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 v0.1.0.

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Version 0.1.0 · November 2026

ILX Calcs User Manual

Version 0.1.0, November 2026. Covers the ILX Calcs web application: AISC 360-22 Specification for Structural Steel Buildings and the AISC *Steel Construction Manual*, 16th Edition.

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 is the web edition of the ILX Calcs structural calculation tools: quick, documented member and element calculations for steel, wood, concrete, foundations, masonry, cold-formed steel and lateral loads, in one place. Every calculation shows its equations with the code clause, the governing load combination, a pass/fail status for each check, diagrams, and a calculation report you can print or save as PDF.

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.

Calculation types

Calculations are grouped by material. This version has:

Group	Calculation	Chapter
Steel	Steel beam (AISC 360-22)	Steel beam

More types are added in each version; the New calculation window lists the ones this server has.

Code basis

- AISC 360-22 *Specification for Structural Steel Buildings* and the AISC *Steel Construction Manual*, 16th Ed.
- ASCE/SEI 7-22 load combinations (Chapter 2).
- IBC 2021 Table 1604.3 deflection limits (values you enter).

How each calculation was checked against published worked examples is in Verification.

CHAPTER 2

Getting started

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, Lateral 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. 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 SI) 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.

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.

- **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.

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

Steel beam

A rolled W-shape beam in major-axis bending and shear, to AISC 360-22 Chapters F and G, with ASCE 7-22 load combinations. One model covers simple spans, cantilevers, overhangs, propped and fixed beams and continuous beams.

Inputs of the steel beam

- **Member:** the W-shape (AISC Shapes Database v16.0), the steel (A992, A572 Gr. 50, A913 Gr. 65, A36) and whether the self-weight is added to the dead load.
- **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.
- **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 W-shape (lightest first, optional depth limits); steel grade when unlocked. Objectives: lightest (steel weight); shallowest, then lightest. Kept unless you select them: steel. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Flexure, each unbraced segment and combination	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)
Shear	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)
Live load deflection	Each of L, Lr, S and R alone (service), against span / n
Total load deflection	The ASD gravity combinations (service), against span / n

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

- Doubly symmetric rolled W-shapes with compact webs (F2, F3). A shape whose web is not compact (F4, F5) is reported as not feasible.
- Major-axis bending only; no axial load, torsion, web openings, concentrated-force checks (J10) or bearing.
- Lateral-torsional buckling uses the unbraced segments between brace points; bracing strength and stiffness (Appendix 6) are the engineer's responsibility.

Verification examples

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

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.

CHAPTER 5

Steel column

A W-shape, rectangular or square HSS, round HSS or pipe member in axial compression or tension with bending about both axes 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.

Inputs of the steel column

- **Member:** the shape (AISC Shapes Database v16.0: W, HSS, pipe) and the steel. W-shapes take A992, A572 Gr. 50, A913 Gr. 65 or A36; HSS take A500 Gr. C (Fy = 50 ksi), A500 Gr. B (46 ksi) or A1085 (50 ksi); pipe takes A53 Gr. B (35 ksi). A steel that does not suit the section is reported as not feasible.
- **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 $|Mtop - Mbot|/L + 4|Mmid|/L$).
- **Second-order effects:** either the moments come from a second-order analysis, or the approximate method of Appendix 8 is applied: $B1 = Cm/(1 - \alpha Pr/Pe1) \geq 1$ on the no-translation moments, with Cm from the end moments (Eq. A-8-4) or 1.0 when there is transverse loading, $Pe1 = \pi^2 EI^*/L^2$ (K1 = 1.0) with $EI^* = 0.8\tau bEI$ 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:** 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 $\sqrt{(1 + \alpha Pr/Pey)}$ under axial tension can be switched off.
- **Tension end connection:** no holes and all elements connected (Ae = Ag), or the net area An (B4.3b) and the shear lag factor U (Table D3.1) entered.

Auto-design (editor toolbar) varies the shape of the given family (W, rectangular HSS, round HSS or pipe), lightest first, optional depth limit; steel grade when unlocked. Objectives: lightest (steel weight); smallest depth, then lightest. Kept unless you select them: steel. Every other input is kept; see Auto-design.

