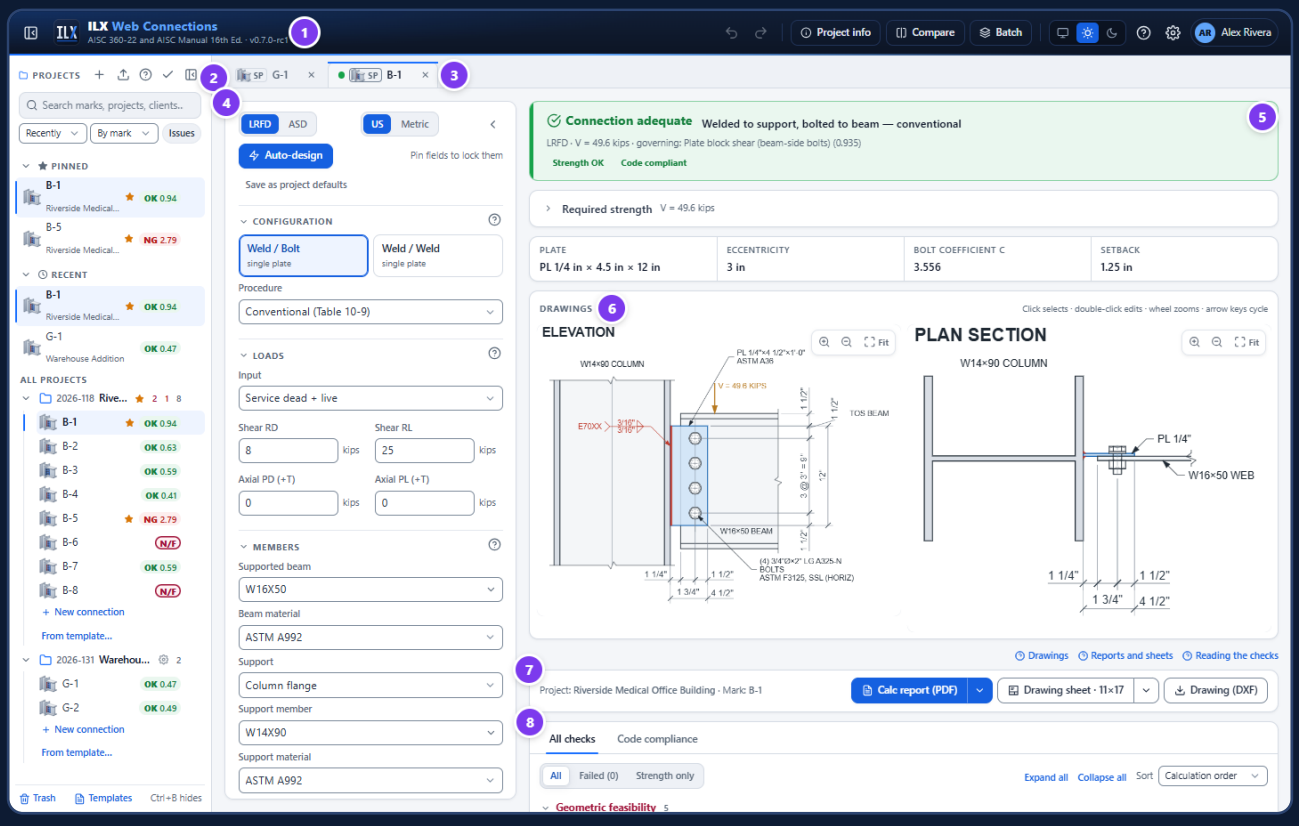
ILX Connection

STEEL CONNECTION DESIGN TO AISC 360-22

# User Manual

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




**ILX Studio, LLC** · support@ilxstudio.com

Version 1.0.0 · October 2026

### ILX Connection User Manual

Version 1.0.0, October 2026. Covers the ILX Connection web application: AISC 360-22 Specification for Structural Steel Buildings and the AISC *Steel Construction Manual*, 16th Edition.

### Responsibility

ILX Connection 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://ilxconnection.com/legal/disclaimer.

### Notices

ILX Studio, LLC is not affiliated with or endorsed by AISC. ILX Connection on the web is the online version of the upcoming ILX Connection desktop application.

The latest version of this manual is always online at https://ilxconnection.com/manual. Questions and feedback: support@ilxstudio.com.

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CHAPTER 1

# Overview

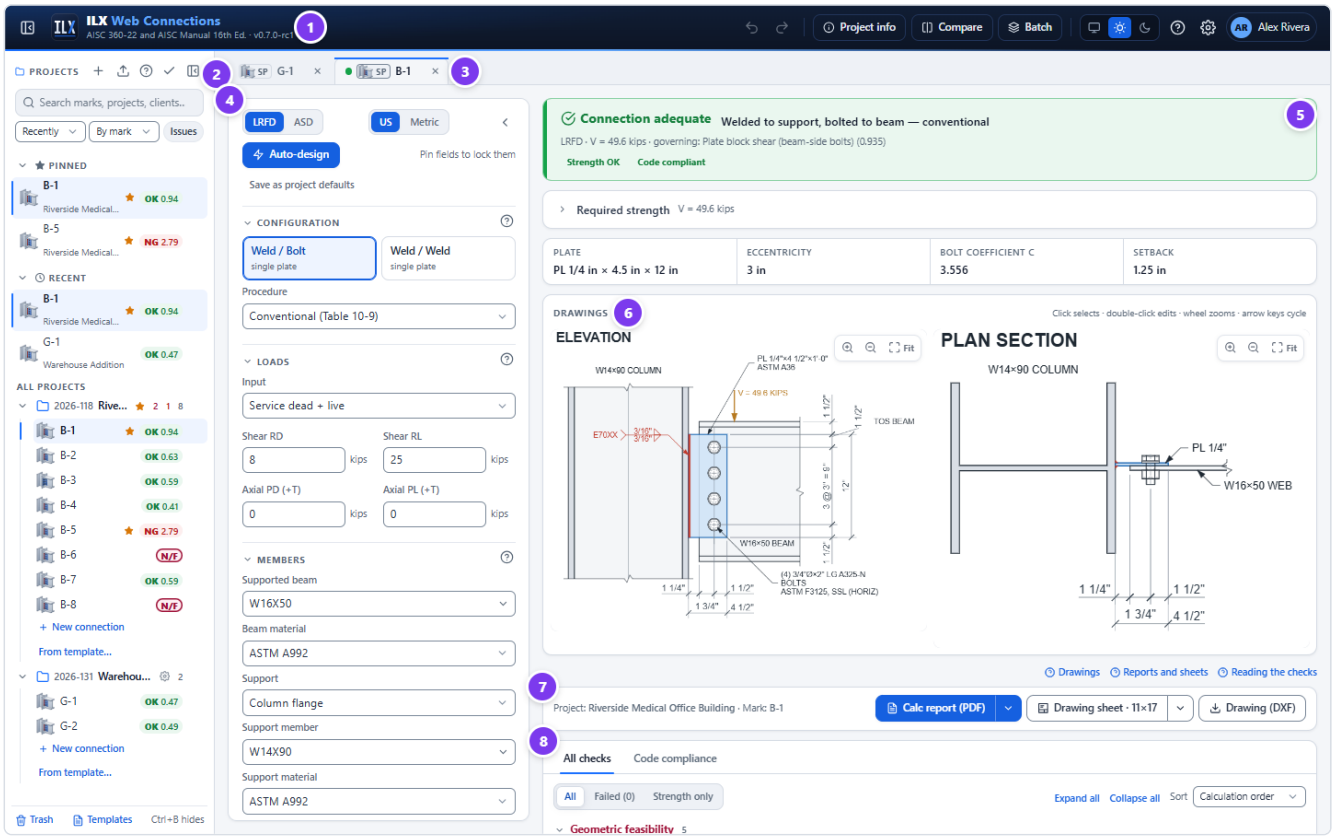
ILX Connection v1.0.0 · AISC 360-22 · AISC *Steel Construction Manual*, 16th Ed.

ILX Connection designs and details steel connections in a web browser. You enter the members, loads and connection geometry. ILX then checks every applicable limit state and detailing rule, draws the connection, and produces a calculation report and a title-blocked drawing sheet.

This release covers the connection types and component calculators below, each with its own editor, checks, drawings, report and sheet:

| Group | Type | Page |
|---|---|---|
| Simple shear | Single plate (shear tab, extended shear tab, all-welded plate; HSS through plate and round HSS) | Designing a single-plate connection |
| Simple shear | Single angle (bolted to the support; bolted or welded to the beam) | Designing a single-angle connection |
| Simple shear | Double angle (all-bolted, bolted / welded, all-welded) | Designing a double-angle connection |
| Simple shear | Shear end plate | Designing a shear end-plate connection |
| Simple shear | Seated (unstiffened bolted or welded, stiffened) | Designing a seated connection |
| Simple shear | Shear tee | Designing a shear tee connection |
| Simple shear | Skewed double bent plate (beam to girder web) | Designing a skewed double bent-plate connection |
| Moment | FR moment (directly welded flanges, bolted or welded flange plates; column stiffeners and doublers) | Designing an FR moment connection |
| Moment | End-plate moment (the 15 flush and extended configurations of AISC Design Guide 39; column continuity plates and doublers) | Designing an end-plate moment connection |
| Moment | Double-tee moment (AISC 358-22 Chapter 13, SMF and IMF; T-stubs, stiffness, column flange, continuity plates, panel zone) | Designing a double-tee moment connection |
| Splices | Beam shear splice; compression splice (column or chord) | Designing a splice |
| Truss & bracing | Truss support joint and panel point | Designing a truss connection |
| Truss & bracing | Tension member end connection | Designing a tension member end connection |
| Truss & bracing | Vertical bracing gusset (Uniform Force Method) | Designing a vertical bracing gusset (UFM) |
| Truss & bracing | Chevron bracing gusset | Designing a chevron bracing gusset |

| Group | Type | Page |
|---|---|---|
| Truss & bracing | Horizontal bracing wrap-around gusset (cut out around the column; Manual Fig. 13-10) | Designing a horizontal bracing wrap-around gusset |
| Members | Built-up member connectors | Designing built-up member connectors |
| Members | Beam web opening | Designing a beam web opening |
| Hangers & bearing | Hanger; beam bearing plate; HSS strut end plate | Hangers, bearing plates and HSS strut end plates |
| Hangers & bearing | Column bearing on a beam | Designing a column bearing on a beam |
| Hangers & bearing | Crane-rail connection (rail fastening and rail splices, Manual Part 15) | Designing a crane-rail connection |
| Column bases | Column base plate (axial, moment, shear; anchor rods to ACI 318-19 Chapter 17) | Designing a column base plate |
| Column bases | Embedded column base (column set into the foundation, DG1 Chapter 5) | Designing an embedded column base |
| Column bases | Braced-frame base (brace gusset on the column and the plate; DG29 §4.3) | Designing a braced-frame column base |
| Components & calculators | Bracket plate, bolt group, weld group, bolted joint, welded joint | Components and calculators |



