eq. C-A-7-1, solved numerically from 0.5 to 1.0
K, sidesway uninhibited	Eq. C-A-7-2, solved numerically from 1.0 up
K, idealized	Table C-A-7.1, theoretical or recommended
K2, story stiffness	Eq. C-A-7-5 with RM of Eq. C-A-7-6, and its lower limit
K2, story buckling	Eq. C-A-7-8 and its lower limit √(5/8) Kn2
Lc, Pe	Lc = K L, Pe = π²EI/(K L)²

The type reads **Calculated**. It has no pass or fail: a column calculation uses K. Pc from K2 may not exceed the sidesway inhibited value: compare it there.

Limitations

- The charts assume elastic behavior, constant sections, rigid joints, and the curvature stated for each frame. These seldom hold exactly, and you judge where they do.
- MF/MN of 2 or more makes G negative: the type stops, since it solves the equations for positive G only. Solve it yourself and enter G.
- Under the direct analysis method (AISC 360-22 Chapter C) K = 1.0, and this type is not needed.
- ACI 318-19 gives no idealized table or K2: those ways stop for that code. For nonsway members ACI permits k = 1.0 and its commentary recommends it.
- The LeMessurier approach of the Commentary is not built.

Verification examples

Verification status: 32 values compared in 26 examples (16 published, 10 hand calculations from the cited equations): 32 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
W14X82 moment frame column, both segments	AISC Design Examples v16.0, E.4A, pp. E-30 to E-33	G 1.38, 1.63, 1.62, 1.0; K about 1.5 and about 1.4, read off the chart by the source, so compared to within 0.05
The same pinned at the base	E.4B, p. E-34	G 10; K about 2.0, the same comparison
Idealized end conditions	AISC 360-22 Commentary Table C-A-7.1	All six theoretical and recommended values

No published value of Eq. C-A-7-5 or C-A-7-8 was found. Hand calculations from the Commentary's equations cover both story approaches and their lower limits, the chart equations' limits (0.5, 0.699, 1.0 braced; 1.0, 1.316, 2.0 sway), the girder adjustments and ACI's factors.

CHAPTER 236

Moving load

Moves a train of wheel loads over a beam of 1 to 6 spans and gives the envelopes of moment, shear, reaction and deflection, the absolute maximum moment, the moment influence line at a station, and the forces at any one position of the train. Use it for crane runways, forklifts, equipment on rails and vehicles on a slab strip. The results are numbers to copy into a member calculation: nothing links them.

Inputs of the moving load

- **Spans,** up to 6, with a support at each end of each span. The left and right ends are a pin or fixed.
- **Stiffness:** an AISC shape (Ix, E = 29,000 ksi, with its self-weight if you choose) or E and I entered, constant along the beam. It sets the moments on continuous spans and all deflections.
- **Uniform dead load:** on all spans, no pattern.
- **Wheels,** up to 16: distance from the first wheel and load, without impact. For a crane use the maximum wheel load of ASCE 7-22 4.9.2.
- **Impact:** the percentages of ASCE 7-22 4.6.3 (machinery) and 4.9.3 (cranes), or entered. A manufacturer's larger value governs. ASCE 7-22 gives none for vehicles and forklifts in this list, so enter yours.
- **Run the train both ways:** matters for a train that is not symmetrical on a beam that is not symmetrical. On by default.
- **Deflection limit (span / n):** under the train with impact. 0 = none. Your limit: AISC Design Guide 7 lists limits for runways.
- **Influence line at:** station for the moment influence line; 0 = where the absolute maximum moment occurs.
- **Forces at one position:** with the first wheel at a position you give, the moment and shear at that station and the reactions.

Checks performed

Result	Basis
Absolute maximum moment, one simple span	AISC Manual 16th Ed. Table 3-22, General Rules: under a wheel, with the span's centerline midway between that wheel and the resultant of the wheels on the span. Every wheel and every group of wheels is tried
Absolute maximum moment, other beams	Train moved over a grid and with each wheel at each of 24 stations per span, in both directions, the peak refined under each wheel
Moment, shear, reactions at each station	Slope-deflection with joint rotations as unknowns, exact for point loads, and statics
Deflection at each station	Joint rotations plus the fixed-end deflection under each wheel in that span. Peak at the stations
Envelopes with the dead load	The wheels as L and the uniform load as D, ASCE 7-22 2.3.1 or 2.4.1, each station's largest and smallest

Result	Basis
Deflection limit	Span / n, each span

The type reads **Calculated**, or not adequate only against a deflection limit you set.

Limitations

- Vertical loads only. The lateral and longitudinal crane forces of 4.9.4 and 4.9.5, fatigue and web crippling are not here; the Crane runway type, where available, covers a runway beam on one span.
- A beam of at most 300 ft. A wheel's effect fades within a span or two, so model the spans either side of the part you are designing.
- No overhangs. Supports are rigid and at the span ends. Constant EI. First-order.
- Uniform live load and patterns are not combined with the train: add them in a beam analysis and add the effects.
- The envelope stations are 24 per span, so an envelope's value between stations is slightly under its true peak. The absolute maximum moment is refined; the combined envelope's value is at the stations.

Verification examples

Verification status: 29 values compared in 13 examples (9 published, 4 hand calculations from the cited equations): 29 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Runway beam, 30 ft span, wheels 12 ft apart	AISC Design Guide 7, 3rd Ed. (2019), Example 14.1.1	Moment 9.60 per kip of wheel, 478 kip-ft with impact, 386 kip-ft without
One, two equal and two unequal moving loads	AISC Manual 16th Ed. Table 3-22, cases 43 to 45, p. 3-231	Mmax on both branches, R1 max
Three loads on a simple beam	Udoeyo (2020), Structural Analysis, Example 9.8, PDF p. 261	Reactions, moment and shear at midspan, influence ordinates

No published train over continuous spans is on disk. Hand calculations from the three-moment equation cover the load at the center of one of two spans (R = 13/32, 11/16, −3/32 P), the largest moment over the support (−0.0962 P L at a = L/√3), the fixed-ended span and the dead load on two spans.

CHAPTER 237

Dead load build-up

Adds up the weight of a roof, floor, wall or ceiling assembly, layer by layer, from the tables of the ASCE 7-22 Commentary or your own values. The total is a number to copy into a load takeoff, a chasedown or a member calculation: nothing links them.

Inputs of the build-up

- **Assembly:** a name for the report.
- **Layers,** up to 40:
  - **Table items:** Table C3.1-1a, as a weight per area, per inch or per 1/8 inch of thickness; Table C3.1-2 densities, with the thickness; CMU by unit density, thickness and grout spacing.
  - **Entered weight,** or **entered density** with a thickness, with a name. Use these where you know the product: ASCE 7-22 3.1.2 asks for actual weights, and the table is for when you have no definite information.
  - **Thickness:** for the items rated by thickness. One without a thickness weighs nothing, and a note says so.
  - **On the slope:** a layer measured along a sloped roof.
- **Roof pitch:** rise per 12. Layers on the slope are converted to the horizontal projection. Other layers are left as entered.
- **Tributary width:** gives a line load as well. 0 = none.

Checks performed

Result	Basis
Layer weight	Commentary Tables C3.1-1a, C3.1-2, or entered
Total	The sum of the layers; on the horizontal projection when a layer is on the slope
Line load	Total × tributary width

The type adds weights and reads **Calculated**. It has no drawing: most layers have no thickness in the calculation, so a section would be invented. Masonry weights include mortar but not plaster: add the plaster item for each plastered face. The table has a considerable range for some constructions, as its commentary says: check the actual product where it matters.

Limitations

- Fixed equipment, solar panels and vegetative roofs are yours to enter as layers.
- The psf table prints the slate floor finish "per mm thickness" with 15. Its SI table has 0.028 kN/m² per mm, which is 15 psf per inch. It is taken per inch here and its name says so.
- A metric assembly converts the psf table exactly. It does not use the rounded values of Table C3.1-1b.

- The density table gives ranges for some lightweight concretes. Those rows are not offered: enter the density.

Verification examples

Verification status: 22 values compared in 22 examples (22 published, 0 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Table cells	ASCE 7-22 Commentary, Table C3.1-1a	17 cells across the kinds of row, each at the value printed
Densities	ASCE 7-22 Commentary, Table C3.1-2	Reinforced stone concrete, grout

No published take-off is reproduced. Hand additions cover the total, the slope and the line load.

CHAPTER 238

Unit converter

Converts structural quantities between US customary and metric (SI) units with exact factors. Choose the quantity, enter the value, and pick the unit to convert from and to; the results also list the value in every other unit of the quantity.

Inputs of the unit converter

- **Quantity:** length, area, force, moment, stress and pressure, line load, area load, unit weight (force per volume: pcf, kN/m³), density (mass per volume: lb/ft³, kg/m³), speed, moment of inertia, section modulus, temperature.
- **Value** and the **From** and **To** units of that quantity.

Unit weight and density are different quantities: a unit weight of 1 pcf is a density of 1 lb/ft³ only under standard gravity, so they are kept apart.

Results and checks of the unit converter

Row	Basis
Conversion basis	NIST SP 811 (2008) Appendix B: 1 in = 0.0254 m, 1 lb = 0.45359237 kg and gn = 9.80665 m/s² are exact, so 1 lbf = 4.4482216152605 N, 1 kgf = 9.80665 N; 1 mi = 5280 ft, 1 acre = 43,560 ft², 1 knot = 1852 m/h; the step shows both units in the SI base unit and the factor
Temperature	Through kelvin: 0 °C = 273.15 K, 0 °F = 459.67 °R, 1 °F = 5/9 K (an offset, not a factor)
Round trip	The result converts back to the entered value
Value is physical	A temperature below absolute zero is NOT FEASIBLE

The *ton* is the short ton (2000 lbf); the *tonne-force* is the metric 1000 kgf.