Checks performed

Check	Basis
Compression	E3 flexural buckling about x and y (Eqs. E3-2 to E3-4); E4 torsional buckling of W-shapes (Eq. E4-2); E7 effective area for slender elements (E7.1 with Table E7.1 c1, c2; E7.2 for round HSS)

Check	Basis
Tension	D2 yielding (Fy Ag, φ = 0.90) and rupture (Fu Ae, φ = 0.75)
Major-axis flexure	W: F2 / F3 (yielding, lateral-torsional buckling, flange local buckling); rectangular HSS: F7 (yielding, flange and web local buckling, lateral-torsional buckling); round HSS and pipe: F8
Minor-axis flexure	W: F6; rectangular HSS: F7; round: F8
Shear, both axes	W: G2.1 (major), G6 (minor); rectangular HSS: G4; round HSS and pipe: G5 (Lv = L/2 with transverse load, else L)
Combined force	H1.1 Eqs. H1-1a / H1-1b in compression, H1.2 in tension, per load combination
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 (W-shape, rectangular HSS or round HSS / pipe) with its dimensions. 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

- W, rectangular / square HSS, round HSS and pipe only. Channels, angles, tees and built-up sections are not covered. A W-shape whose web is not compact for flexure (F4 / F5) is reported as not feasible.
- 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

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
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

Table	Shape, Lc	Published	ILX Calcs
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.

CHAPTER 6

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: Pp = 0.85 f'c A1 √(A2/A1) ≤ 1.7 f'c A1, φc = 0.65, Ωc = 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; Mn = Fy tp²/4 (F11) and tp,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

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 7

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

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 8

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

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 9

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:** 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). Live load patterning on continuous beams is not automatic: enter the partial loads that govern.
- **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).

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 NDS 2024 Supplement (design value tables) is not on disk: the reference values of Tables 4A, 4B (Southern Pine No. 2 only; other Southern Pine grades are entered as reference values) and 4D, the Table 4A / 4B size factors, flat use factors and wet service factors are those of the 2018 Supplement and stay marked [verify] (listed in `research/DOCUMENTS_NEEDED.md`). 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.
- Notched ends: see Wood bearing and notches. Fastening of plies, lateral bracing design, vibration and fire design are the engineer's.

Verification examples

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

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 10

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

- **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 (Table 5A footnote) stay marked [verify]: the NDS 2024 Supplement is not on disk (`research/DOCUMENTS_NEEDED.md`). 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

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 11

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 reference values (Supplement Tables 4A, 4B, 4D, 5A) stay marked [verify]: the NDS 2024 Supplement is not on disk (`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

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
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

Example	What it checks	Published / hand value	ILX Calcs
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 12

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).
- **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)
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	$5w\ell^4/(384\ E'\ I)$ with the reduced wind load, against height / n

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

- Openings, headers, jack and king studs, 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).
- 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 $F_{c\perp}'$ takes no λ in LRFD (Table 4.3.1).
- Reference values and the Table 4A / 4B size factors stay marked [verify]: the NDS 2024 Supplement is not on disk (`research/DOCUMENTS_NEEDED.md`). Stud grade wider than 6 in and Southern Pine grades other than No. 2 are reported as not feasible.

Verification examples

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 $F_{c\perp}'$ = 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
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 $F_{c\perp}'$ = 781 psi, Δ = 0.108 in	same

CHAPTER 13

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 [verify] 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

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
WW-V1-5d	Same panel with the table values entered	ΔST = 0.159 in, ΔT = 0.477 in	0.1589, 0.4766

Example	What it checks	Value	ILX Calcs
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 14

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 itself is not yet on file at ILX Studio (AWC offers it view-only). The SDPWS 2021 design factors (4.1.4) and a part of Table 4.3A (vn and Ga of 15/32 in rated sheathing with 8d and 10d nails and of 15/32 in Structural I with 8d nails) are confirmed by the free published examples listed under Verification examples; the other table rows and the provisions marked "[verify]" in the steps will be re-checked against the document. Where you need certainty now, choose **Nominal unit shear: Entered** and type the values from your copy of the table.