## What ILX does

- **Connection types.** The types above, chosen in the wizard's **Connection type** step (see Choosing the connection type).
- **Supports.** Column flange, column web, girder web (with top-of-steel offset) and HSS column wall, as each type allows. Base plates sit on a concrete pier or footing.
- **Loads.** LRFD or ASD in US or metric units, from any of these:
  - service dead and live loads;
  - service components combined by ASCE 7-22;
  - required strength entered directly;
  - a table of factored load cases.

  Shear can be combined with axial tension or compression where the type supports it. FR and end-plate moment connections and base plates take moments as well.
- **Bolts.** Bearing-type and slip-critical joints, every ASTM F3125 grade (plus F3043, A307 and threaded rod), every hole type, and fillers.
- **Welds.** Fillet welds by the instantaneous-center method, one-sided or both-sided support welds (single plate), and CJP groove welds (FR moment, base plates).
- **Copes.** Top, bottom and double copes, with the reentrant-corner radius.
- **Checks.** Each check shows its code clause, the required and available values, the ratio and a fix hint.
- **Feasibility.** A geometric feasibility gate marks any connection that cannot be built as **NOT FEASIBLE**.
- **Design tools.** Auto-design for every type, batch design of a schedule of marks, and side-by-side comparison.
- **Outputs:**
  - interactive on-screen drawings;
  - a configurable PDF calculation report;
  - a drawing sheet with your title block, as PDF and DXF.
- **Project files.** Projects and connections are stored on the server under your account, with search, pins, templates, version history, trash, and import and export.

## What ILX does not do

ILX follows the AISC Specification, Manual, design guides and seismic standards as written. Where a provision leaves a value or a choice to the engineer of record, ILX asks for it (with a stated default) and reports it; it does not assume it. The items below are outside what ILX designs today.

- **Member design.** The beam, column, brace and support are checked only for the local limit states of the connection (web shear, coped sections, column flange and web local limit states, panel zone, out-of-plane strength). Member design, including runway girders, is not included.
- **Shop drawings.** The drawings are schematic connection details, not fabrication drawings.
- **Seismic.** AISC 358-22 RBS, WUF-W, BFP, BUEEP/BSEEP and double-tee connections and the AISC 341-22 SCBF, OCBF, EBF and BRBF connection rules are designed. Proprietary and cast 358 connections (Chapters 9 to 12, 14 and 15), built-up, boxed and cruciform columns, plug-welded doublers and knife-plate brace details are not.
- **Skewed and sloped details.** Skewed single plates and end plates, single and double bent plates, and sloped beams on single plates and angles are designed. Canted beams, combined skew and slope, skewed or sloped

seats, all-bolted or wrapped single bent plates, and a slope on shear tees, end plates and seats are not; sloped elevations are drawn square.

- **HSS supports.** Angles may be through-bolted or blind-bolted to a rectangular HSS wall; round HSS walls and HSS walls under axial tension from bolted angles (J3.13) are not covered.
- **Fatigue.** AISC 360 Appendix 3 is designed for the details of the wired types (see each type's page); bracing gussets, splices, PR moment and several calculators are not wired yet, and distortion-induced fatigue is not checked.
- **Chevron gussets.** The Concentrated Stress Method follows the Manual's description only (Sabelli and Saxey 2021, EJ 58(1), is not on file); the Uniform Stress Method is the default.
- **Crane rails.** Web yielding and crippling of the runway girder under the wheel, the rail section, and clip, clamp and hook components are not checked; the rail fastening bolts, Part 15 / DG7 / CMAA detailing and the crane forces are.

## Design basis

| Topic | Basis |
|---|---|
| Specification | ANSI/AISC 360-22, *Specification for Structural Steel Buildings* |
| Design aids | AISC *Steel Construction Manual*, 16th Ed.: Part 7 (bolts, instantaneous center), Part 8 (welds), Part 9 (connecting elements, coped beams), Part 10 (simple shear connections), Part 11 (FR and PR moment connections), Part 12 (simple connections for combined forces), Part 14 (column base plates) |
| Base plates | AISC Design Guide 1, 3rd Ed. (base plate and anchor rod design); ACI 318-19 Chapter 17 (anchoring to concrete) |
| Worked examples | AISC *Design Examples* v16.0 (Chapter II.A simple shear, II.B moment, J and K examples), which set the default bolt-group method; DG1 examples for base plates |
| Load combinations | 1.2D + 1.6L and 1.4D (LRFD), or D + L (ASD); or the ASCE/SEI 7-22 strength (2.3) and allowable stress (2.4) combinations |
| Shapes | AISC Shapes Database v16.0 (2,299 shapes) |
| Units | US customary by default, SI selectable. Calculations run internally in US units. |
| Drafting | AISC detailing practice, NCS/UDS layers and title block, AWS A2.4 weld symbols |

The verification of the calculations against published examples is summarised in Verification.

## Engineer-of-record responsibility

ILX is a design aid. It does not replace engineering judgement, and it does not know your project's load path, construction sequence or contract requirements. The engineer of record remains responsible for:

- the design, and for checking that the inputs (loads, members, materials and geometry) are correct;
- deciding whether the limit states and methods ILX applies are appropriate for the project;
- reviewing every report and drawing before it is issued, and sealing it where required.

Every report ends with a disclaimer to this effect, which you can edit in the report settings.

## How this manual is organised

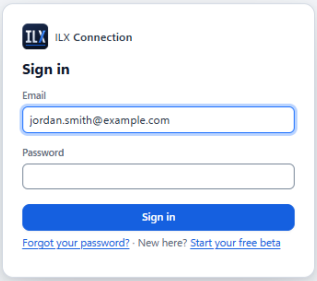
| Page | Read it when you want to… |
|---|---|
| Getting started | Sign in, learn the layout and design your first connection |
| Projects and files | Organise, find, copy, version, import, export or back up your work |
| Designing a single-plate connection | Choose a connection type and a configuration and enter members, loads, bolts, plate, copes and welds; use auto-design, compare and batch |
| Single angle, double angle, shear end plate, seated, shear tee | Design the other simple shear connection types |
| Skewed double bent plate | Design a skewed beam-to-girder connection with bent plates |
| Designing an FR moment connection | Design a fully restrained moment connection and its column stiffening |
| Designing a double-tee moment connection | Design the AISC 358 double-tee connection of a special or intermediate moment frame |
| Designing a splice | Design a beam shear splice or a column / chord compression splice |
| Designing a truss connection | Design a truss support joint or a panel point |
| Designing a horizontal bracing wrap-around gusset | Design a horizontal brace gusset cut out around a column |
| Hangers, bearing plates and HSS strut end plates | Design a hanger, a beam bearing plate or an HSS strut end plate |
| Designing a crane-rail connection | Fasten a crane rail to a runway girder and detail the rail splices |
| Designing a column base plate | Design a column base plate and its anchor rods |
| Designing a braced-frame column base | Design the base plate and gusset at a braced frame |
| Components and calculators | Check a bracket plate, a bolt or weld group, or a bolted or welded joint |
| Checks and compliance | Read the results and fix failing or infeasible connections |
| Drawings | Use the on-screen drawings, drawing sheets and DXF files |
| Reports | Produce and configure calculation reports and project packages |
| Settings | Set up your title block and report defaults, project overrides, users |
| Verification | See how ILX was verified against the AISC Design Examples |
| FAQ and troubleshooting | Solve common problems |
| Glossary | Look up a term or abbreviation |

CHAPTER 2

# Getting started

This page covers signing in, the parts of the screen, themes and keyboard shortcuts, and ends with a five-minute first design.

## Signing in




1. Open ILX Connection, for example `https://connections.ilxstudio.com/webapp/`. It runs in your browser; there is nothing to install. (The address without `/webapp/` opens the ILX Connection website, with a link to the app.)
2. Enter the **Email** and **Password** of your ILX account (below). New here? Select **Start your free beta**.
3. Select **Sign in**.

Everyone signs in with an ILX account, administrators included; ILX Connection keeps no separate usernames or passwords. Your session stays open for 30 days from the last time you used it. To sign out, open the account menu in the header (your name) and choose **Sign out**.

> **Forgot your password?** Select **Forgot your password?** on the sign-in page (see below). After repeated failed attempts, sign-in for that email is paused for 15 minutes.

# Your ILX account (ilxconnection.com)

On ilxconnection.com you sign in with an **ILX account**: your email address and a password. The same account will also work for the ILX Connection desktop application.

**Start your free beta.**

1. On the website, select **Start free beta**.
2. Enter your first and last name, your company (optional), your email address and a password (at least 8 characters, typed twice). Accept the Terms of Service and the Privacy Policy, then select **Create account**.
3. We email you a **6-digit code**. Enter it on the **Confirm your email** page. You are signed in and the app opens.