Limitations

- Results are shown to six significant figures; the factors are exact to double precision.
- Mass, energy, power and angle are not offered.

Verification examples

Verification status: 34 values compared in 34 examples (34 published, 0 hand calculations from the cited equations): 34 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Exact values from the NIST SP 811 definitions (tests/test_verify_unit_converter.py), matched to 1 part in 10^9:

Example	Conversion	Exact	ILX Calcs
UC-1	1 ft	304.8 mm	304.8
UC-2	1 kip	4.4482216152605 kN	4.44822
UC-3	1 ksi	6.894757293168 MPa	6.89476
UC-4	1 psf	47.88025898034 Pa	47.8803
UC-5	1 kip-ft	1.3558179483314 kN-m	1.35582
UC-6	1 klf	14.593902937206 kN/m	14.5939
UC-7	1 pcf	157.0874638 N/m³	157.087
UC-8	1 in⁴	416,231.4256 mm⁴	416,231
UC-9	100 mph	44.704 m/s	44.704
UC-10, UC-11	32 °F, 212 °F	0 °C, 100 °C	0, 100
UC-12	1 lb/ft³ (mass)	16.01846337 kg/m³	16.0185

Published factors of NIST SP 811 (2008), Appendix B.8 (seven significant digits; matched to 1 part in 10^6; PDF page of the free NIST document in brackets):

Example	Conversion	NIST SP 811	ILX Calcs
SP811-1, -2	1 kip; 1 ksi	4.448 222 kN [63]; 6.894 757 MPa [63]	same
SP811-3, -4	1 psi; 1 psf	6.894 757 kPa [65]; 47.880 26 Pa [65]	same
SP811-5, -6, -7	1 lbf·ft; 1 lbf·in; 1 kgf·m	1.355 818; 0.112 984 8; 9.806 65 N·m [63, 65]	same
SP811-8, -9	1 lbf/ft; 1 lbf/in	14.593 90; 175.126 8 N/m [65]	same
SP811-10, -11	1 ton-force (2000 lbf); 1 kgf	8.896 443 kN [67]; 9.806 65 N [63]	same
SP811-12, -13	1 in⁴; 1 ft⁴	4.162 314 × 10^{-7}; 8.630 975 × 10^{-3} m⁴ [61, 62]	same
SP811-14, -15	1 in²; 1 ft²	6.4516 × 10^{-4}; 9.290 304 × 10^{-2} m² [66, 67]	same
SP811-16 to -19	1 mile; 1 yd; 1 mph; 1 ft/s	1.609 344 km; 0.9144 m; 1.609 344 km/h; 0.3048 m/s [61, 64, 68]	same
SP811-20	1 lb/ft³ (mass)	16.018 46 kg/m³ [66]	same
SP811-21, -22	1 atm; 1 bar	101.325 kPa; 100 kPa [57]	same

CHAPTER 239

Beam with varying stiffness

A linear elastic analysis of a continuous beam whose stiffness EI changes along its length: haunches, cover plates over a support, lapped members, a cracked zone, a stiffer member in one span. It gives the shear, moment and deflection envelopes and the reactions under the ASCE 7-22 load combinations or one load set, with pattern live load. No code sets this method: it is beam theory, solved by finite elements. Size the member in its material calculation with these forces.

Inputs of the beam with varying stiffness

- **Base stiffness:** E and I entered, or an AISC shape (Shapes Database v16.0, Ix about the strong axis; its self-weight can be added as dead load). This is the stiffness wherever no segment is entered. E = 29,000 ksi is prefilled (steel); enter the modulus of your material and the gross, effective or cracked I you judge right.
- **Stiffness segments:** each row gives E and I from x0 to x1, measured from the left end of the beam (overhang included). A segment replaces the base stiffness where it lies: for a cover-plated zone enter the I of the member with its plates. Where two segments overlap, the lower row in the table wins.
- **Linear haunch:** enter I at x1 as well. I then varies linearly from I at x0 to I at x1, modelled as the number of steps of constant I you choose (8 prefilled), each at its midpoint value. A haunch whose depth varies linearly has an I that varies roughly with the cube of the depth: if that matters, enter it as several segments that follow your I.
- **Spans and supports:** the left support and each span with the support at its right end (pin, fixed or none), and the left and right overhangs, as in the continuous beam analysis.
- **Loads:** line loads (uniform, partial or linearly varying), point loads and concentrated moments, each with its load type (D, L, Lr, S, R, W, E) and where it acts. *From* and *To* are measured from the start of the span or overhang chosen; *From* = *To* = 0 loads all of it. Downward and clockwise positive.
- **Load combinations:** ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) by the method switch, or one unfactored load set.
- **Pattern live load:** ASCE 7-22 4.3.3: live load on all spans, on each span alone, on alternate spans and on each pair of adjacent spans, enveloped.
- **Deflection limits:** span / n for live load and for total load, your choice; the IBC 2021 Table 1604.3 floor values (360 and 240) are prefilled and 0 turns a check off. For an overhang or cantilever the span is twice its length.

The beam is an analysis: it has no auto-design.

Checks performed

Check	Basis
Shear, moment and deflection envelopes, reactions	Cubic Hermite beam elements with EI constant in each element and a node at every change of stiffness, every support and every load end: exact at the nodes for these loads
Stiffness as analysed	The pieces of constant EI, drawn under the elevation; the haunch steps show as steps
Pattern live load	ASCE 7-22 4.3.3

Check	Basis
Live and total load deflection	Service loads (single live load types; ASD combinations without W and E) against span / n entered, IBC 2021 Table 1604.3

A stiffer zone attracts moment: over an interior support a cover plate raises the hogging moment there and lowers the span moments. The analysis shows how much.

Limitations of the beam with varying stiffness

- Euler-Bernoulli, first-order, linear elastic: no shear deformation (it counts in deep or short members), no second-order effects, no cracking that follows the load (enter the cracked I where you judge it applies).
- Stiffness is constant in each piece. A linear haunch is approximated in steps; add steps to see the result settle.
- Up to 6 spans and 20 segments. Supports are rigid; no springs, no settlement.
- No member strength checks, no connection or haunch detailing: the drawing shows the stiffness entered, not plates or welds.

Verification examples

Verification status: 13 values compared in 5 examples (1 published, 4 hand calculations from the cited equations): 13 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Two 12 ft spans, fixed and pinned, I and 3I, triangular and point loads	Udoeyo, Structural Analysis (2020), Example 11.2 (slope-deflection)	End and support moments, moment under the load, three reactions
Two spans, I and 2I, load on the first span	Hand calculation, three-moment equation	Support moment $-wL^2/12$ ($-wL^2/16$ for a prismatic beam), end reactions
Simple span with a stiffer middle, midspan load	Hand calculation, unit load method	Midspan deflection; midspan moment
Cantilever with a stiffer half at the fixed end	Hand calculation, unit load method	Tip deflection
Cantilever with a linear haunch, 40 steps	Hand calculation, closed-form integral	Tip deflection

Udoeyo's example is a published worked example with members of different I; the others are closed forms worked by hand from the equations named. A published example of a haunched or cover-plated continuous beam (for example from a bridge design text) is not on hand; the haunch is checked against the exact integral of the linear variation.

CHAPTER 240

Cable and catenary

A flexible cable between two supports, level or at different heights, under its own weight, uniform loads along the span and point loads: horizontal and largest tension, sag, length, elastic stretch, the effect of a temperature change, and the largest tension against the breaking strength. Use it for guy and stay cables, catenary supports of canopies and nets, messenger cables and lifelines. The cable is solved exactly (catenary, parabola or the polygon of the point loads), with the parabolic approximation shown beside it.

Inputs of the cable

- **Span L and support heights:** the horizontal distance between the supports and how far the right support is above the left (negative when lower).
- **Sag under dead load:** the vertical distance from the chord to the cable, at midspan or at a point you give, under the dead load at the installation temperature. This is the state the cable is erected to; it fixes the cable's unstressed (fabrication) length.
- **Cable weight q:** along the cable, as dead load.
- **Minimum breaking strength, E and metallic area A:** from the manufacturer or the strand and rope standards (ASTM A586, A603). A 1 in. structural strand is prefilled as an example (122 kips, 24,000 ksi, 0.596 in²): enter your cable. Elastic stretch can be turned off for an inextensible cable.
- **Thermal coefficient α and the temperature rise and drop:** α in millionths per degree, in the same degree scale as the temperature changes (°F in US units, °C in metric). Steel is about 6.6 per °F (12 per °C). The rise and drop are your design range from the installation temperature.
- **Loads:** vertical, downward: uniform loads per horizontal length over the whole span or from *From* to *To*, and point loads at *From*, each with its load type (D, L, Lr, S, R).
- **Load combinations:** ASCE 7-22 2.3.1 (LRFD) or 2.4.1 (ASD) by the method switch, or one unfactored load set. ASCE 19 (structural applications of steel cables) may add its own requirements: to confirm against its text.
- **Resistance factor φ and safety factor Ω:** 0.65 and 2.2 are prefilled; ASCE 19 and the fittings set them, and you choose them (to confirm against ASCE 19-16, which is not on hand).
- **Largest service sag allowed:** for clearance; 0 turns the check off.

The cable has no auto-design: change the cable or the sag and run it again.