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: vs, vw and Ga of the whole wall (both faces), before the species factor; the seismic check uses vn = 1.4 vs (SDPWS 2021 tabulates one vn equal to the former vw).
- **3/8 and 7/16 in panels:** SDPWS Table 4.3A 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: in proportion to each segment's adjusted capacity, or to its stiffness (equal top deflection by Eq. 4.3-1, with the anchorage slip). For a perforated wall enter the total length Ltot and the openings (width × height).
- **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, wind and seismic) or, for a project still under SDPWS 2015, 2bs/h for wind and seismic (as the AWC 2015/2018 design

examples apply it); with the 2015 rule a perforated wall also counts each segment over 2:1 with 2bs/h of its length.

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 4.3.4, Table 4.3.4 (blocked wood structural panels)
Unit shear, wind and seismic	v = load factor × V × share / bs against the unified nominal capacity vn (Table 4.3A) × species factor × 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 (SDPWS 2021 4.1.4; vn equals the SDPWS 2015 vw, so vn / 2.8 ≈ vs / 2.0); both faces add, a seismic wall with dissimilar faces takes the larger of the stronger face and twice the weaker face (4.3.3.2)
Perforated wall	Co = r / (3 − 2r) × Ltot / ΣLi, r = 1 / (1 + Ao / (h ΣLi)); v = V / (Co ΣLi); one face vn ≤ 2,435 plf; height ≤ 20 ft (4.3.5.3). With the SDPWS 2015 aspect ratio rule, segments over 2:1 count with 2bs/h of their length in ΣLi (also in r and Co), as in AWC example E3.4
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); perforated walls T = Vh / (Co ΣLi) with no dead load; 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
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 or 10d at 3 in; joints offset on two-faced walls; nailing assumed by the table; 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.
- Force transfer around openings (4.3.5.2) is not covered; a wall with openings is either segmented or perforated.

- 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, collectors, the foundation.
- Confirmed from published examples (2026-11-01): the SDPWS 2021 factors 2.0 / 2.8 and 0.80 / 0.50 (4.1.4), vn of 15/32 in Structural I 8d at 3 in (1,540 plf, Ga 24 kips/in OSB), of 15/32 in rated sheathing 10d at 6 / 4 / 2 in (Ga 22 / 30 / 52 kips/in OSB) and 8d at 6 and 4 in, Eq. 4.3-1, Cd δ / Ie drift, and the SDPWS 2015 2bs/h rule for wind as well as seismic. Still [verify] until SDPWS 2021 is on file: the other Table 4.3A rows, the footnote numbers (species, Ga and framing), the dissimilar-faces rule (4.3.3.2), the SDPWS 2021 aspect ratio factor 1.25 − 0.125 h/bs (4.3.4.2), the SDPWS 2021 perforated wall equations, ΣLi and limits (4.3.5.3), the detailing rules (4.3.7).
- 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

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

Example	What it checks	Published value	ILX Calcs
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).

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 (SDPWS 2015 vs = 860 plf), Ga = 20 kips/in.

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 = 0.467 in, Δ = 1.87 in	625, 602.5, 7.63, 0.467, 1.87
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	8 ft share 0.753, v = 565 plf	0.753, 564.7
SW-3b	8 ft + 3 ft by stiffness (Δa = 0)	3 ft share 0.255	0.255
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	seismic 0.50 × max(1,205, 2 × 785) = 785 plf; wind 0.80 × 1,990 = 1,592 plf	785, 1,592
SW-6	Story above (QE = 3 kips at 19 ft)	T = (75 − 7.84) / 7.5 = 8.95 kips	8.95

CHAPTER 15

Wood diaphragm

A wood structural panel diaphragm to AWC SDPWS 2021 Section 4.2, spanning simply between two shear wall lines under a uniform wind or seismic load, with the ASCE 7-22 load factors: unit shear, chord force, collector (drag) force, aspect ratio and deflection.

> **Source note.** SDPWS 2021 itself is not yet on file at ILX Studio (AWC offers it view-only). The SDPWS 2021 design factors (4.1.4), Eq. 4.2-1 and part of Tables 4.2A / 4.2C are confirmed by the free published examples listed under Verification examples; the other values and the provisions marked "[verify]" in the steps will be re-checked against the document. The apparent shear stiffness Ga is an input (transcribe it from Table 4.2A 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 $1 - (0.5 - G) \le 1.0$). **Entered** replaces the table values (vs and vw as in SDPWS 2015; the seismic check uses vn = 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).
- **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 $\rho = 1.0$ for diaphragms (12.3.4.1). $\Omega 0$ 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 4.2.2 leaves the permissible deflection to the engineer).