Your free beta lasts 90 days from the day you confirm your email, with every connection type, drawings and reports. No email? Check your spam folder; it can take a minute or two. To get a new code, sign up again with the same email. Your name and company start your title block (you can change them in **Settings**).

**Sign in.** Open the app, enter your **Email** and **Password**, and select **Sign in**. If you have not confirmed your email yet, the sign-in page links to the code page.

**Forgot your password?** Select **Forgot your password?** on the sign-in page, enter your email, and we email you a code. Enter it with your new password on the next page; you are signed in right after. The code is valid for 30 minutes.

**Already used ILX Connection before your ILX account?** Your account was moved for you: you receive a "Welcome to ILX Connection" email with a code to set your password (valid for 3 days; after that use **Forgot your password?**). Sign in with that email address and your projects are where you left them. If your projects were moved to another person's account instead, your administrator has told you so.

## When your beta or subscription ends

You can still sign in, open every project, and print or download the reports, drawing sheets, DXF files and project files of your saved connections. You cannot create or change designs until you subscribe. A banner at the top of the app says so, with a **Subscribe** button.

## Subscribing and managing your subscription

**Subscribe** (in the banner, or in the account menu during the beta) opens the Pricing page; choose **Subscribe monthly** or **Subscribe yearly**. Payment is made on a Stripe checkout page with the email address of your ILX account. Stripe sells the subscription as the merchant of record ("Sold through Link") and sends your receipt; your card details never reach ILX Studio. When you come back to ILX Connection it checks your license at once and shows "Your ILX Connection subscription is active"; if it takes longer, select **Check again**.

The account menu shows your license (for example "Free beta · 42 days left" or "Yearly subscription"). With a subscription it has **Manage subscription**, which opens the Stripe customer portal to cancel, switch between monthly and yearly, or change your card.

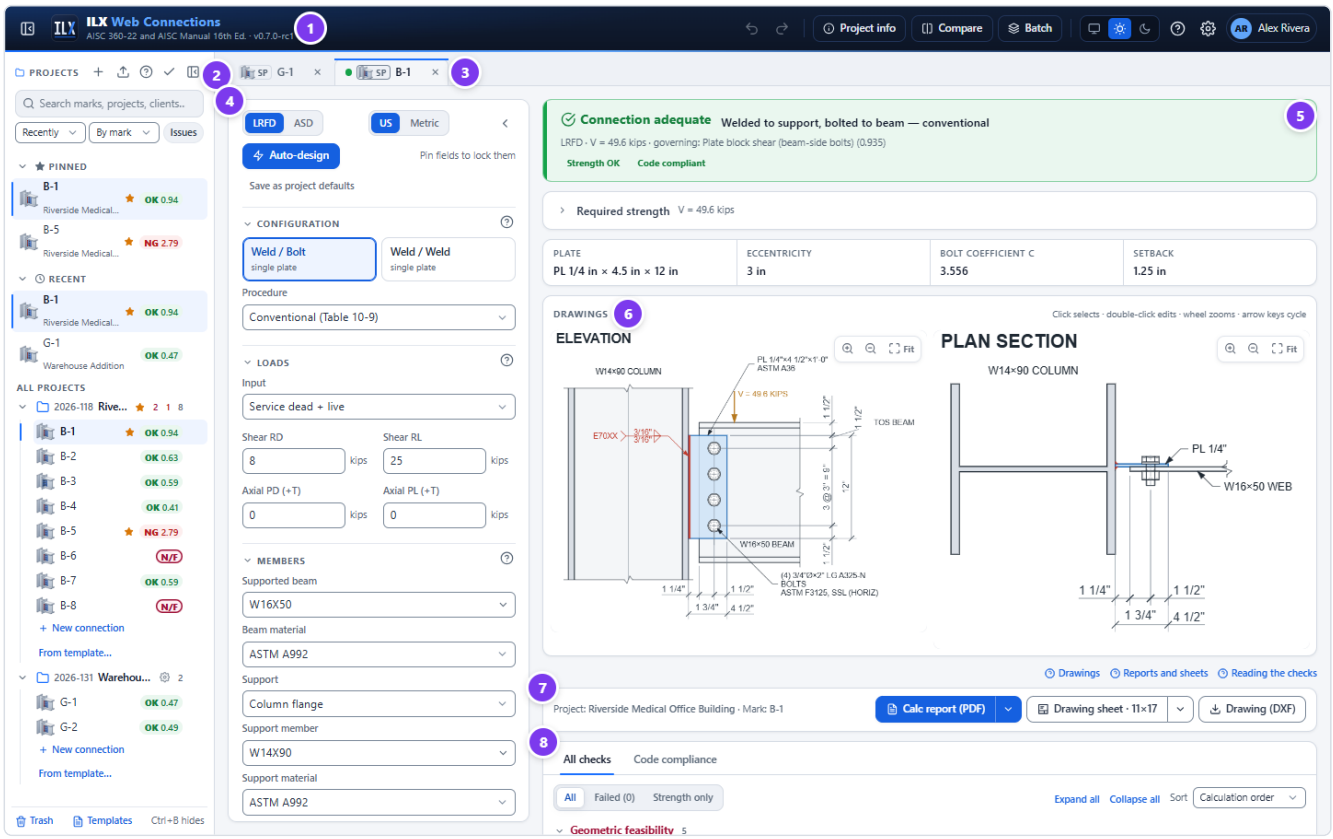
If ILX Connection cannot reach the ILX licensing service for a long time (more than 3 days), it also switches to read-only for safety, with a message that says so. Your work is never locked away; editing comes back as soon as the service answers again.

## Administrators

There is no administrator setup screen. The administrators are ILX accounts too: whoever runs the server lists their email addresses in the server setting `ILX_ADMIN_EMAILS` (see `INSTALL.md`, section 4), and they sign in like everyone else. An account whose email is on that list is an administrator from its next sign-in or page load; taking the email off the list makes it a user again. Administrators see **Manage users…** in the account menu and are never read-only.

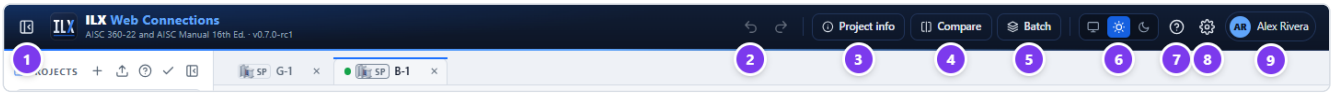
When you sign in with no projects, ILX creates **My first project** with one connection, **ST-1**, so there is something to open.

## A tour of the interface



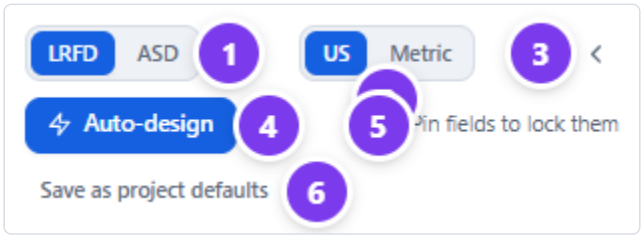

1. **Header.** The application name, code basis and version, with the main commands (described below).
2. **Project panel.** Your projects and their connections, with search, pinned and recent lists. See Projects and files.
3. **Tabs.** One tab per open connection. The dot shows the live status (green adequate, red not adequate, a warning triangle for not feasible). ● means a change is waiting to be saved; **saving…** means it is being saved.
4. **Input panel.** The connection inputs, in collapsible sections. See Designing a single-plate connection.
5. **Status bar.** Adequate or not, the governing check and its ratio, and the counts of strength failures and code-compliance violations.
6. **Drawings.** Elevation and plan views, drawn to scale. Click to select an element, double-click to edit it. See Drawings.
7. **Output bar.** Calc report (PDF), drawing sheet (PDF) and drawing (DXF). See Reports.
8. **Results. All checks** and **Code compliance**. See Checks and compliance.

## The header




1. **Show or hide the project panel** (Ctrl+B).
2. **Undo** and **Redo** for the active connection (Ctrl+Z, Ctrl+Y).
3. **Project info** opens the project information of the active connection's project.
4. **Compare** two open connections side by side. You need at least two tabs open.
5. **Batch** opens batch design for the active project.
6. **Theme**: System, Light or Dark.
7. **Help** opens this manual (F1).
8. **Settings**: title block and report settings, the Verification page and the user manual.
9. **Account menu**: your profile and password, **Manage users…** (administrators) and **Sign out**.