Checks performed

Check	Basis
Reference state: H, tensions at both supports, length, stretch, unstressed length	General cable theorem H y = M of the simple beam (Irvine 1981; Udoeyo 2020, 6.2.1), iterated with the weight along the cable
Parabola against the exact shape	H, tension and length of the parabolic approximation (weight along the chord taken as uniform along the span)

Check	Basis
Temperature rise and drop	Same unstressed length, grown by α ΔT and stretched by ∫ T ds / EA: the sag and H after each
Cable tension	Largest tension of the strength combinations, at the installation temperature and after the drop, against φ Tb or Tb / Ω
Largest service sag	ASD combinations (or the load set) at the installation temperature and after the rise, against your limit
Advisory: sag ratio	Outside about 1/40 to 1/7

Every combination is solved as its own equilibrium: the cable is not linear, so the combinations are not superposed. The unstressed length is the one installed, so a heavier load sags the cable more and raises H less than in proportion when the cable stretches.

Limitations of the cable

- Vertical gravity loads only: no wind on the cable, no uplift, no ice accretion shape, no dynamic effects (galloping, vortex shedding) or fatigue.
- Rigid supports at fixed positions; no prestress from a turnbuckle other than through the sag entered; the sag up to 30 % of the span.
- The cable is perfectly flexible and linear elastic; strand constructional stretch and creep are not counted (use the manufacturer's prestretched modulus).
- Fittings, sockets, anchorages and the supporting structure are not designed. ASCE 19-16 is not on hand: its factors and load combinations are to be confirmed.

Verification examples

Verification status: 18 values compared in 6 examples (2 published, 4 hand calculations from the cited equations): 18 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: written without the code document on hand, and confirmed only in part through published examples.

Example	Source	Values
Three point loads, sag at a load	Udoeyo, Structural Analysis (2020), Example 6.6	H, two sags, tensions at both supports
Uniform load along a 20 m span	Udoeyo Example 6.8	H, largest tension, length (the book's series value)
Catenary under its own weight	Hand calculation, catenary closed form (Irvine 1981)	H, largest tension, length, the parabola's H
Elastic stretch of a parabolic cable	Hand calculation	H, stretch $H/EA \int(1 + y'^2)\,dx$
Right support 20 ft higher	Hand calculation, general cable theorem	H unchanged, the two end tensions
Temperature rise, inextensible	Hand calculation with the length series of Udoeyo Eq. 6.20	Sag increase

Udoeyo's Example 6.8 prints a horizontal reaction of 9,000 N that does not agree with its own T0 of 3,000 N (the theorem is misapplied there); the 3,000 N is the right value and is the one used. Irvine's Cable Structures is not on hand: the catenary relations used are the standard closed forms, quoted from knowledge.

CHAPTER 241

Natural frequency and dynamic response

The natural frequency of a mass on a spring or of a uniform beam, and the response to a harmonic force or a pulse: dynamic amplification, phase, transmissibility, the largest displacement and the equivalent static force. Use it for machine supports, equipment platforms, a first look at a floor or footbridge frequency, and impact or blast-type pulses. It is structural dynamics, closed forms of the textbooks; no code sets it.

Inputs of the dynamic system

- **System:** a mass on a spring (weight W and stiffness k), or a uniform beam: simply supported, cantilever, fixed both ends, or fixed and pinned, with E, I, the span L and the weight per length w that moves with it (for a floor, the dead load and the expected live load, not the design live load).
- **Damping ratio ζ:** your estimate, in percent of critical. Welded steel about 1 to 2 %, bolted steel 2 to 3 %, concrete 2 to 5 %. It is used for the harmonic response; the pulse peaks are undamped.
- **Force:** none (frequency only), a harmonic force F0 at the forcing frequency f, a rectangular pulse or a half-cycle sine pulse of peak F0 and duration td. On a beam the force acts at midspan, or at the tip of a cantilever.
- **Limits:** the lowest natural frequency you accept and the largest dynamic displacement, each optional (0 turns it off).

This is an analysis: it has no auto-design.

Checks performed

Check	Basis
Natural frequency, period, damped frequency	$\omega n = \sqrt{(k/m)}$, $m = W/g$; beam $f = \lambda^2/(2\pi L^2)\sqrt{(EI/\bar{m})}$, the first three modes (Blevins, Table 8-1)
Equivalent system of a beam	Static stiffness at the load point (48, 3, 192 and 768/7 EI/L^3) with the mass that gives the first-mode frequency
Harmonic response	$Rd = 1/\sqrt{((1 - r^2)^2 + (2\zeta r)^2)}$, phase, transmissibility (Chopra 3.2, 3.7)
Rectangular and half-sine pulses	Undamped peak of the forced and free phases (Chopra 4.7, 4.8)
Natural frequency at least the limit	Your limit
Largest dynamic displacement	Rd F0 / k against your limit
Advisory: near resonance	r from 0.8 to 1.25, where the response depends on damping

The equivalent static force Rd F0 is what to apply in a static calculation of the support for the peak response.

Limitations of the dynamic system

- One degree of freedom, linear, viscous damping. A beam is reduced to its first mode at the load point; the higher modes are listed but not in the response.
- Uniform beams of constant EI and mass only; no axial load, shear deformation or rotary inertia.
- Pulses: undamped and of the two shapes given; no general load history, no impulse entered as an area (a pulse much shorter than the period acts as one, and the report says so).
- Not a floor vibration check: for walking and rhythmic excitation of floors use the floor vibration calculation (AISC Design Guide 11).

Verification examples

Verification status: 19 values compared in 8 examples (8 published, 0 hand calculations from the cited equations): 19 within tolerance, 0 recorded deviations. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Example	Source	Values
Mass on a spring, harmonic force	Chopra, Dynamics of Structures, Ch. 2 and 3 (closed forms, hand values)	fn, Tn, Rd, phase, transmissibility, displacement
Simply supported beam	Blevins, Table 8-1; AISC Design Guide 11 Eq. 3-3	First three modes; the simplified 0.18 √(g/Δ), within 0.5 %
Cantilever, fixed-fixed, fixed-pinned	Blevins, Table 8-1	First mode, stiffness at the load point
Rectangular and half-sine pulses	Chopra 4.7, 4.8	Rd at td/Tn = 1/4 and 1

Chopra and Blevins are not on hand: their equations are standard and quoted from knowledge, and the values are computed from them by hand. AISC Design Guide 11 is in the reference library.

CHAPTER 242

Stress transformation

Plane stress at a point: the stresses on a rotated plane, the principal stresses and their angle, the largest shear in the plane and in three dimensions, the von Mises stress, and Mohr's circle. Use it for a point in a plate, a web, a weld throat or a connection element where you have the stress components. It is mechanics of materials, in closed form.

Inputs of the stress state

- **σx, σy, τxy:** the stress components, tension positive; τxy positive on the positive x face in the positive y direction (the convention of Gere and Goodno). For the von Mises check they are the required stresses of the method chosen: factored for LRFD, service for ASD.
- **Angle θ:** of the rotated plane, counterclockwise from x to the plane's normal, in degrees.
- **Limit stress Fy, φ and Ω:** the yield stress for first yield, or the limit you choose. 0.90 and 1.67 are prefilled, the AISC 360-22 factors for yielding; no code sets this check for a general stress state, so they are yours to set.

This is a check of one stress state: it has no auto-design.

Checks performed

Check	Basis
Stresses on the rotated plane	Transformation equations, σx1 + σy1 = σx + σy
Principal stresses and angle	σ1,2 = σavg ± R, tan 2θp = 2τxy/(σx − σy)
Largest in-plane shear and its plane	R, at 45° from the principal planes
Largest shear in three dimensions	σz = 0: half the largest difference of σ1, σ2 and 0
Von Mises stress against the limit	√(σ1² − σ1σ2 + σ2²) ≤ φ Fy or Fy / Ω

On Mohr's circle the normal stress is drawn to the right and the shear downward, so the diameter turns in the same sense as the element, by twice its angle.

Limitations of the stress transformation

- Plane stress (σz = 0) at one point; no three-dimensional state, no strain transformation.
- Von Mises is a first-yield criterion for ductile metals: for brittle materials, concrete or wood use the criterion of their code.
- The stresses are entered: the calculation does not find them from forces and a section.

Verification examples

Verification status: 25 values compared in 5 examples (1 published, 4 hand calculations from the cited equations): 25 within tolerance, 0 recorded deviations. **Thin: only 1 published example.** Code basis: written without the code document on hand, and confirmed only in part through published examples.

Example	Source	Values
Element rotated 45°	Gere and Goodno, Mechanics of Materials, Ch. 7, first transformation example	Rotated stresses, principal stresses and angle, shears, von Mises
Metric input, element rotated 30°	Hand calculation, transformation equations	Rotated stresses in MPa
Pure shear, mixed signs, uniaxial compression	Hand calculation	Principal stresses, angles, largest shears, von Mises

Gere and Goodno is not on hand: the numbers of its example are as recalled and are exactly what the transformation equations give.

CHAPTER 243

Custom calculation

Your own calculation sheet, printed in the standard report and saved in projects like any other calculation: variables with units, formulas shown as written, then with the values substituted, then evaluated with units, narrative text, headings, code references and pass / fail checks. Units are part of every value, so adding a length to a force, or comparing a moment with a stress, is an error shown on that cell, not a wrong number. You are the engineer of record: the method, the references and the limits on the sheet are yours.