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 4.2.4, Table 4.2.4: 4:1 blocked, 3:1 unblocked

Check	Basis
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; vn equals the SDPWS 2015 vw, so vn / 2.8 ≈ vs / 2.0)
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	Eq. 4.2-1: 5vL³ / (8EAb) + 0.25vL / (1000 Ga) + Σ(x Δc) / 2b, under the strength-level seismic load and the serviceability wind
Detailing notes	Nail penetration, boundary nailing by case, 3 in framing with staggered nails at 2-1/2 or 2 in

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

- One simple span between two shear wall lines, uniform load, symmetric reactions. Cantilever and open-front diaphragms (4.2.5.2), openings in the diaphragm, continuous or multi-span diaphragms and concentrated loads are not covered.
- High-load blocked diaphragms (Table 4.2B) and lumber sheathing (4.2.8) are 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 flexibility classification of the diaphragm (ASCE 7-22 12.3.1) and the distribution to the walls are not done here.
- Confirmed from published examples (2026-11-01): the SDPWS 2021 factors 2.0 / 2.8 and 0.80 / 0.50 (4.1.4); vn = 1.4 vs rounded to 5 plf; the blocked rows of 7/16 in (3 in framing), 15/32 in (8d and 10d) and 19/32 in (10d) rated sheathing (SDPWS 2015 Table 4.2A as reproduced by WoodWorks) and vn = 895 plf (19/32 in, 10d at 6 in); vn = 505 plf of 3/8 in Structural I unblocked, 8d, 2 in framing, cases 2 to 6 (Table 4.2C); Eq. 4.2-1 including the chord splice term. Still [verify] until SDPWS 2021 is on file: the other rows, the footnotes, Table 4.2.4.
- 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

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 examples themselves are outside the module's scope (simple spans only); only their capacities are compared.

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

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 2015 vs = 580 plf).

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

CHAPTER 16

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

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

Example	What it checks	Published (Im / Is / II / IIIm / IIIs / IV; Z) lb	ILX Calcs
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 17

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	$f_{c\perp} = R/(b_w l_b) \le F_{c\perp}' = F_{c\perp}$ CM Ct Ci Cb (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 reference values (Supplement Tables 4A, 4B, 4D, 5A) stay marked [verify]: the NDS 2024 Supplement is not on disk (`research/DOCUMENTS_NEEDED.md`).

Verification examples

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
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 18

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), and the area of longitudinal torsion bars provided in addition to the flexural bars.
- **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)	Threshold Table 22.7.4.1(a); closed stirrups; section limit Eq. 22.7.7.1a; Av/s + 2At/s with At/s by Eq. 22.7.6.1a (θ = 45°, Ao = 0.85Aoh); Al by Eq. 22.7.6.1b 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 rectangular web with closed stirrups, θ = 45°; overhanging flanges are neglected in Acp and pcp (conservative for Tth and Tcr); the entered Tu is applied at the critical section for shear; all stirrup legs are taken to resist shear and the outer legs torsion. Hollow sections and prestressed members are not covered.
- 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) [verify against the latest errata].
- Skin reinforcement and flange distribution of negative-moment bars (24.3.4) are advisories, not designed.

Verification examples

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
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-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

Example	What it checks	Hand calculation	ILX Calcs
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

CHAPTER 19

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) [verify 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

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 20

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 [verify: 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

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 21

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).

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

Check	Basis
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 only (no equivalent frame or 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).
- The direct design method text is that of ACI 318-14 8.10 [verify: 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) [verify: 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

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)
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

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 22

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 (marked [verify] in the source): 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

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
W-10	Eq. 11.6.2 minimum vertical steel (not below 0.0025)	0.300 in.²/ft	0.300

CHAPTER 23

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

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
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

Example	What it checks	Published	ILX Calcs
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 24

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

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
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

Example	What it checks	Published	ILX Calcs
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 25

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

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

What it checks	Hand	ILX Calcs
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 26

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

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 27

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

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 28

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		

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
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); 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

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
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 29