## The input panel toolbar



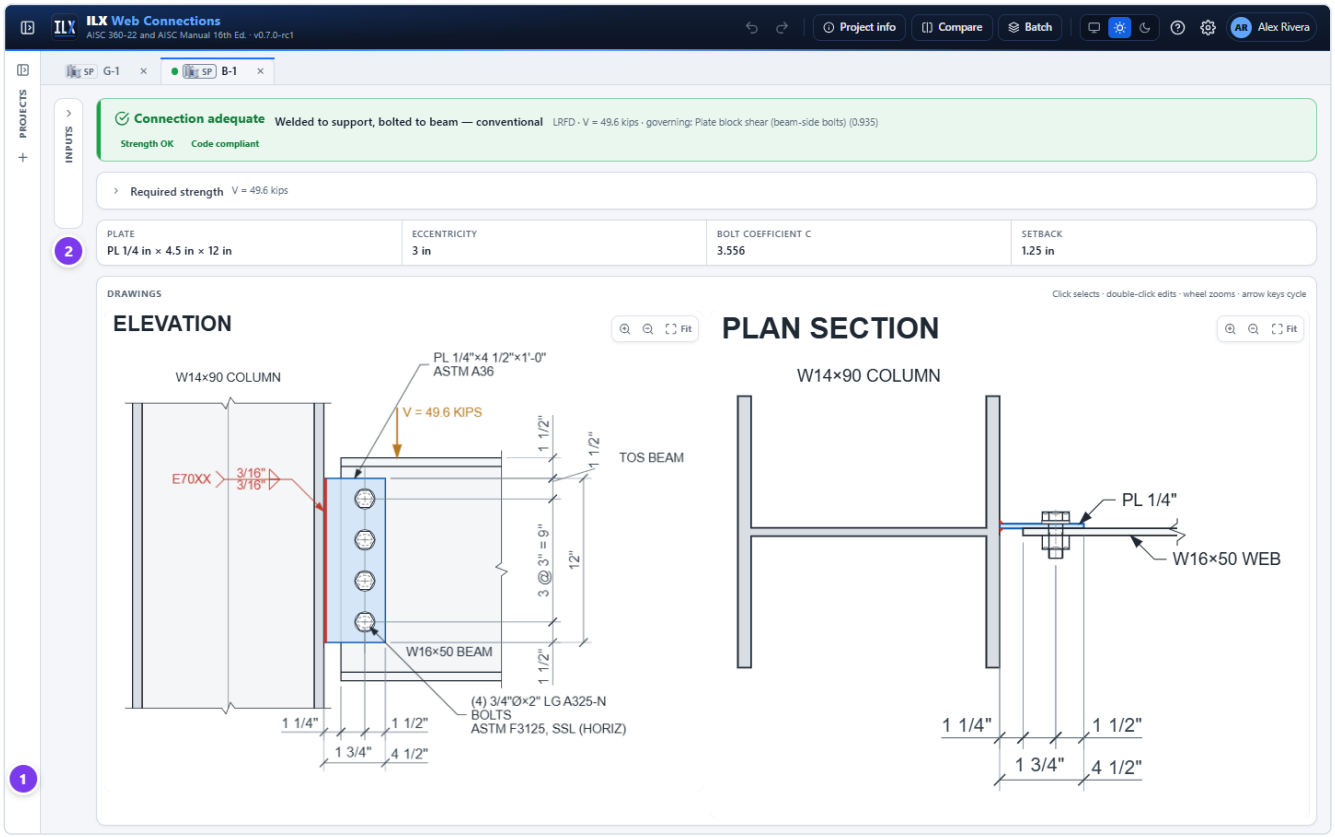

1. **LRFD / ASD** sets the design method.
2. **US / Metric** switches the units. Every length and force is converted, so the design is unchanged.
3. **Hide inputs** collapses the input panel to a narrow rail. Select the rail to bring it back.
4. **Auto-design** searches for an adequate design (see Auto-design).
5. The number of **pinned** fields, which auto-design keeps.
6. **Save as project defaults** stores this connection's materials, bolts, welds and methods as defaults for new connections in the project.

## Entering values

- **Sections.** Select a section title to collapse or expand it. ILX remembers which sections you collapsed.
- **Fractions.** US lengths accept fractions and feet-inches: `1 1/2`, `3/4`, `1-1/2` or `1'-2 1/2"`. The unit column shows the value as a fraction, for example `1 1/2″`.
- **Arrow keys.** Up and Down step a length by 1/16 in (1 mm in metric) and a count by 1. Hold Shift for steps four times larger.
- **Invalid values.** Text that is not a valid number is never sent as zero. The field shows an error, and the last valid value comes back when you leave it.
- **Autosave.** ILX saves about one second after you stop typing. There is no Save button.
- **Undo.** Each tab keeps 100 undo steps.

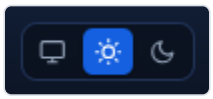
## Hiding the panels

To give the drawings and results the full width, hide the project panel (Ctrl+B) and the input panel.




1. The project panel rail: select it to show the panel again, or use **+** to create a project.
2. The input panel rail: select **Inputs** to show the panel again.

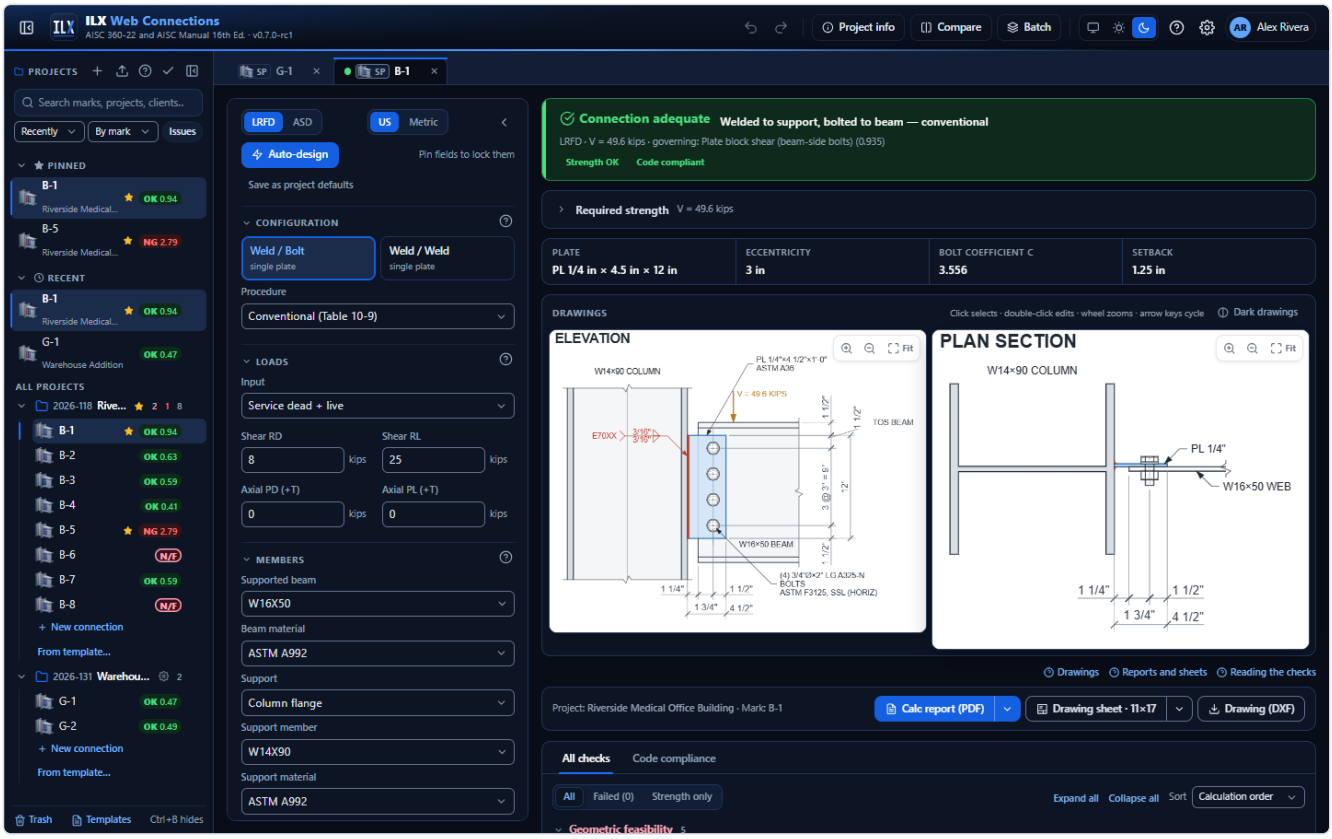
## Themes



The theme switcher in the header has three options:

- **System** follows your operating system's light or dark setting.
- **Light**.
- **Dark**.

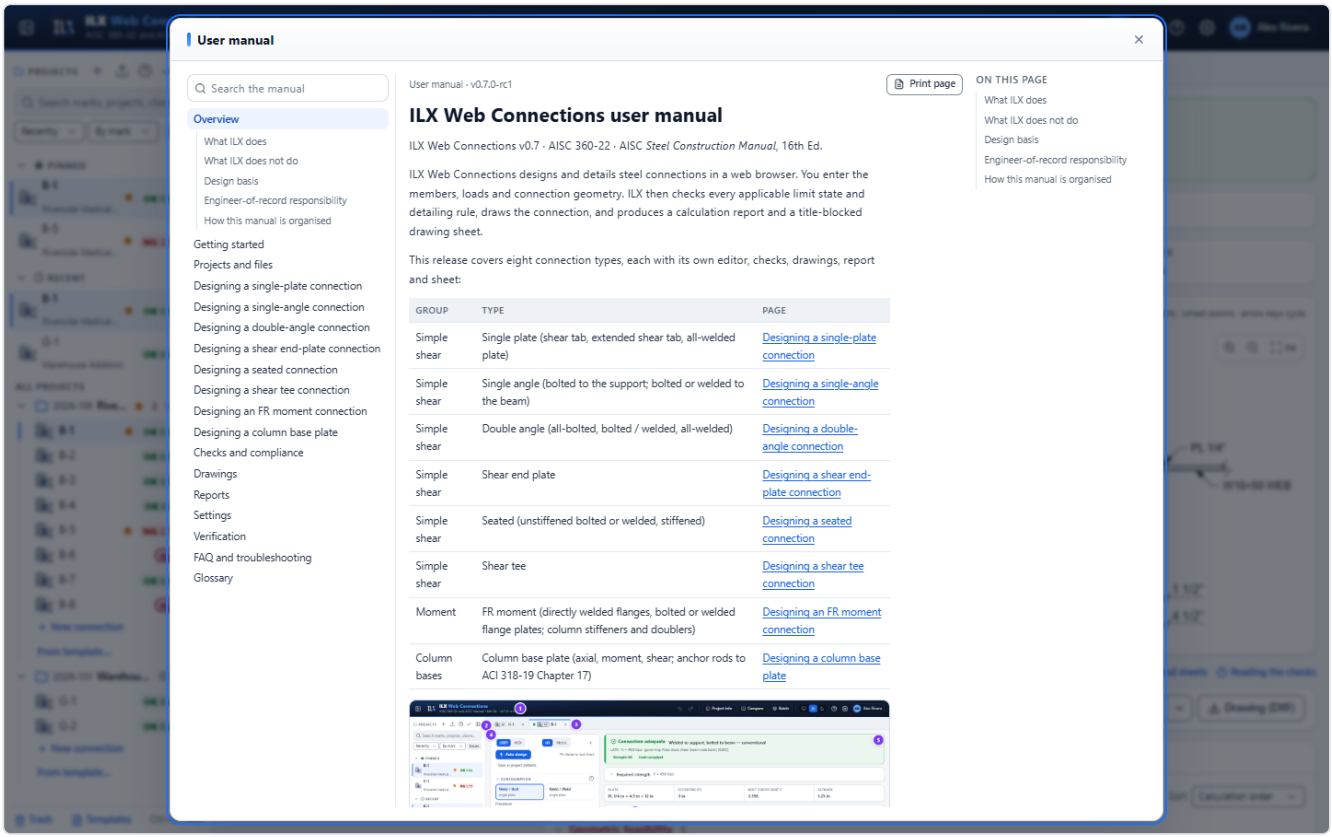
Your choice is saved to your account, so it follows you to other browsers.