Inputs of the custom calculation

- **Title:** the calculation's heading in the report.
- **Cells**, in the order they print. Each cell has a kind:
  - **Variable:** a name and a value with its unit, for example `L = 20 ft`, `w = 1.2 kip/ft`, `Zx = 54.6 in^3`, `fc = 4000 psi`, `phib = 0.90`. A variable may also be a short formula (`12 ft + 6 in`).
  - **Formula:** a name and an expression of other names, for example `Mu = w*L^2/8`. The report shows `Mu = w × L² / 8 = (1.2 kip/ft) × (20 ft)² / 8 = 60 kip-ft`.
  - **Check:** a comparison, for example `Mu <= phiMn`, with a title and a reference. It reads OK or NG like any check and counts in the calculation's status; `<=` and `<` give the ratio left / right, `>=` and `>` right / left.
  - **Text** (up to 500 characters), **Heading**, and **Code reference** (what the provision says, and its section).
  - **Display unit** (variables and formulas): the unit the result is shown in, for example `kip-ft` or `kN-m`. Left empty, the unit follows from the values (kip/ft × ft² reads kip-ft; ksi × in³ reads kip-in).
  - **Reference:** the code section, table or source of the cell (AISC 360-22 Eq. F2-1).
- **Units switch:** shows every result in US or metric units (kip -> kN, ft -> m, ksi -> MPa, psf -> kPa); a display unit you entered is shown as its counterpart in the other system. What you typed is never changed.

Cells may be in any order: each one is evaluated after the cells it names. A sheet holds up to 150 cells.

Writing expressions

Write	Meaning
`20 ft`, `1.2 kip/ft`, `4000 psi`, `54.6 in^3`, `54.6 in³`, `29000 ksi`, `30 deg`, `5 %`	a number and its unit (the unit directly after the number; inside a compound unit no spaces: `kip-ft`, `kN/m`)
`[psi]`, `1 [kip / ft]`	a unit in brackets: 1 psi; spaces allowed inside
`+ - * / ^` (also `**`, ×, ·, ÷, −, superscript digits)	arithmetic; `^` is a power, right to left (`2^3^2` = 512), `-2^2` = −4
`sqrt`, `sin`, `cos`, `tan`, `asin`, `acos`, `atan`, `atan2(y, x)`, `log` (base 10), `ln`, `exp`, `abs`, `min(a, b, ...)`, `max(...)`, `pi`	functions; trigonometric functions take radians or an angle with `deg`
`<`, `<=`, `>`, `>=`, `==` (also ≤, ≥)	in check cells only

Names start with a letter and may hold digits, `_` and `'` (`fc`, `f'c`, `phiMn`, `Mu`). A leading Greek word prints as its letter: `phiMn` prints φMn, `lambda` prints λ, `DeltaL` prints ΔL. Unit names may also be variable names, because a unit is read only right after a number or in brackets; to divide a number by a variable whose name is a unit, put the variable in parentheses (`10 ft / (h)`).

Units: length in, ft, yd, mi, mm, cm, m, km; force lbf (lb), kip, N, kN, MN, kgf; mass kg, g, tonne, lbm; time s, ms, min, h; frequency Hz, rpm; temperature difference degF (°F), degC (°C), K; stress and pressure psi, ksi, psf, ksf, Pa, kPa, MPa, GPa, bar; line load plf, klf; unit weight pcf, kcf; energy and power J, kJ, W, kW; speed mph; angle rad, deg (°); percent %. Factors are the exact NIST SP 811 definitions (1 in = 25.4 mm, 1 lbf = 4.4482216152605 N).

Empirical equations with √f'c. ACI writes √f'c with f'c in psi and the result in psi. Write it so the units say so: `2*lambda*sqrt(fc/[psi])*[psi]*bw*d`: the root of the number of psi, times 1 psi.

There is no auto-design: the sheet is yours, nothing on it is a design variable the program could know.

Checks performed

Item	What it does
Each check cell	Your comparison, OK or NG, with its ratio; it counts in the status like any check
Each formula and variable	Printed step by step (symbols, values, result) under Detailed Calculations, and in the calculation sheet table
Cell errors	A cell that cannot be evaluated makes the calculation NOT FEASIBLE and names the cell and the reason: a syntax error, an undefined name, a name used twice, a circular reference (the cells of the ring are named), adding or comparing unlike quantities, a display unit of another kind, a square root of a negative number, an overflow. Cells that use a cell in error say so
How the sheet is evaluated	Notes the cell count and how many checks are satisfied

The report prints the whole sheet in order as the **Calculation sheet** table (in the Summary of Checks), then each formula and check step by step.

Limitations

- No user-defined functions, conditionals (if / else), tables, loops, matrices or vectors. Each check holds one comparison.
- Temperatures are differences: °F and °C convert by 5/9 with no offset. Enter a temperature change, not a reading.
- Plain numbers in trigonometric functions are radians.
- Expressions are limited to 300 characters, 200 symbols and 40 levels of nesting; exponents to ±1000. This keeps a mistyped or hostile entry from tying up the server.
- Results print to four significant figures.
- The units switch converts what is shown; text cells and headings print as typed in either system.

Verification examples

Verification status: 8 values compared in 3 examples (3 published, 0 hand calculations from the cited equations): 8 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Published examples typed as sheets (tests/test_verify_custom_calc.py):

Example	Quantity	Published	ILX Calcs
AISC Design Examples v16.0, F.1-1A	wu, Mu, φMn of a W18X50 over 35 ft	1.74 kip/ft, 266 kip-ft, 379 kip-ft	1.74, 266.4, 378.8
AISC Manual Table 3-23, case 1 (formula by hand)	$5wL^4/384EI$ for the same beam, wL = 0.75 kip/ft, I = 800 in^4	1.092 in.	1.092
ACI 318-19 Table 22.5.5.1 (formula by hand)	$Vc = 2\lambda\sqrt{f'c}\ bw\ d$, 4000 psi, 12 × 21.5 in.	32.64 kips	32.64
NIST SP 811 Appendix B.8	1 ksi, 1 kip-ft, 1 lbf/ft	6.894 757 MPa, 1.355 818 kN-m, 14.593 90 N/m	same

The parser is tested against hostile entries (attempts to reach Python, attribute access, subscripts, huge exponents, deep nesting, very long expressions): each stops with a message in a fraction of a second (tests/test_custom_calc.py).

CHAPTER 244

Thermal movement

Free and restrained thermal strain, movement and force of a member or a building segment, the movement an expansion joint must take, and the length between expansion joints against your criterion. Use it for long roof and floor framing, exposed members, rails and pipes, parking structures and masonry or concrete walls.

Inputs of the thermal movement

- **Material:** structural steel (α = 6.5 × 10⁻⁶ /°F), stainless steel (9.6), aluminum (13), normalweight concrete (5.5, the middle of the ACI 209R range, which depends on the aggregate), clay masonry (4.0) and concrete masonry (4.5, TMS 402-22 4.2.4, to confirm against the code text), wood along the grain (2.0); or **Entered** with your own α.
- **Length between joints L:** the member, or the building segment between expansion joints.
- **Design temperature change ΔT:** your value. Federal Construction Council Technical Report No. 65 (1974) takes the larger of the summer temperature exceeded 1 % of the time less the mean temperature during construction, and that mean less the winter temperature exceeded 99 % of the time. Metric: °C (a change converts by 5/9, without offset).
- **Restrained force:** degree of restraint r (100 % between rigid supports, the upper bound; your estimate where the supports flex), E, A, and the member's available axial strength from its own calculation (φPn or Pn/Ω).
- **Expansion joints:** the joint's rated movement; the allowable length for ΔT that you read from TR-65 Fig. 1 (heated building, hinged column bases) or your own criterion; heating and cooling (air-conditioned +15 %, unheated −33 %), fixed column bases (−15 %) and a building much stiffer at one end (−25 %), the report's adjustments as the AISC and NRC guidance quote them (to confirm against the report).

Each of the three checks can be turned off. There is no auto-design: nothing here is a member size.

Checks performed

Check	Basis
Free thermal strain	ε = α ΔT
Free thermal movement	δ = α ΔT L
Restrained thermal force	σ = r E α ΔT, P = σ A against the member's available axial strength
Expansion joint movement	δ against the joint's movement capacity
Length between expansion joints	L against the allowable length times the TR-65 adjustments

Limitations

- Axial movement only: no thermal gradient through the depth (bowing), no frame analysis of the columns that restrain a roof, no creep or shrinkage of concrete and masonry (add them to the movement where they matter).
- The restraint is one number; the force in a real frame depends on the column and connection stiffness.
- The TR-65 chart is not built in: you read the allowable length from it.

Verification examples

Verification status: 7 values compared in 5 examples (5 published, 0 hand calculations from the cited equations): 7 within tolerance, 0 recorded deviations. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Hand values of the formulas (tests/test_verify_thermal_movement.py); no published worked example is on disk:

Example	Case	Hand value	ILX Calcs
TH-1	Steel, 100 ft, ΔT = 100 °F	δ = 0.780 in., σ = 18.85 ksi, P = 188.5 kips (10 in²)	same
TH-2	Concrete, 150 ft, ΔT = 60 °F	δ = 0.594 in.	same
TH-3	Aluminum, 2 in², ΔT = 50 °F, r = 50 %	P = 6.565 kips	same
TR65-1, -2	Adjustments	300 × 1.15 × 0.85 = 293.3 ft; 400 × 0.67 × 0.75 = 201 ft	same

CHAPTER 245

Quantity takeoff

Structural steel tonnage from AISC shapes and plates, concrete volume, reinforcing bar weight and formwork contact area, each with your waste or allowance factor. Use it for budget quantities, a check on a fabricator's or contractor's takeoff, or the quantities that go with a set of calculations.