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 design basis is TMS 402-22 / TMS 602-22. On 2026-11-01 the module was checked provision by provision against TMS 402-13 / TMS 602-13 (code, commentary and specification), the edition on hand. Section numbers quoted below are those of TMS 402-13 unless noted; TMS 402-16 renumbered parts of Chapters 7 and 9. The items that may differ in the 2022 edition 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 (9.1.9.1.1); 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.
  - Whether the design uses R ≤ 1.5. That allows the 9.3.3.5.4 exemption from the maximum reinforcement where Mu/(Vu dv) ≤ 1. Without it, a wall with Mu/(Vu dv) < 1 is checked with α = 1.5 whatever its type (9.3.3.5.4 sends it to 9.3.3.5.1).
  - 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	9.3.4.1.1, Eqs. 9-19 / 9-20 (9-11 / 9-12 in 402-22): 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 = 0.90 (9.1.4.4)
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 = 0.90
In-plane shear	9.3.4.1.2, 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 = 0.75 for partial grout; φ = 0.80 (9.1.4.5)
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	9.3.3.5: 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 = 1.5 (ordinary), 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 (9.3.3.5.4)

Check	Basis
Special boundary elements (note)	9.3.6.5 (9.3.6.6 in 402-22): 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	9.3.3.1: diameter ≤ nominal thickness / 8; No. 9 or smaller
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 (9.3.5).
- Shear friction (sliding) at the base.
- The special boundary element alternative to the maximum reinforcement (9.3.6.5 in 402-13, 9.3.6.6 in 402-22): 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 cell limits (1/4 of the clear cell, 4 % of the cell area) 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 9.3.3.5 does).
- The bar layout is symmetric.

Checked against TMS 402-13 on 2026-11-01. These were confirmed in TMS 402-13 / 602-13 and are, to our knowledge, unchanged in the 2022 edition:

- 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 = 0.90 for flexure and axial load, 0.80 for shear (9.1.4.4, 9.1.4.5);
- the axial strength equations, with tied bars only (9.3.4.1.1, 9.3.2(e));
- the shear equations and the 6 / 4 Anv √f'm limits (9.3.4.1.2);
- the bar size limits (9.3.3.1);
- 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 9.3.6.5.1 conditions.

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 (marked [verify] in the code and the check references):

- the section and equation numbers, renumbered since 2013 (for example the maximum reinforcement is 9.3.3.2 in 402-16, the boundary elements 9.3.6.6, the axial strength Eqs. 9-11 / 9-12, and Chapter 7 was reorganized);
- the shear strength of partially grouted walls and γg = 0.75, which the 2022 edition may have revised;
- the α rule and the axial load of the maximum reinforcement;
- 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

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, 163.2, 130.6
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, 198.6, 236.8, 158.8
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, c at the maximum reinforcement	18.6, 685 in.², 0.625, 91.9, 56.2 kips, 83.8 in.	18.6, 685, 0.625, 91.9, 56.2, 83.9
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, 102.9, 100, 0.00096 / 0.00169 / 0.00265, 83.9

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

Example	What it checks	Hand calculation	ILX Calcs
MW-1	Maximum reinforcement, α = 3	Cm + Cs − T = 257.8 kips	257.8
MW-2	Partially grouted	An = 322 in.², φVn = 26.6 kips (γg = 0.75), Pn = 388.6 kips	322, 26.6, 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

Not yet compared with a published example: the axial strength of 9.3.4.1.1 (Eqs. 9-11 / 9-12) on its own, and a published TMS 402-22 example. Both examples above use earlier editions (402-13 and 402-16), whose shear, flexure and maximum reinforcement provisions the module applies unchanged.

CHAPTER 30

Cold-formed steel stud

A single lipped C-section (SSMA stud or joist) as a simply supported member: a wall stud under axial load and out-of-plane wind or seismic load, or a floor or roof joist. 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:** only unpunched webs are covered.
- **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 not computed: the engineer states that it is restrained (sheathing attached to both flanges) or not critical.
- **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.
- **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
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
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

- Distortional buckling (E4 / F4) and the Direct Strength Method are not covered; the engineer confirms that distortional buckling is restrained or not critical, or the module reports *not feasible*.
- Webs with holes or punchouts (Appendix 1 1.1.1, G3, the G5 hole factors) are not covered (*not feasible*).
- 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). Built-up members are not covered.
- 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

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

Example	What it checks	Published (SSMA)	ILX Calcs
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. The SSMA axial and combined-load tables include distortional buckling with kφ = 0, which this module does not compute, so compression (E2 / E3) and the interaction (H1.2) are not compared with a published value yet; the AISI Cold-Formed Steel Design Manual (D100) column examples would close that.