The drawings stay on white paper in the dark theme, so they look like the printed sheet. To draw them in light ink on navy instead, select **Dark drawings** in the drawings bar (see Dark drawings).

## Getting help

- **The whole manual.** Press **F1** or select **?** in the header. The manual opens in a window over your work; close it to carry on.
- **Help for one section.** A small **?** at the right of an input section (Configuration, Loads, Members, Bolts, Plate, Welds), in the project panel and in Settings opens the manual at that topic.
- **Help links.** Under the drawings, the links **Drawings**, **Reports and sheets** and **Reading the checks** open the matching pages.
- **Inside Settings.** The manual is also on the **User manual** tab of Settings.




## Keyboard reference

| Where | Keys | Action |
|---|---|---|
| Anywhere | F1 | Open this manual |
| Anywhere | Ctrl+B | Show or hide the project panel |
| Editor | Ctrl+Z / Ctrl+Y (or Ctrl+Shift+Z) | Undo / redo |
| Number field | Up / Down (Shift for ×4) | Step the value |
| Tabs | Left / Right, Home / End | Move between open connections |
| Tabs | Delete | Close the focused tab |
| Project panel | Up / Down, Home / End | Move through the tree |
| Project panel | Right / Left | Expand / collapse a project |
| Project panel | Enter | Open the connection, or expand the project |
| Project panel | F2 | Rename |
| Project panel | Delete | Move to the trash |
| Project panel | Space, Ctrl+click, Ctrl+A | Select connections for bulk actions; Esc clears the selection |
| Project panel | Shift+F10 or the Menu key | Open the row's menu |
| Project panel | Type letters | Jump to the next matching row |

| Where | Keys | Action |
|---|---|---|
| Project panel search | Down | Move into the results; Esc clears the search |
| Drawings (panel focused) | Arrow keys or Tab | Select the next or previous element |
| Drawings | Enter | Edit the selected element |
| Drawings | Esc | Clear the selection |
| Element editor | Esc | Close |

## Your first connection in five minutes

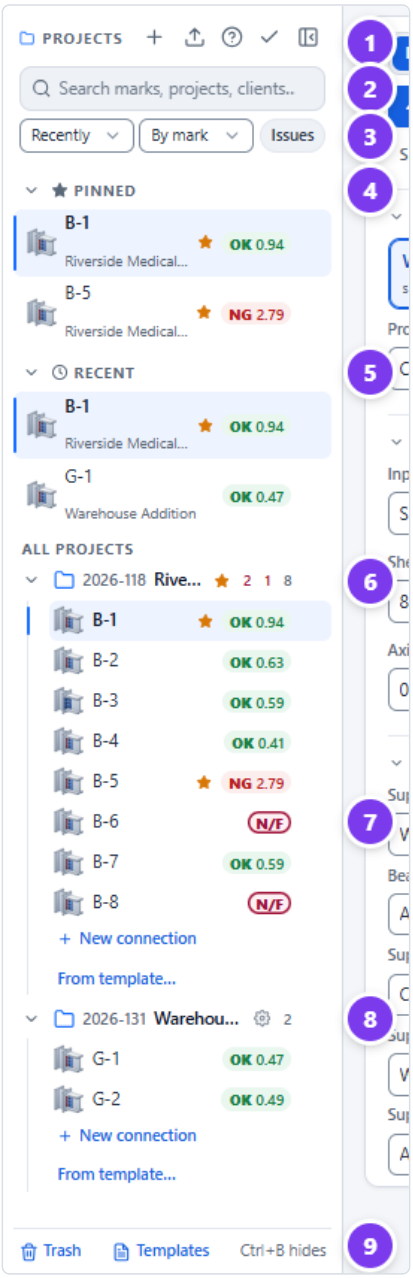
1. In the project panel, select **+** (New project), type a name and select **Create**. The **Project information** dialog opens. Fill in what you know, or select **Skip for now**.
2. On the project row, open the menu (**⋯**) and choose **New connection…**. The wizard opens.
3. Step 1, **Joint**: pick the joint type (for example **Beam to column flange**) and the connection (**Shear tab**). Select **Next**.
4. Step 2, **Members**: check the mark (ST-1), and pick the **Supported beam** and the **Column**. Type to search the AISC shapes, for example `w16x50`. Select **Next**.
5. Step 3, **Loads**: enter the service dead and live reactions, or choose **Required strength** and enter Ru (Ra in ASD). Tick **Auto-design the connection** if you want ILX to size the bolts, plate and weld. Select **Create**.
6. The connection opens in a tab and designs itself. Read the status bar: **Connection adequate** or **Connection not adequate**, with the governing check.
7. Change any input. The results, drawings and file-tree status update as you type.
8. Select **Calc report (PDF)** or **Drawing sheet** in the output bar to produce the deliverables.

CHAPTER 3

# Projects and files

Your work is organised as **projects** that hold **connections**. Each connection is one designed connection with its own mark (B-1, ST-3 and so on). Everything is stored on the ILX server under your account. Other users cannot see your projects.

# The project panel




1. **Panel commands**:
   - **+** New project.
   - **Upload** Import a project file.
   - **?** Open this page of the manual.
   - ✓ Select several connections.
   - **Hide** the panel (Ctrl+B).
2. **Search**: marks, project names, project numbers and clients.
3. **Sort and filter**:
   - Projects: **Recently changed**, **Project name** or **Project number**.

- Connections: **By mark**, **Worst status first**, **Highest ratio first** or **Last edited**.
- **Issues** shows only connections that are NG, not feasible or have input errors.

4. **Pinned**: connections you pinned, across all projects.
5. **Recent**: the six connections you opened last.
6. **Project row**:
   - project number and name;
   - a star if the project is pinned to the top;
   - counts: not feasible (outlined), not adequate (red) and the total number of connections.
7. **Connection row**: mark and status pill. Hover over the pill for the governing check.
8. **Project settings badge**: the gear marks a project with its own title block or report settings (see Project overrides).
9. **Trash** and **Templates**.

**Status pills**

| Pill | Meaning |
|---|---|
| **OK 0.94** | Adequate; the governing ratio is 0.94 |
| **NG 2.79** | Not adequate (a strength or code-compliance check fails); the governing strength ratio is 2.79 |
| **N/F** | NOT FEASIBLE: the geometry cannot be built (see NOT FEASIBLE) |
| **ERR** | The saved input is invalid |

Open a tab and the pill follows the live result as you edit.