Inputs of the quantity takeoff

- **Steel shapes:** item, AISC shape name (W12X26, HSS6X6X3/8, C10X15.3, L4X4X1/2, any shape of the AISC Shapes Database v16.0), count and length. The weight per foot is the database's nominal weight.
- **Plates:** item, thickness, width, length and count, at the **unit weight of steel** (490 pcf).
- **Steel waste and connections:** your allowance for connection material, stiffeners, bolts and waste (5 % prefilled).
- **Concrete:** item, length, width, depth, count and the **formed faces** (none, sides, bottom, sides and bottom, all faces) for the formwork contact area. **Concrete waste** and **formwork allowance** are your factors.
- **Reinforcing bars:** item, bar size (#3 to #18), count and cut length (with hooks and laps, or allow for them with the **bar laps and waste** factor). Weights are the ASTM A615 / A615M Table 1 nominal weights, the same for A706.

There is no auto-design: nothing is sized here.

Checks performed

Item	Basis
Steel shapes in the AISC database	A name the database does not hold is NOT FEASIBLE
Structural steel	Σ W × count × length, plates Σ γ t b L × count, times (1 + waste); tons of 2000 lb (metric tonnes in metric units)
Concrete volume	Σ L B D × count, times (1 + waste)
Formwork contact area	Sides 2 (L + B) D, bottom L B, per row, times (1 + allowance)
Reinforcing bars	Σ unit weight × count × length, times (1 + laps and waste)
Reinforcement per volume of concrete	Bar weight over net concrete volume, to compare with similar work

The report prints a table per material with every row.

Limitations

- Concrete elements are rectangular prisms: deduct openings or add haunches as rows of their own.

- Connections, bolts, welds, stiffeners, deck, studs and miscellaneous metals are only in the waste factor unless you enter them.
- No costs.

Verification examples

Verification status: 6 values compared in 5 examples (5 published, 0 hand calculations from the cited equations): 6 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Case	Published or hand value	ILX Calcs
QT-1	10 W12X26 × 20 ft (AISC Shapes Database, 26 lb/ft)	5,200 lb	same
QT-2	Plate 1/2 × 12 in. × 10 ft, 490 pcf	204.2 lb	same
QT-3, QT-4	100 #5 × 20 ft; 10 #8 × 30 ft (ASTM A615 Table 1: 1.043, 2.670 lb/ft)	2,086 lb; 801 lb	same
QT-5	Footing 10 × 10 × 2 ft, sides formed	7.407 yd^3; 80 ft^2	same

CHAPTER 246

Rebar tables

Bar areas, diameters and weights, area per foot (per metre) for every bar at the usual spacings, the bars that give a required area, and quick development, lap splice and standard hook lengths per ACI 318-19 25.4 and 25.5 for the concrete and bar detail you describe. For a full development or splice calculation with every factor, use the Rebar development calculation.

Inputs of the rebar tables

- **Bar, spacing s, required area per foot** (checked: the area provided must reach it) and **required total area** (the tables list how many bars of each size give it).
- **Materials:** f'c, reinforcement grade (ψg of Table 25.4.2.5: 1.0 for Grade 60, 1.15 for Grade 80), concrete type and density (λ, Table 19.2.4.1).
- **Development length:** Table 25.4.2.3, the simplified expressions, with **spacing and cover condition met** your judgment of the detail (clear spacing and cover at least db with the minimum stirrups or ties along ℓd, or clear spacing at least 2db with cover at least db); or Eq. 25.4.2.4a with the **confinement term (cb + Ktr)/db** you enter (at most 2.5; 1.5 prefilled; Ktr = 0 is permitted).
- **Top bar** (ψt = 1.3), **epoxy coating** (ψe = 1.5 or 1.2; ψt ψe ≤ 1.7).
- **Lap splice class:** Class B (1.3 ℓd), or Class A (1.0 ℓd) where the area provided is at least twice that required over the lap and at most half the bars are spliced there.
- **Hooks:** ψr (1.0 with enough confining ties, else 1.6) and ψo (1.0 inside a column core with side cover ≥ 2.5 in., or side cover ≥ 6db; else 1.25), Table 25.4.3.2.
- **Length available** for the straight bar beyond the critical section.

There is no auto-design: these are reference tables for the bar you choose.

Checks performed

Check	Basis
Area per foot provided	Ab × 12 / s (Ab × 1000 / s per metre) against the required area
Bars for the required total area	n = As,req / Ab, rounded up
Development length of a straight bar	ACI 318-19 Table 25.4.2.3 or Eq. 25.4.2.4a, √f'c ≤ 100 psi (25.4.1.4), ℓd ≥ 12 in. (25.4.2.1); against the length available
Tension lap splice	25.5.2.1: max(1.3 ℓd or 1.0 ℓd, 12 in.); #14 and #18 bars are not lap spliced in tension (25.5.1.1)
Standard hook	25.4.3.1: ℓdh = (fy ψe ψr ψo ψc / (55 λ √f'c)) db^1.5 ≥ max(8db, 6 in.)

The report prints the area table for every bar and spacing and the development, lap and hook lengths for every bar.

Limitations

- Deformed bars in tension. Compression development (25.4.9), headed bars (25.4.4), bundled bars (25.6), excess reinforcement (25.4.10), welded wire reinforcement and seismic provisions of Chapter 18 are not covered.
- Hook detail (bend diameter, tail) is not drawn or checked.
- Weights are the ASTM A615 Table 1 nominal weights.

Verification examples

Verification status: 9 values compared in 8 examples (8 published, 0 hand calculations from the cited equations): 9 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Hand values of the ACI 318-19 expressions for f'c = 4000 psi, Grade 60, normalweight, uncoated (tests/test_verify_rebar_tables.py):

Example	Case	Hand value	ILX Calcs
RT-A1, -A2	#5 at 12 in.; #4 at 6 in.	0.31; 0.40 in²/ft	same
RT-D1	#5, condition met	ℓd = 23.72 in.; Class B lap 30.84 in.	same
RT-D2, -D3	#8, condition met; other cases	47.43; 71.15 in.	same
RT-D4	#6 top bar, condition met	37.00 in.	same
RT-D5	#5, Eq. 25.4.2.4a, (cb + Ktr)/db = 2.5	14.23 in.	same
RT-H1	#5 standard hook, ψr = 1.6, ψo = 1.25	14.77 in.	same

CHAPTER 247

Roof geometry

The geometry of a gable, hip or shed roof over a rectangular plan: the pitch as rise per 12, angle and percent, the slope factor, the ridge height and length, the common, hip, valley and jack rafter lengths with their cuts, and the plan and sloped areas of every roof plane. Use it to lay out rafters, to find the true length of a hip for its design, or to turn plan loads and areas into roof surface areas. Nothing is sized: design the rafters, hips and valleys in their material modules.

Inputs of the roof geometry

- **Roof:** gable (two planes to a ridge), hip (four planes; the end planes may have their own pitch) or shed (one plane from a low plate to a high one).
- **Length L and width W:** between the outside faces of the wall plates (the plate lines, where the rafter's measuring line crosses the wall). L runs along the ridge; on a hip roof it is the longer side.
- **Pitch:** as rise per 12 (6 for 6:12), as an angle, or as a slope in percent. A hip roof can have steeper or flatter end planes.
- **Eave and rake overhangs:** horizontal, from the plate lines to the eave and rake lines. A hip roof has the eave overhang all around.
- **Plate height:** for the ridge height above your datum.
- **Ridge board thickness:** half of it, measured level, is taken off the common rafters and, across their plan angle, off the hips. 0 gives the theoretical lengths to the ridge centre, as on the framing square.
- **Rafter spacing:** for the jack rafters of a hip roof.
- **Intersecting wing span:** a gable wing at the same pitch meeting the roof at right angles gives two valleys; 0 for none.

The roof geometry has no auto-design: there is nothing to size.

Checks performed

Result	Basis
Pitch, angle, slope, slope factor; common and hip length per foot of run	tan θ = rise / run; slope factor 1/cos θ; the framing-square rafter table
Rise, ridge height, common rafter (with tail and ridge deduction), ridge length	Right-triangle trigonometry on the measuring line
Hips: plan length and angle, true length, slope, extension to the eave, side cuts	The hip as the intersection of two planes; equal pitches at 45° in plan
Jack rafters: lengths and the difference between them	Spacing × the hip's plan ratio × slope factor
Valleys of an intersecting wing	As the hips, at the wing's run

Result	Basis
Plan and roof surface areas, plane by plane	The eave outline clipped where one plane is lower than another; area / cos θ of each plane
Advisory: low slope	Below 2:12: check the roof covering's minimum slope

Limitations of the roof geometry

- Rectangular plans with flat planes; no dormers, crickets, turrets, curved or gambrel roofs. An intersecting wing is at the main pitch and at right angles, and is not drawn on the plan.
- Lengths are on the measuring line; cuts for the heel, birdsmouth, fascia and the rafter depth are not deducted. The ridge deduction is the only one.
- Equal horizontal overhangs on planes of unequal pitch give eaves at different heights; the areas follow the planes as entered.
- The framing-square table values are quoted from knowledge (to confirm against a printed table); the trigonometry is exact.