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 31

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

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 32

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). 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), 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).

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
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, flexible buildings (n1 < 1 Hz), roof slopes of 90°, C&C for h > 60 ft (Ch. 30 Part 2), roof C&C above 45° or in zones the 7-22 figures do not have, without entered GCp.
- Not covered and not checked: the envelope procedure (Ch. 28), other structures and building appurtenances (Ch. 29: rooftop equipment, signs, solar panels), parapet loads (27.3.4, 30.8) and roof overhangs (30.7), monoslope, stepped, multispan, mansard and domed roofs, tornado loads (Ch. 32), the reduction factor Ri for large partially enclosed buildings (Ri = 1.0 is used, 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

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
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 %)

Example	What it checks	Hand calculation	ILX Calcs
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

CHAPTER 33

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 [verify: ASCE 7-22 errata or supplement for the 11.6 cross-reference].

Verification examples

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
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

Example	What it checks	Published / hand value	ILX Calcs
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 34

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

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 35

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

Closed-form beam formulas (hand calculation), one unfactored load set, E = 29,000 ksi, I = 1,000 in⁴, 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 36

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.

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.

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	A note: P-Δ and P-δ are not included (AISC 360-22 C1, C2, Appendix 8)
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. Member design, connections and bases are not part of the calculation. Member moments are positive with tension on the member's local −y side (the bottom of a member drawn left to right).

Limitations

- First-order linear elastic analysis: second-order effects (P-Δ, P-δ), geometric and material nonlinearity are out of scope. For stability design amplify the forces (AISC 360-22 Appendix 8) or use a second-order analysis.
- No shear deformation, 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.
- No member design: use the forces in the member's material calculation.

Verification examples

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
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, SI 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

Example	What it checks	Published	ILX Calcs
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.

CHAPTER 37

Section properties

Geometric properties of a cross-section: area, centroid, moments and product of inertia, principal axes, elastic and plastic section moduli, radii of gyration and the torsion constant, for common shapes and for sections built up from rectangles. The tool has no design code and no LRFD / ASD choice; the values feed any member calculation.

Inputs of the section properties

- **Shape:** rectangle, circle, hollow rectangle, hollow circle (tube or pipe), I-shape from plates, T-shape, channel, angle, or built up from rectangles.
- **Dimensions** (only those of the chosen shape are shown):
  - rectangle: width b and height h; circle: outside diameter D;
  - hollow rectangle: width, height, wall thickness t and the outside corner radius (0 = sharp corners; 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);
  - tube: D and t;
  - I-shape: depth d, flange width bf and thickness tf, web thickness tw, the web-to-flange fillet radius, and with *Equal flanges* off a different bottom flange (width and thickness);
  - T-shape and channel: d, bf, tf, tw and the fillet radius (a channel's flanges are taken with a constant thickness, so for a rolled channel use the average flange thickness);
  - angle: the vertical leg h, the horizontal leg b, the thickness t and the inner-corner fillet radius;
  - built up: a table of rectangles, each solid or a hole, by width, height and centre (x, y). Solid rectangles must not overlap and each hole must lie inside one solid rectangle.
- **Unit weight:** only for the weight per length (steel 490 pcf, concrete 150 pcf).

For a rolled W-shape the fillet radius is kdes − tf from the AISC Shapes Database.

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.