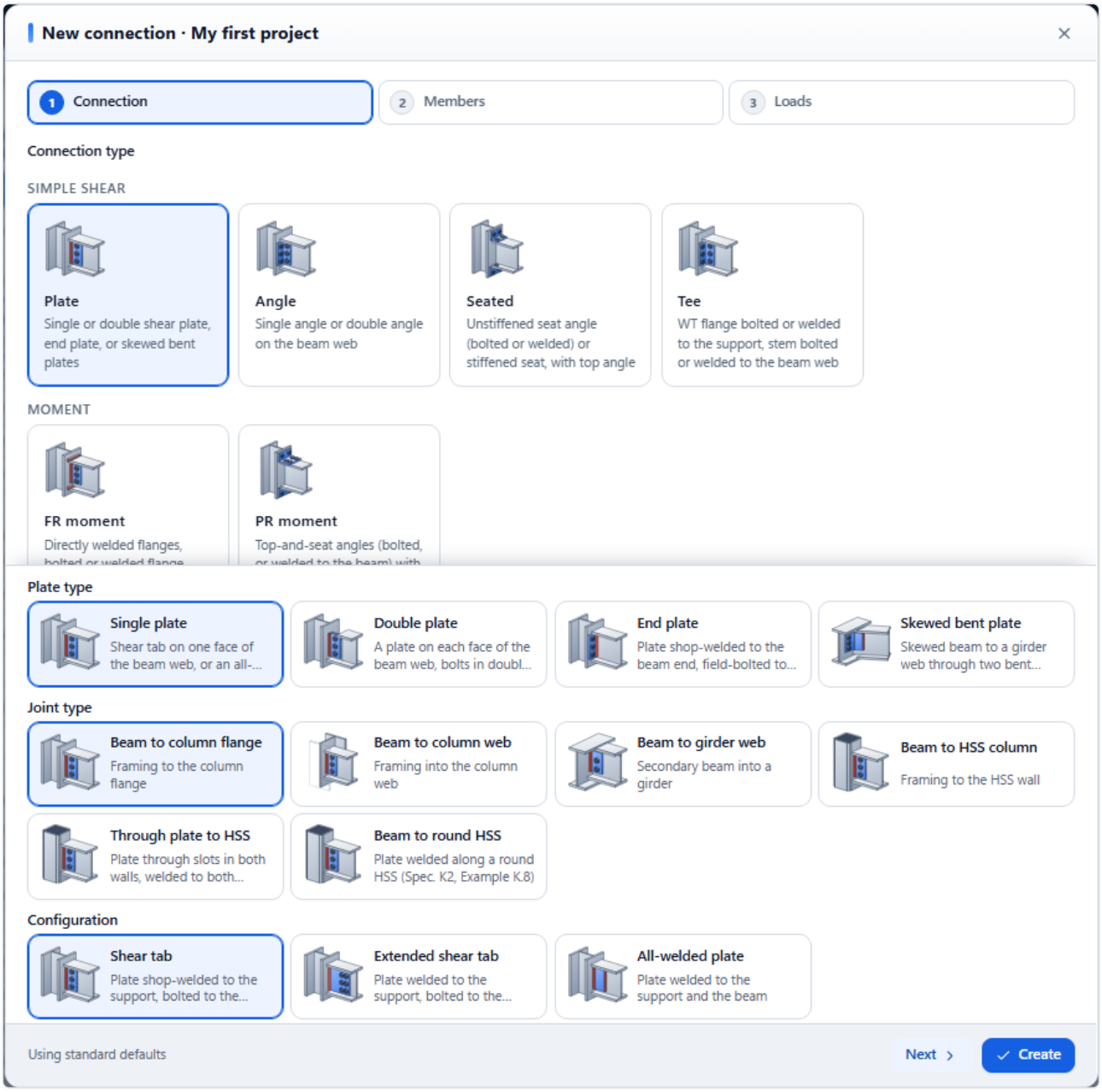
## Projects and connections

**To create a project**

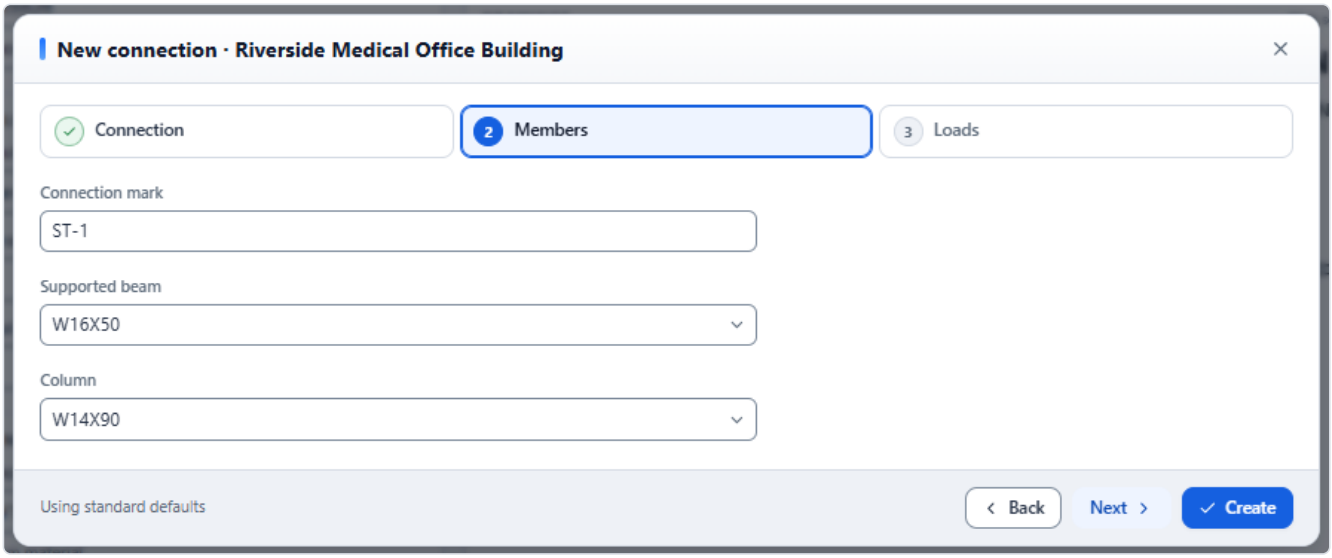
1. Select **+** at the top of the project panel.
2. Type the project name and select **Create**.
3. The **Project information** dialog opens. Fill it in now, or select **Skip for now** and come back later.

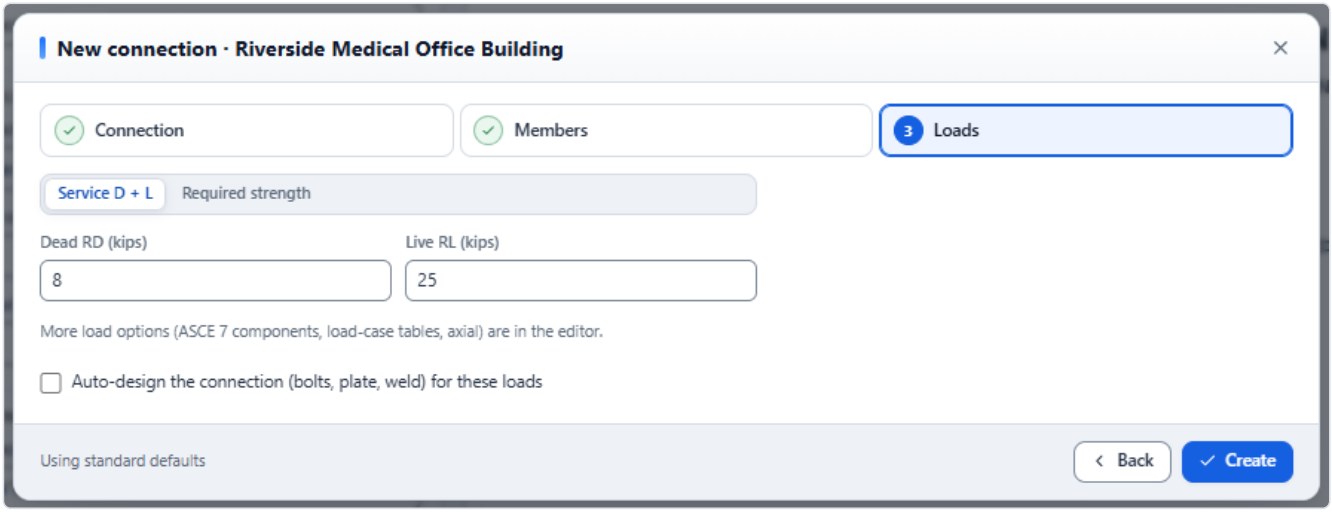
**To add a connection**

1. Open the project's menu (**⋯** on the project row, or right-click it) and choose **New connection…**, or select **+ New connection** under the project's connections.
2. Follow the three wizard steps (**Connection**, **Members**, **Loads**). Select **Create**. You can select **Create** on any step; the remaining values come from the project defaults or the standard defaults.




The **Connection** step holds the **Connection type** cards, grouped **Simple shear**, **Moment** and **Column bases** (see Choosing the connection type), then the type's options: **Plate type** or **Angle type** for the **Plate** and **Angle** cards, the **Joint type** and the type's **Configuration** cards, each with a small picture. For a single plate, **Shear tab** is Weld / Bolt (conventional), **Extended shear tab** is Weld / Bolt (extended) and **All-welded plate** is Weld / Weld. Cards that cannot be built on the chosen joint are disabled, with the reason in their tooltip; for example angles cannot be bolted to an HSS column. Picking **Beam to HSS column** sets the column to an HSS8X8X1/2 in ASTM A500 Gr. C (rectangular), which you can change on the next step; a round HSS switches the material to A500 Gr. C round. Picking a column or girder again restores a W shape (W14X90 or W24X68) in A992. The **Members** step asks for the type's own element where it has one (angle, seat angle or tee), and an FR moment connection asks for the beam and column and for moments in the **Loads** step. **Base plate** opens its own wizard.

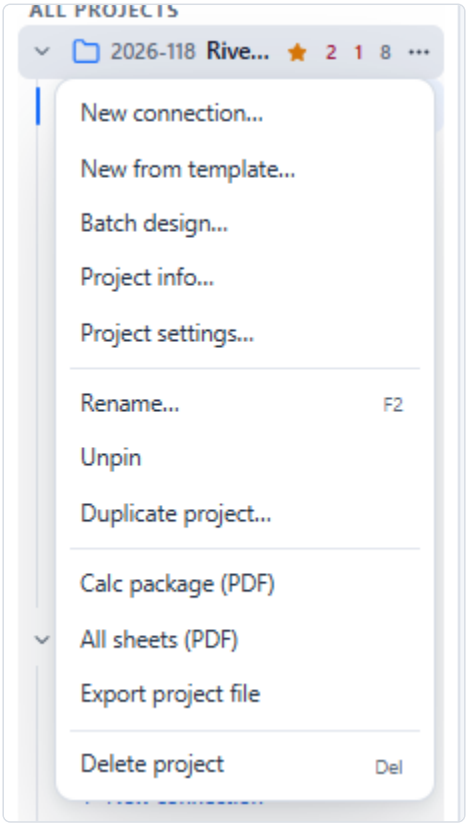


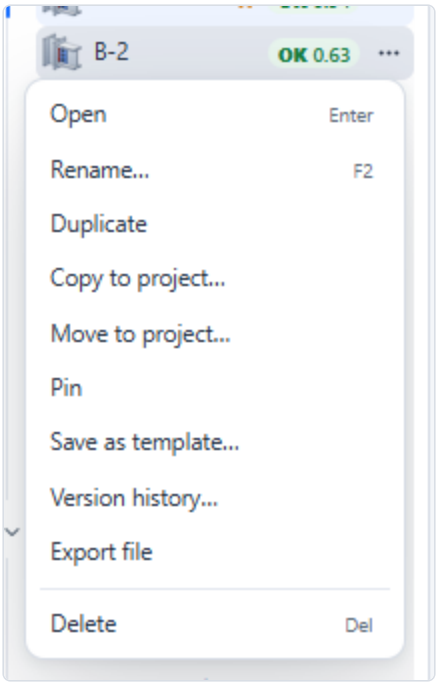




New connections get the next free mark, ST-1, ST-2 and so on. Rename them to your marks with **F2** or **Rename…** in the connection menu.