Verification examples

Verification status: 27 values compared in 7 examples (0 published, 7 hand calculations from the cited equations): 27 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Example	Source	Values
Hip roof 40 ft × 30 ft at 6:12	Hand trigonometry; framing-square table (13.42, 18.00, 17 7/8 in.)	Angle, lengths per foot, common, hip, β, ridge, area, jacks, side cuts
Rafter table at 12:12	Framing-square table (16.97, 20.78 in.)	Common and hip per foot of run
Steeper ends (8:12 and 12:12)	Hand calculation	End run, ridge, hip, area
2 ft overhang	Hand calculation	Area, common rafter, hip to the eave
Gable with eaves and rakes	Hand calculation	Area, ridge, plan area
Valleys of a 20 ft wing	Hand calculation	Valley length
Gable 10 m wide at 30°	Hand calculation (metric)	Rise, common rafter

No published worked example of roof framing geometry is on hand; the framing-square values are as printed on the square, quoted from knowledge.

CHAPTER 248

Shape browser and comparison

Up to six AISC shapes side by side, with every property the AISC Shapes Database v16.0 publishes for them, and a search for the lightest shapes of a family that meet the property minimums you enter. Use it to compare candidate sections before a member design, or to find the lightest W-shape with a required Zx and Ix within a depth, the way Manual Tables 3-2 and 3-3 are used by hand, by any property.

Inputs of the shape comparison

- **Shapes to compare:** up to six AISC designations, US or metric (W21X50 or W530X74, HSS8X8X1/2, C12X20.7, L4X4X1/2, 2L4X4X1/2X3/8, Pipe6STD). The first one is the reference for the ratios.
- **Find the lightest shape:** turns the search on.
- **Family:** W, M, S, HP, C, MC, WT, MT, ST, single and double angles, HSS (rectangular, square and round) or pipe.
- **Minimums:** A, Ix, Sx, Zx, rx, Iy, Sy, Zy, ry, J and Cw; 0 means no minimum. A shape without a published value (Cw of HSS and pipe) does not meet a minimum on it.
- **Largest depth:** d (HSS: Ht; round HSS and pipe: OD); 0 for no limit.
- **Shapes listed:** how many of the lightest are listed; shapes tied at the lightest weight are always all listed.

The shape comparison has no auto-design: its search is the selection.

Checks performed

Result	Basis
Properties side by side, ratios of weight, Zx, Ix and ry to the first shape	AISC Shapes Database v16.0 (Manual Part 1)
Lightest shapes meeting every minimum and the depth limit, by weight	The selection of Manual Tables 3-2 (by Zx) and 3-3 (by Ix), by any property

Limitations of the shape comparison

- Hot-rolled shapes, HSS and pipe of the AISC Shapes Database v16.0 only: no cold-formed, aluminum, built-up or foreign shapes, no availability or cost.
- Properties are nominal, as published (HSS on the design wall thickness, AISC 360-22 B4.2). No strength, slenderness or local buckling is checked: design the shape chosen in its member calculation.
- The double angle drawing uses the gap in its designation (none, 3/8 in. or 3/4 in.).

Verification examples

Verification status: 22 values compared in 6 examples (6 published, 0 hand calculations from the cited equations): 22 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Properties of a W21X50	AISC Manual Table 1-1; Table 3-2, p. 3-25	A, d, Ix, Sx, Zx, rx, Iy, ry, J, Cw
Lightest W with Zx ≥ 100 in³	Manual Table 3-2, p. 3-25	W21X48 (48 lb/ft, 107 in³)
Lightest W with Zx ≥ 90 in³	Manual Table 3-2, p. 3-25	W21X44 (44 lb/ft, 95.4 in³)
Lightest W with Ix ≥ 800 in⁴	Manual Table 3-3	W21X44 (843 in⁴)
Zx ≥ 100 in³ within 18 in.	Manual Table 3-2, p. 3-25	W18X50 (50 lb/ft, 101 in³)
Properties of an HSS8X8X1/2	Manual Table 1-12	Weight, A, Ix, J

CHAPTER 249

Influence lines

Influence lines of a continuous beam for every support reaction and for the moment and the shear at a point you choose, and the largest and smallest effect of a train of point loads and of a uniform load placed where they govern. Use it for crane, vehicle and other moving loads on beams and girders, for pattern loading, and to see how a load anywhere on the beam reaches a support.

Inputs of the influence lines

- **Spans and supports:** 1 to 6 spans, each with the support at its right end: pin or roller, fixed, or none (a free end, or a beam continuous over a removed support). The beam is prismatic; influence lines of forces do not depend on EI, so no stiffness is entered.
- **Point:** where the moment and the shear are wanted, from the left end. The shear is taken just right or just left of it: at a support the two differ by the reaction.
- **Load train:** point loads at fixed distances from the first load (up to 16), run across the beam both ways or one way. Loads are as entered: apply load factors and impact yourself.
- **Uniform load:** a lane load placed only where the influence line is positive (for the largest effect) or negative (for the smallest).
- **Add the train and the uniform load:** both at once, each at its own worst placement; otherwise the larger of the two governs.

The influence lines have no auto-design: there is nothing to size.

Checks performed

Result	Basis
Influence line of each reaction, of the moment and of the shear at the point	Müller-Breslau principle; ordinates by a unit load at every node of one stiffness analysis (Maxwell-Betti)
Largest and smallest ordinate, positive and negative areas	The lines as computed
Train: largest and smallest effect with the position of the first load	Every position with a load on a node, both sides of the shear jump, both ways
Uniform load: w $\int IL^{+}$ and w $\int IL^{-}$	Loading only the parts of the beam of one sign

Reactions are positive upward, moments positive sagging, shears positive with the left part pushed up.

Limitations of the influence lines

- Prismatic beams on rigid supports; no internal hinges, springs or varying EI (use the beam with varying stiffness for a haunched beam under fixed loads).
- Between nodes the lines are interpolated linearly on 48 elements per span; the ordinates at the nodes are exact.
- Loads are vertical and as entered; no impact, factors, dynamic effects or deflection influence lines. The member is not designed.

Verification examples

Verification status: 17 values compared in 5 examples (2 published, 3 hand calculations from the cited equations): 17 within tolerance, 0 recorded deviations. **Thin: only 2 published examples.** Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Propped cantilever, 20 ft	Udoeyo, Structural Analysis (2020), Example 13.1	By at 5, 10, 15 ft, Ay at 5 ft, MA at 10 ft
Two spans of 3 m	Udoeyo Example 13.2	By, Ay and Cy at the quarter points
Two loads on a 40 ft span	Hand calculation	Largest midspan moment, end reaction, shear just right of midspan both signs
Uniform load on two 20 ft spans	Hand calculation, three-moment equation	Support moment, interior reaction, moment at 0.4 l both signs

Udoeyo's Table 13.1 prints its ordinates to two decimals; the exact values of its own equations are used, with the printed ones in the value names. Its moment at C is not used: those printed values do not satisfy the statics of its own reactions.

CHAPTER 250

Arch analysis

Three-hinged, two-hinged and fixed (hingeless) arches of parabolic or circular shape, with the supports at the same level, under vertical and horizontal point loads, uniform loads along the span and the arch's own weight: the reactions, the horizontal thrust, and the moment, normal thrust and radial shear along the arch. Use it for steel, glulam and concrete arches, arched roof ribs and frames, and to see how far a load pattern takes the arch from its thrust line. The forces are for the design of the arch in its material module.

Inputs of the arch analysis

- **Arch:** three-hinged (supports and a hinge at the crown or at a point you give), two-hinged (pinned supports) or fixed.
- **Shape, span L and rise f:** parabolic, $y = 4 f x (L - x) / L^2$, or a circular segment through the supports and the crown (at most a half circle). The geometry is the arch's axis.
- **Section (two-hinged and fixed):** E, A and I, constant or growing as sec θ from the crown ($I = I0 \sec \theta$, the classical assumption behind the textbook formulas). Axial shortening can be neglected, as in those formulas; with it, a shallow arch carries less thrust. A three-hinged arch is statically determinate and needs no section.
- **Loads:** uniform loads per horizontal length over the span or part of it, point loads (P downward, H to the right) and the weight of the arch along its length. One load set, as entered: run the arch once per load case or combination.

The arch analysis has no auto-design: there is nothing to size.

Checks performed

Result	Basis
Reactions, horizontal thrust (end moments of a fixed arch), the moment at the hinge, equilibrium	Statics; redundants from a chord model of the arch (the plane-frame library, first order)
Moment, normal thrust and radial shear at every station, both sides of each point load	Statics of the part left of the section on the true curve: $M = M0 - H y$ (+ end moment), $N = Fx \cos \varphi + Fy \sin \varphi$, $V = Fy \cos \varphi - Fx \sin \varphi$ (Udoeyo 2020, Eqs. 6.3 to 6.5)
Largest positive and negative moment, thrust, shear	Over the stations

Moment is positive with tension inside the arch (on the intrados); the normal thrust is positive in compression.

Limitations of the arch analysis

- Supports at the same level and rigid; no ties, elastic supports, settlement or temperature.

- First-order elastic analysis: no second-order effects, in-plane or out-of-plane buckling of the arch. Check the arch's stability in its member design.
- The redundants of two-hinged and fixed arches come from 64 chords plus a node at every load (their error falls as the square of the chord length); the three-hinged arch is exact.
- No member strength is checked.

Verification examples

Verification status: 23 values compared in 9 examples (3 published, 6 hand calculations from the cited equations): 23 within tolerance, 0 recorded deviations. Code basis: the provisions it uses were read in the target edition.