Result	Basis
A, $\bar{x}$, $\bar{y}$	Polygon integration (Green's theorem) over the outline; holes count negative
Ix, Iy, Ixy	Second moments about the centroid (parallel-axis transfer of the integrals)
I1, I2, θ, r2	Principal axes (Mohr's circle), shown when Ixy ≠ 0, as for an angle; r2 is the least radius of gyration (rz)
Sx top / bottom, Sy left / right	I / c to each extreme fibre
Zx, Zy	Plastic moduli about the equal-area axes (found by bisection of the area)

Result	Basis
rx, ry	√(I / A)
J	Exact for circles and tubes; Roark's series for a solid rectangle; Bredt's 4 Am² t / pm on the mean line for the hollow rectangle; Σ b t³ / 3 on the centre-line lengths for I, T, channel and angle (fillets ignored); the sum of the rectangles for a built-up section
Area check	The integrated area against the closed-form area of the shape (≤ 0.01 %): curved edges are divided into chords, 720 per full circle
Section dimensions	The parts must fit together (thickness against the size, fillets between the web and the flange tips, no overlapping rectangles); otherwise NOT FEASIBLE

The drawing (on screen, in the report and on the PDF / DXF drawing sheet) shows the section to scale with the solid parts and holes as integrated, the centroid, the centroidal x and y axes and, when Ixy ≠ 0, the principal axes 1 and 2, the overall dimensions with the centroid located from the left and bottom edges, and the thickness callouts of the shape. The sheet's design summary lists A, Ix, Iy, Sx, Zx, Zy, rx, ry and J (and I1, I2, θ when Ixy ≠ 0).

Limitations

- Homogeneous sections only (no transformed sections of two materials; enter a transformed width yourself).
- J of open sections ignores the fillets, so it is a little below the AISC tabulated value of a rolled shape; J of a built-up section is the sum of its rectangles and does not apply to closed cells or to plates with holes.
- No warping constant Cw, shear centre, or effective (reduced) section for slender elements.
- Plastic moduli assume the whole section yields at one stress (Mp = Fy Z).
- Channels and the rolled flange-tip rounding are modelled with flat plates.

Verification examples

Closed-form hand calculations and the AISC Shapes Database v16.0 (tests/test_verify_section_properties.py):

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	490.86, 166.66
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	9.74, 71.7, 23.6, 23.5, 61.1

The W14X90 values are within 0.7 % of the database, which also includes the rolled flange-tip rounding.

Published section properties of the AISC Shapes Database v16.0, each shape built from its tabulated dimensions (W and WT: fillet radius kdes − tf; HSS: tdes with outside corner radius 2 tdes; L: heel fillet kdes − t):

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, 1.75*
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
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	A = 5.87, Ix = 78.9, Sx = 15.8, Zx = 19.4, Iy = 2.80	5.85, 78.7, 15.74, 19.32, 3.51*
SP-18	L4X4X1/2	A = 3.75, Ix = 5.52, y = 1.18, Iz = 2.25	3.78, 5.57, 1.18, 2.32*

* Documented deviations: `AISC-J-FILLETS` (J of open shapes by Σ b t³/3 without the fillet term, 6.5 % low), `AISC-C-SLOPE` (channel flanges modelled parallel at the average thickness: weak-axis values differ), `AISC-L-TOES` (square angle toes: Iz 3 % high). Use the database values for those properties of rolled shapes.

CHAPTER 38

Unit converter

Converts structural quantities between US customary and 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

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

Example	Conversion	Exact	ILX Calcs
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 39

Reports and settings

Calculation report

Report creates the calculation report as a PDF: a first page with the project, the calculation and its key results, then the inputs, the loads and combinations, every check with its equations and code clauses, and the diagrams. **Customize** changes what the report includes for this one report. A project report puts several calculations in one PDF.

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).

Title block

Firm name, address, engineer, logo and the fields printed on report pages and sheets.

Report

Sections and their order, the level of detail, units (including dual units), headers and footers, signatures and the disclaimer.

CHAPTER 40

Verification

How ILX Calcs is verified

Every calculation type is checked against published worked examples. Each published value (a required strength, an available strength, an intermediate quantity such as Cb) is an automated test: the app's value must match within 1 % (with a small absolute allowance for rounded published values), and the tests run on every release. The **Verification** page lists every example and value with the app's result and the difference; a value the app computes differently on purpose is listed as a documented deviation with its reason.

Calculation	Examples
Steel beam	AISC Design Examples v16.0: F.1-1A, F.1-2B, F.1-3B, F.3B, G.1B

CHAPTER 41

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.
- **Something else.** Use **Support** in the header to ask; include the calculation so we can see what you see.



ILX **Calcs**

Structural calculations, step by step

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