### To open, rename or delete

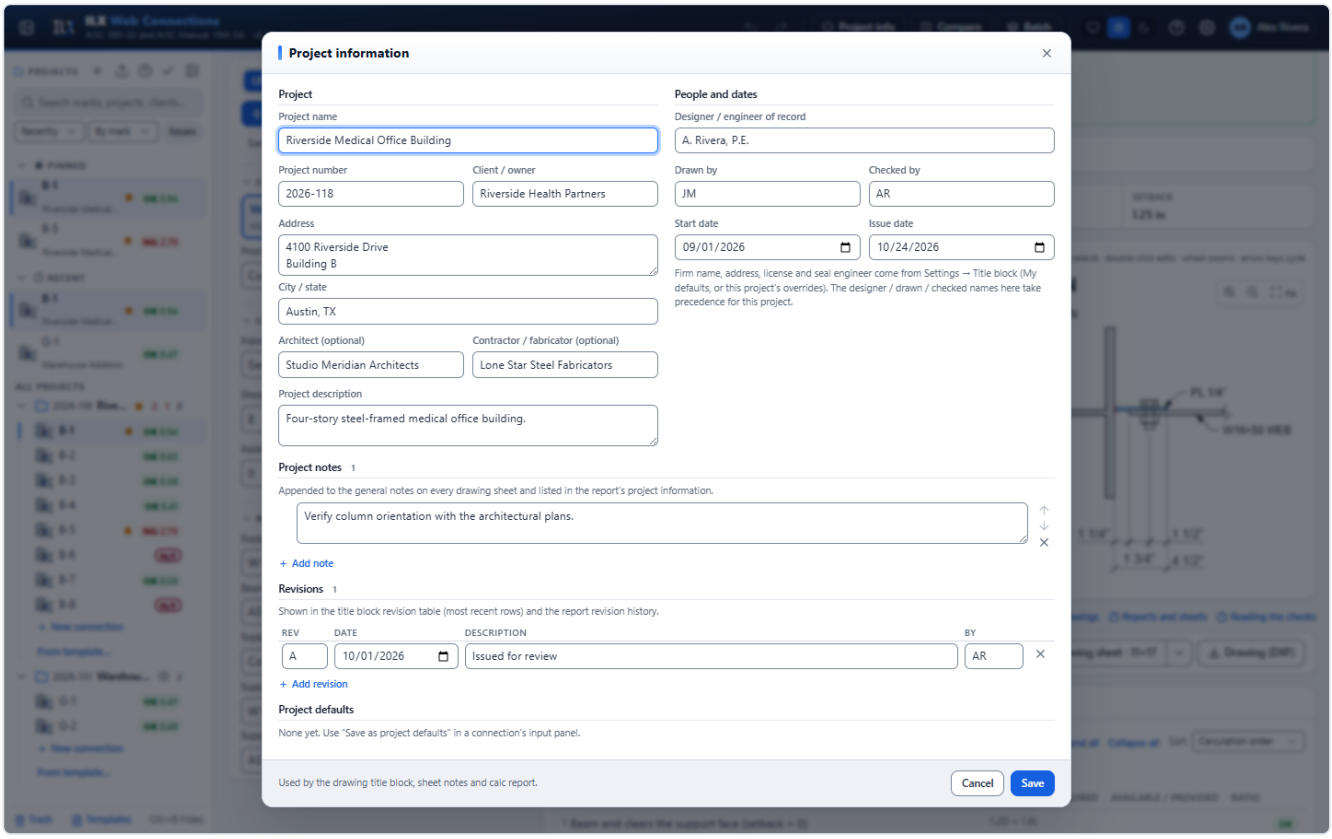
| Action | How |
|---|---|
| Open | Click the connection. It opens in a tab, or switches to its tab if already open. |
| Rename | Select the row and press **F2**, or choose **Rename…** in its menu. |
| Delete | Press **Delete**, or choose **Delete** in its menu. Deleted items go to the trash. |




## Project information

The project information feeds the drawing title block, the general notes on every sheet and the report's project section.




1. Open it with **Project info** in the header (for the active connection's project) or **Project info…** in a project's menu.
2. Fill in the fields:
   - **Project**: name, number, client or owner, address, city and state, architect, contractor or fabricator, description.
   - **People and dates**: the designer or engineer of record, **Drawn by**, **Checked by**, and the start and issue dates.
   - **Project notes**: appended to the general notes on every drawing sheet and listed in the report. Reorder them with the arrows.
   - **Revisions**: **Add revision** proposes the next letter or number with today's date. The latest rows print in the title block revision table; all of them can print in the report's revision history.
3. Select **Save**.

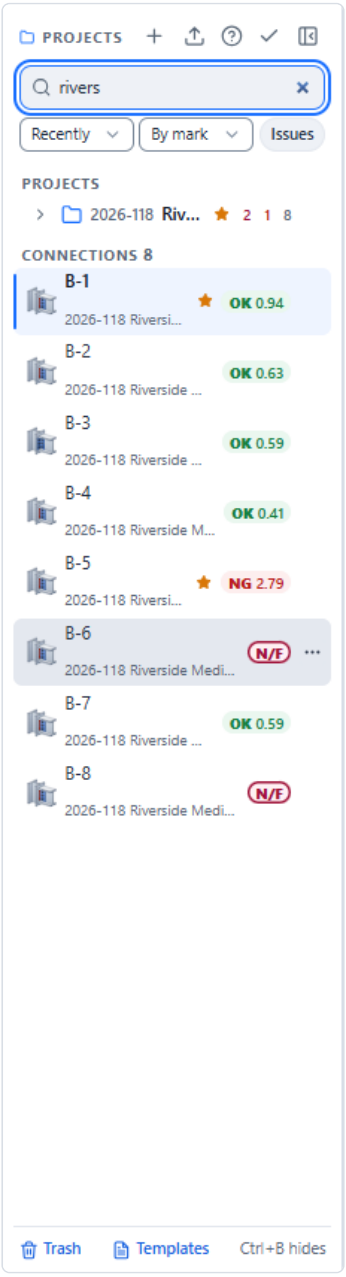
Firm-level data (firm name, address, license, seal engineer) is not entered here. It belongs to the title block settings (see Settings). Names entered here override those defaults for this project only. A blank field shows *Blank = project default* and uses the project's title block settings: the project override, or your own default.

## Project defaults

**Save as project defaults** in a connection's input panel stores its materials, bolts, welds and methods for the project. New connections in the project start from these values, and the wizard footer says **Project defaults applied**. To remove them, select **Clear defaults** at the bottom of the project information dialog.

## Finding connections

- **Search.** Type in the search box. It matches connection marks and project names, numbers and clients across all your projects. Press **Down** to move into the results, and **Esc** to clear.
- **Sort.** The two lists under the search box sort projects and connections. **Worst status first** puts NOT FEASIBLE, then NG, then errors at the top.
- **Issues.** Turn on the **Issues** chip to hide every adequate connection.




Your sort, filter and section choices are saved to your account.

## Pins and recent connections

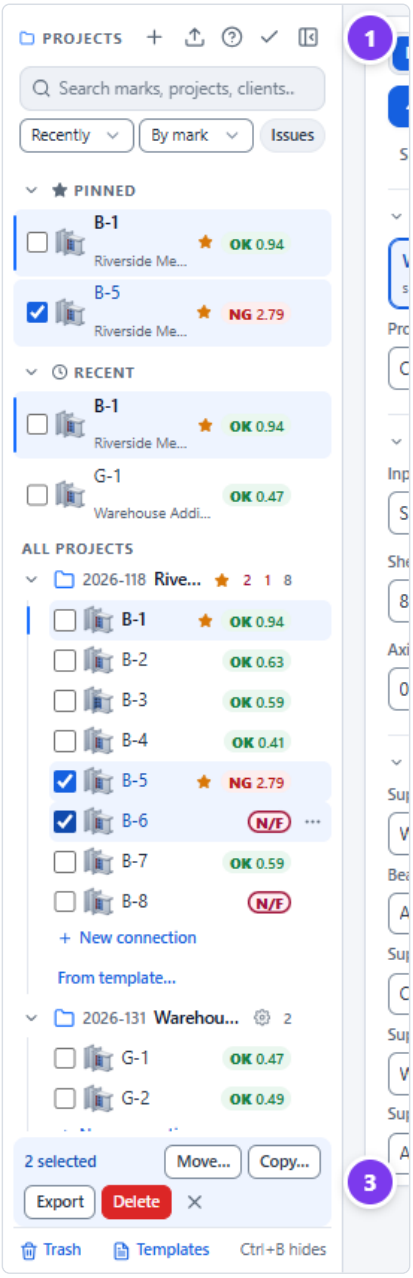
- **Pin a connection.** Choose **Pin** in its menu. Pinned connections appear in the **Pinned** list at the top of the panel and carry a star.
- **Pin a project.** Choose **Pin to top** in the project menu. Pinned projects sort first in **All projects**.
- **Recent.** The **Recent** list shows the six connections you opened most recently.
- **Collapse a list.** Select the **Pinned** or **Recent** heading.