Example	Source	Values
Three-hinged parabola, point and partial load (metric)	Udoeyo, Structural Analysis (2020), Example 6.2	Ay, By, thrust, normal thrust and radial shear left of the load
Three-hinged parabola, uniform load	Udoeyo Example 6.3	Ay, thrust, moment at 18 ft (zero; the book's −75 lb-ft is rounding)
Three-hinged parabola, two point loads	Udoeyo Example 6.4	Ey, Ay, thrust, moments under the loads
Two-hinged parabola, I sec θ	Hand calculation ($H = \int M_0\, y\, dx / \int y^2\, dx$)	$wL^2/8f$, $wL^2/16f$, $25PL/128f$ and the crown moment
Fixed parabola, I sec θ	Hand calculation (elastic equations)	$15PL/64f$, end moment $PL/32$, crown moment $3PL/64$; $wL^2/8f$
Three-hinged circular arch	Hand calculation	Thrust, moment at the quarter point

The two-hinged and fixed closed forms are derived from the compatibility equations; a textbook that prints them (Timoshenko and Young, Theory of Structures, or a structural analysis text) is not on hand.

CHAPTER 251

Beam grid (grillage)

A grid of beams in two directions, connected where they cross and loaded normal to the plan: the deflection and rotations of every joint, the moment, shear and torque along every beam, and the support reactions. Use it for floor and roof framing where beams of both directions share loads (a two-way grid of girders, a mezzanine, a beam grid under equipment, a framed opening), and to see how much a crossing beam relieves another.

Inputs of the beam grid

- **Spans along x and y:** the lengths of the beams of each direction, end to end. The plan is rectangular and the beams orthogonal.
- **Beams along x and along y:** up to 12 each, at their y (or x) positions, with their moment of inertia I, torsional constant J (AISC Manual Part 1) and a uniform load w along each. Every beam is connected to every beam it crosses.
- **E and G:** steel 29,000 and 11,200 ksi (AISC 360-22); your material's values otherwise.
- **Include torsion:** with it, a beam twists where a crossing beam bends, and its St. Venant stiffness GJ spreads load; without it the beams only cross and share loads by bending. Warping torsion of open sections is never counted.
- **Supports:** the ends of each family: pin (deflection held), fixed (deflection and the bending rotation held) or free (the beam frames into the beams it crosses, for example edge beams). Whether a pinned end also holds the beam's twist is your judgment; fixed ends hold it.
- **Point loads:** downward, at a point of any beam (its x and y); a joint is added there.

The beam grid has no auto-design: size the members in their material modules with these forces.

Checks performed

Result	Basis
Joint deflections and rotations	Grid stiffness method: three unknowns per joint, Hermite beam elements with consistent loads, GJ/L torsion
Moment, shear and torque along each beam, largest values	Element end forces and statics inside each element
Deflection along each beam	End values interpolated, plus the deflection of the element's own uniform load
Support reactions and equilibrium	K u − F at the supports

Deflection is positive downward, moment positive sagging, torque by the right-hand rule along the beam.

Limitations of the beam grid

- Orthogonal beams spanning the full grid, a rectangular plan, supports at the beam ends only; no skew, columns inside the grid, springs or partial beams.
- Linear elastic, first order; shear deformation and warping torsion neglected; the beams are connected rigidly at every crossing (bending of one to twist of the other).
- One load set, as entered; no member strength is checked.

Verification examples

Verification status: 11 values compared in 3 examples (0 published, 3 hand calculations from the cited equations): 11 within tolerance, 0 recorded deviations. **No published example: every value is a hand calculation.** That checks the program against this reading of the code, not the reading itself. Code basis: written without the code document on hand, and confirmed only in part through published examples.

Example	Source	Values
Two simple beams crossing at midspan	Hand calculation (load shared by $48EI/L^3$)	Both moments, deflection
A cantilever stub twisting a fixed beam	Hand calculation ($PL^3/192EI$, $TL/4GJ$, $Pl^3/3EI$)	Tip and midspan deflection, torque, moments
One simple beam, uniform load	Hand calculation	$wL^2/8$, $wL/2$, $5wL^4/384EI$

No published grillage example is on hand; the cases are hand calculations that each test one action (bending share, torsion, element load).

CHAPTER 252

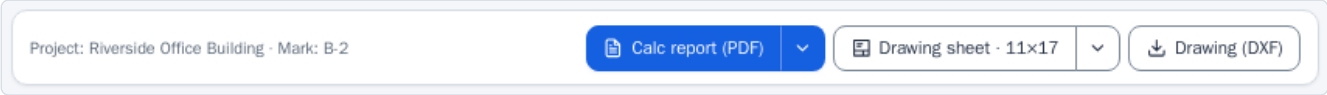
Reports and settings

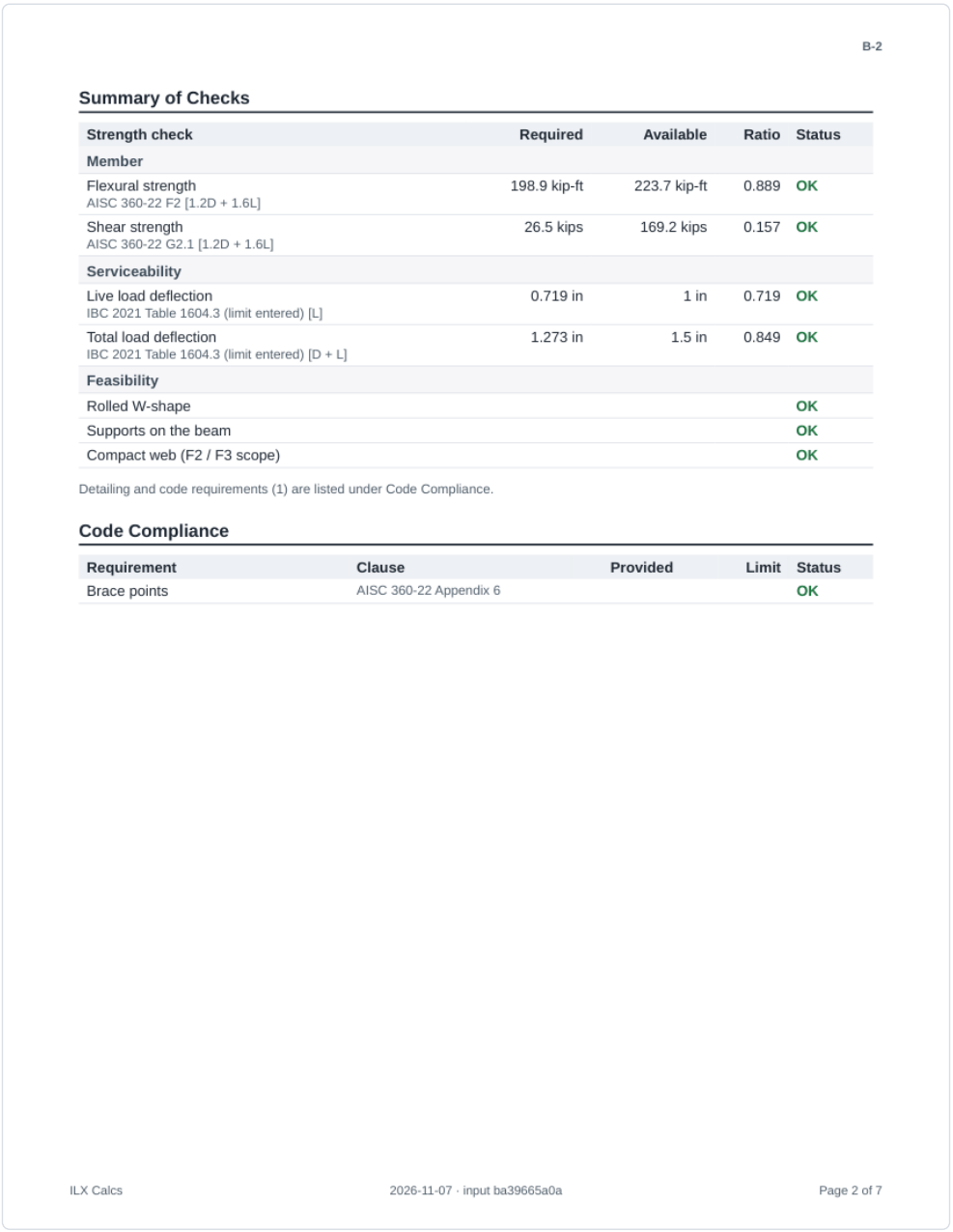
A walk-through with pictures is in Training 2.

Calculation report

Calc report (PDF) creates the report: a first page with the project, the calculation and its status, then the inputs, the loads and combinations, every check with its equations and code clauses, and the diagrams. The arrow beside it offers **Summary report**, **Full report** and **Customize this report**, which changes what this one report includes. In a project's menu, **Calc package (PDF)** puts its calculations in one PDF.

Drawing sheet creates the sheet as a PDF on the chosen size, and **Drawing (DXF)** the same sheet for CAD; **All sheets (PDF)** in a project's menu makes the set. Load chasedown, load combinations, dead load build-up and the unit converter make no sheet.





B-2

Summary of Checks

Strength check	Required	Available	Ratio	Status
Member				
Flexural strength AISC 360-22 F2 [1.2D + 1.6L]	198.9 kip-ft	223.7 kip-ft	0.889	OK
Shear strength AISC 360-22 G2.1 [1.2D + 1.6L]	26.5 kips	169.2 kips	0.157	OK
Serviceability				
Live load deflection IBC 2021 Table 1604.3 (limit entered) [L]	0.719 in	1 in	0.719	OK
Total load deflection IBC 2021 Table 1604.3 (limit entered) [D + L]	1.273 in	1.5 in	0.849	OK
Feasibility				
Rolled W-shape				OK
Supports on the beam				OK
Compact web (F2 / F3 scope)				OK

Detailing and code requirements (1) are listed under Code Compliance.

Code Compliance

Requirement	Clause	Provided	Limit	Status
Brace points	AISC 360-22 Appendix 6			OK

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Settings

Settings belong to you, and a project can override them. **My defaults** apply to every project; choose a project in the scope bar to override a field for that project only (an overridden field shows **Project** and can be reset). **Copy from project**, **Make these my defaults** and **Reset project** work on a project's overrides. No setting is global to the server.

Title block

Firm name, address, phone, web, license, logo text and the engineer of record; the default sheet size, the sheet number prefix, the status, drawn, checked and approved by, the border zones and the general notes. `{codes}` in a note prints the codes and editions the sheet's calculation cites.

Report

Presets, the sections and their order, the level of detail, units (including dual units), headers and footers, signatures and the disclaimer. **Preview report** shows the result before you save.

Design

Design defaults for your calculations. **Save as project defaults** in the editor stores the materials, methods and limits of one calculation as the starting values for new calculations of the same type in that project.

Verification and user manual

Verification is the record described in Verification. **User manual** opens this manual; so do **?** in the header and F1.

CHAPTER 253

Verification

How ILX Calcs is verified

Every calculation type is tested, on every change, against published design examples and code tables where they exist, and against hand calculations from the code equations where they do not. Each compared value (a required strength, an available strength, an intermediate quantity such as Cb) is an automated test: the app's value must match within a stated tolerance, usually 1 %, with a small allowance for rounded published values and more for values the source read from a chart.

This release: 9,654 values compared in 2,841 examples, 2,143 of them published and 698 hand calculations from the cited equations: 9,254 within tolerance, 400 recorded deviations.

Settings > Verification lists every example and value with the app's result and the difference. A value the app computes differently is listed as a documented deviation with its reason, and the same record gives a PDF report. How to read it, with real examples: Training 17.

How far each type goes

Each chapter's **Verification examples** section starts with the status of that type: the values compared, how many of its examples are published and how many are hand calculations, how many values match and how many are deviations, and the basis of the code text. A hand calculation checks the program against this reading of the code, not the reading itself.

No published example, hand calculations only (51): AAC masonry wall, Atmospheric ice loads, Beam grid (grillage), Bond beam chord and collector, Built-up cold-formed steel member, Cold-formed steel bypass and infill framing, Cold-formed steel ceiling framing, Cold-formed steel clip angle, Cold-formed steel floor vibration, Cold-formed steel girt, Cold-formed steel jamb studs, Cold-formed steel parapet, Cold-formed steel shear wall chord studs, Cold-formed steel sloped and mansard wall, Cold-formed steel soffit framing, Cold-formed steel special bolted moment frame, Cold-formed steel storage rack, Cold-formed steel track, Cold-formed steel truss bracing, Concrete stair, Corbelled masonry, Curved and tapered glulam, Dapped-end beam, Fall-protection anchorage, Fluid and hydrostatic pressure, Freestanding masonry wall, Gabion retaining wall, Gravity retaining wall, Helical pile, Joist and waffle slab, Laterally loaded pile (p-y), Ledge and inverted-tee beam, Liquefaction triggering, Liquid-containing tank wall, Masonry arch, Masonry bearing, Masonry chimney, Masonry parapet, Micropile, Modal response spectrum, Movement joints, Notched and tapered member ends, Pile capacity from CPT, Prestressed concrete beam, Roof geometry, Roof ponding, Segmental retaining wall, Seismic member checks, Special moment frame members, Wood shrinkage and moisture, Wood stair.

Thin, 2 published examples or fewer (36): Basement (restrained) retaining wall, Beam with varying stiffness, Braced excavation, Built-up and spaced columns, Cable and catenary, Cold-formed steel track anchorage, Combined footing, Concrete deflection, Continuous concrete beam, Corbel, Coupling beam, Glulam and SCL beam, Ground improvement, Guard or barrier post, Gust-effect factor, Impact and machinery loads, Influence lines, Ledger to masonry wall, Load chasedown, Masonry beam and deep beam, Masonry partition wall, Mat foundation, Monorail

beam, One-way slab, Plate girder, Post-tensioned slab strip, Prestressed masonry wall, Seismic site parameters and spectrum, Sheathing-braced cold-formed steel stud, Slope stability, Soldier pile wall, Steel side and knife plate connections, Strap footing, Stress transformation, Unreinforced masonry wall, Wood floor vibration.

The code basis

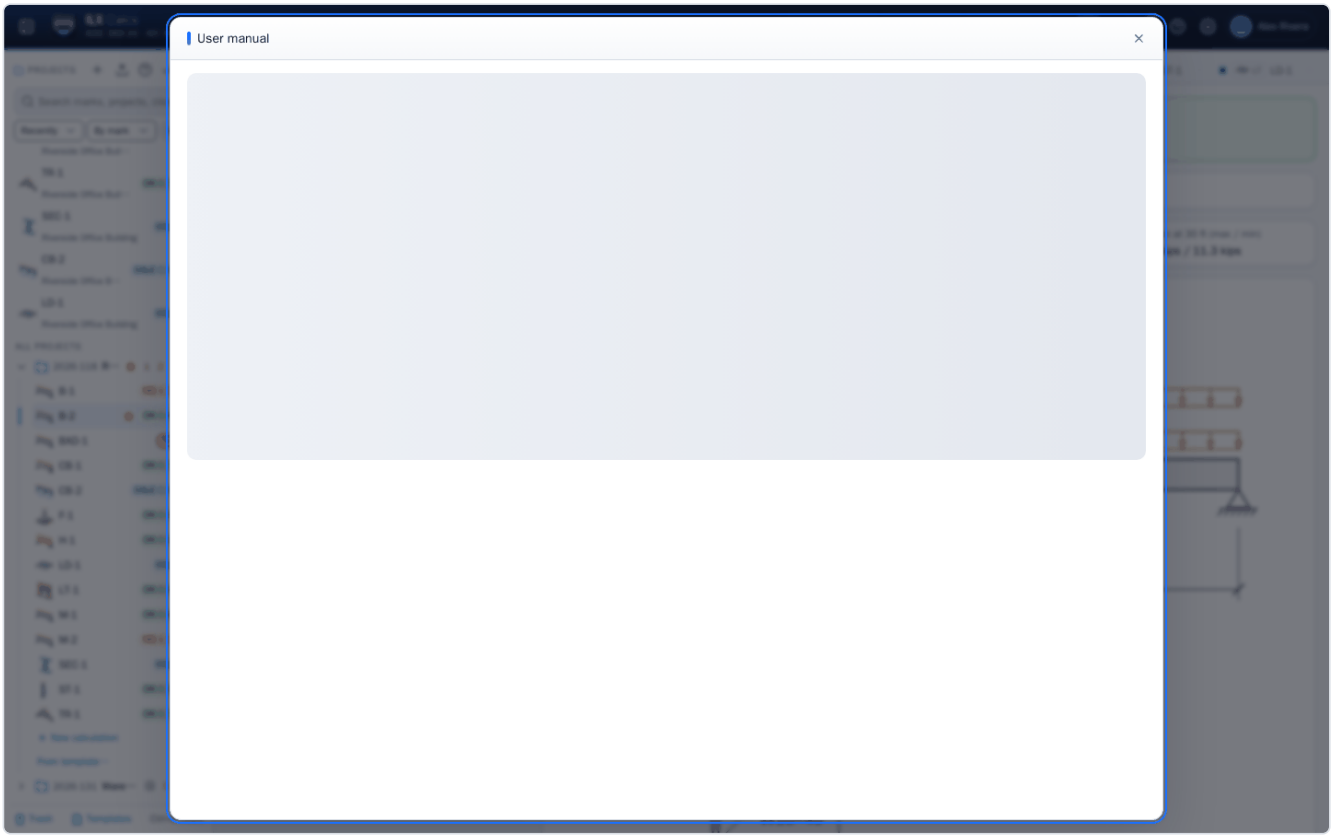
The target editions are listed in the overview. Not every module has been read line by line in its target edition: some were written from older editions, summaries or without the document on hand, and each chapter says so. A reference followed by "(to confirm against the code text)", on the screen, in a report or here, is one this version has not yet confirmed against the code: read that clause yourself. ILX Studio plans to obtain the purchased references once the module list is complete, and to verify the whole system against them.

What this does not do

Verification shows that the program reproduces certain results. It does not show that a calculation suits your structure, and it does not replace your check. See Limits and your responsibility.

CHAPTER 254

Keyboard shortcuts and troubleshooting



Keyboard shortcuts

Keys	Action
F1	Open this manual
Ctrl+Z / Ctrl+Y	Undo / redo in the open calculation
Ctrl+B	Show or hide the project panel
F2	Rename the selected calculation (project panel)
Del	Delete the selected calculation (project panel)

Troubleshooting

- **A field is outlined in red.** The value is not accepted; the message under the field says why.
- **"Not feasible".** The inputs describe something that cannot be calculated (for example a support off the beam). The banner lists each problem with **Show**, which takes you to the input.

- **The results look faded.** The calculation is updating; it finishes within a moment.
- **The status is red right after I picked a card.** A few cards start not adequate on purpose: a cut wood stair stringer reads the NDS notch limit literally, and a monorail beam waits for its allowable wheel load. The check says why.
- **A red banner reads "Preview: not yet verified".** That is the truss designers: see Training 16.
- **My account says the trial ended.** The trial lasts 30 days. Your account then stays read-only, with your projects in it; **Subscribe** is in the account menu.
- **Something else.** Use **Support** in the header to ask; include the calculation so we can see what you see. We answer within 24 hours on business days. You can also write to services@ilxstudio.com.



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