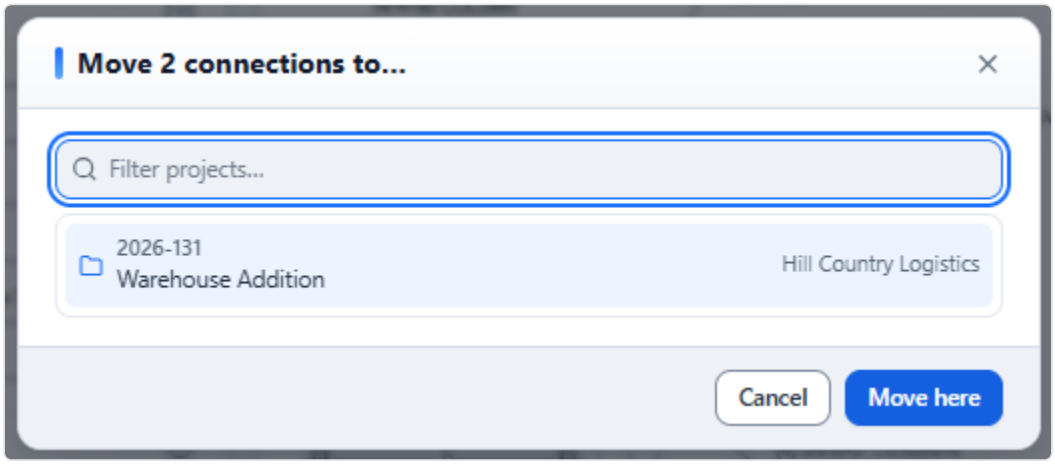
## Moving, copying and duplicating

| To… | Do this |
|---|---|
| Duplicate a connection in the same project | Connection menu → **Duplicate**. The copy gets the next free mark and opens. |
| Copy a connection to another project | Connection menu → **Copy to project…**, then pick the project. |
| Move a connection | Connection menu → **Move to project…**, or drag the connection onto another project. |
| Copy by dragging | Hold **Ctrl** (or Alt) when you drop. |
| Duplicate a whole project | Project menu → **Duplicate project…**. The copy has the project information, defaults, settings overrides and every connection, but not the trash or version history. |

### To act on several connections at once

1. Select ✓ (Select connections) at the top of the panel, or Ctrl+click the connections.
2. Tick the connections you want. Ctrl+A selects every visible connection.
3. Use the bar at the bottom of the panel: **Move…**, **Copy…**, **Export** or **Delete**.
4. Select **×** or press **Esc** to clear the selection.

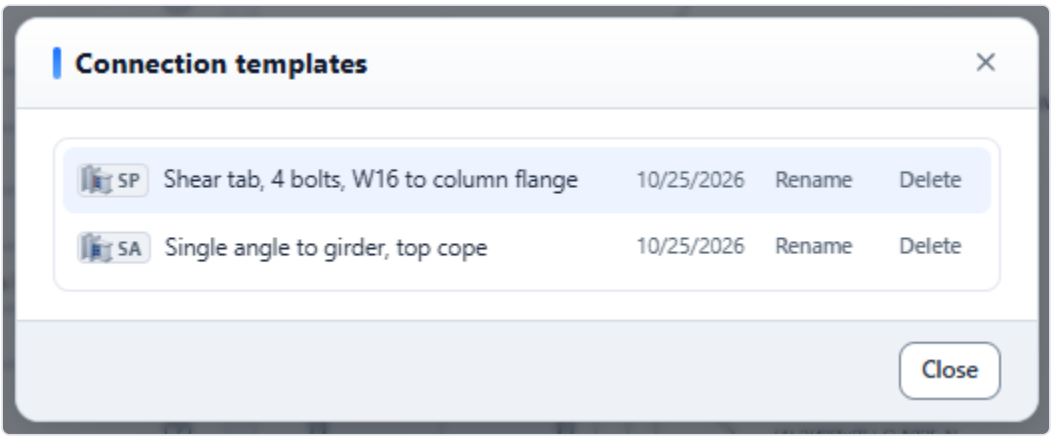

PROJECTS
Search marks, projects, clients..
Recently
By mark
Issues
PINNED
B-1
Riverside Me...
OK 0.94
B-5
Riverside Me...
NG 2.79
RECENT
B-1
Riverside Me...
OK 0.94
G-1
Warehouse Addi...
OK 0.47
ALL PROJECTS
2026-118 Rive...
2 1 8
B-1
OK 0.94
B-2
OK 0.63
B-3
OK 0.59
B-4
OK 0.41
B-5
NG 2.79
B-6
N/F
B-7
OK 0.59
B-8
N/F
+ New connection
From template...
2026-131 Warehou...
2
G-1
OK 0.47
G-2
OK 0.49
2 selected
Move...
Copy...
Export
Delete
Trash
Templates
Ctrl+B hides
1
3


Move 2 connections to...
Filter projects...
2026-131
Warehouse Addition
Hill Country Logistics
Cancel
Move here

## Templates

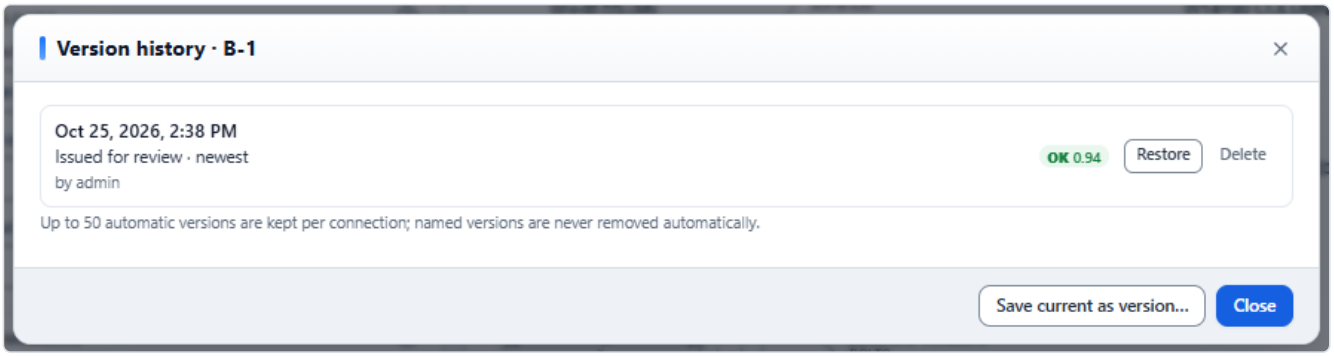
A template stores every input of a connection so you can reuse it in any project. Templates are private to your account.

1. **Save a template.** Open a connection's menu and choose **Save as template…**. Name it and select **Save template**.
2. **Use a template.** Open a project's menu and choose **New from template…**, or select **From template…** under the project's connections. Pick the template and select **Create connection**, or double-click it. The new connection gets the next free mark.
3. **Manage templates.** Select **Templates** at the bottom of the project panel to rename or delete them. Deleting a template does not affect connections made from it.




## Version history

ILX keeps earlier states of each connection's inputs.



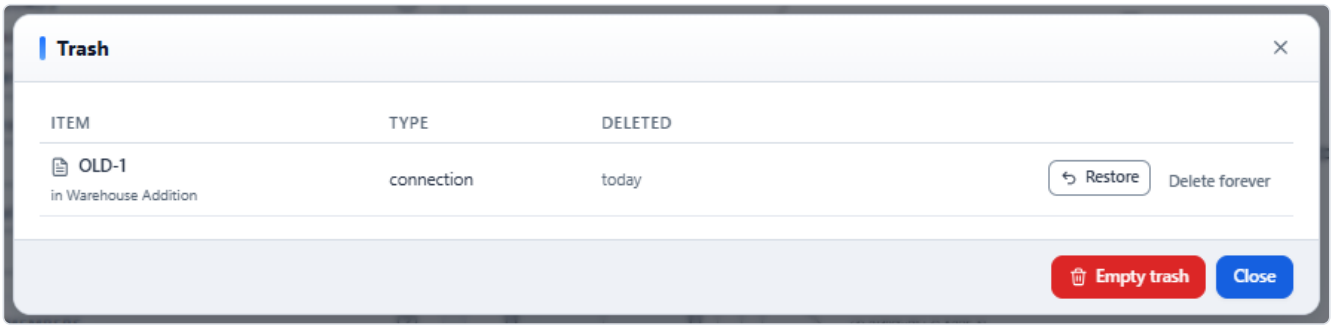

- **Automatic versions.** While you edit, ILX keeps at most one version every 10 minutes, and up to 50 per connection.
- **Named versions.** Select **Save current as version…** and give a name, for example *Issued for review*. Named versions are never removed automatically.
- **Restore.** Select **Restore** on a version. ILX first saves the current inputs as a version, so a restore can always be undone.
- **Delete.** Remove a version you no longer need.

Open the history from the connection menu (**Version history…**).

> **"Results changed since this connection was saved."** When you open a connection and today's result differs from the result saved with it, ILX shows this banner with the saved and current status and ratio. It means the calculation engine was updated after the connection was last saved. Review the change before you reissue the calculations, then select **Dismiss**.

## The trash

Deleting a project or connection moves it to the trash.




1. Select **Trash** at the bottom of the project panel.
2. Select **Restore** to put an item back. A connection whose project is also in the trash cannot be restored until you restore the project.
3. Select **Delete forever** to remove one item permanently, or **Empty trash** to remove everything. These cannot be undone.

## Import and export

Project files are JSON files (`*.ilxproj.json`). Use them to archive an issue, move work between servers or send a project to a colleague.

| To… | Do this |
|---|---|
| Export a project | Project menu → **Export project file**. The file holds the project information, defaults, settings overrides and every connection. |
| Export some connections | Connection menu → **Export file**, or select several and choose **Export** in the bulk bar. The file imports as a new project. |
| Import | Select **Upload** (Import project file) at the top of the project panel and choose the file. It becomes a new project in your account. |

ILX imports project files from every earlier version (formats 1 to 4) and upgrades older connection inputs as it reads them. The file size limit is 5 MB. A file with errors is refused, and the errors are listed.

## Backups

- **Nightly backups.** The server keeps a backup of the whole database every night, and keeps the last 14. They live in the server's data volume (`/data/backups`). Ask your administrator to restore one.

- **Full database download.** Administrators can download the whole database at `/api/backup` on the ILX address while signed in. The file holds every user's data, so store it securely.
- **Archive at issue.** For a durable record of what was issued, export the project file and keep it with the issued PDFs.

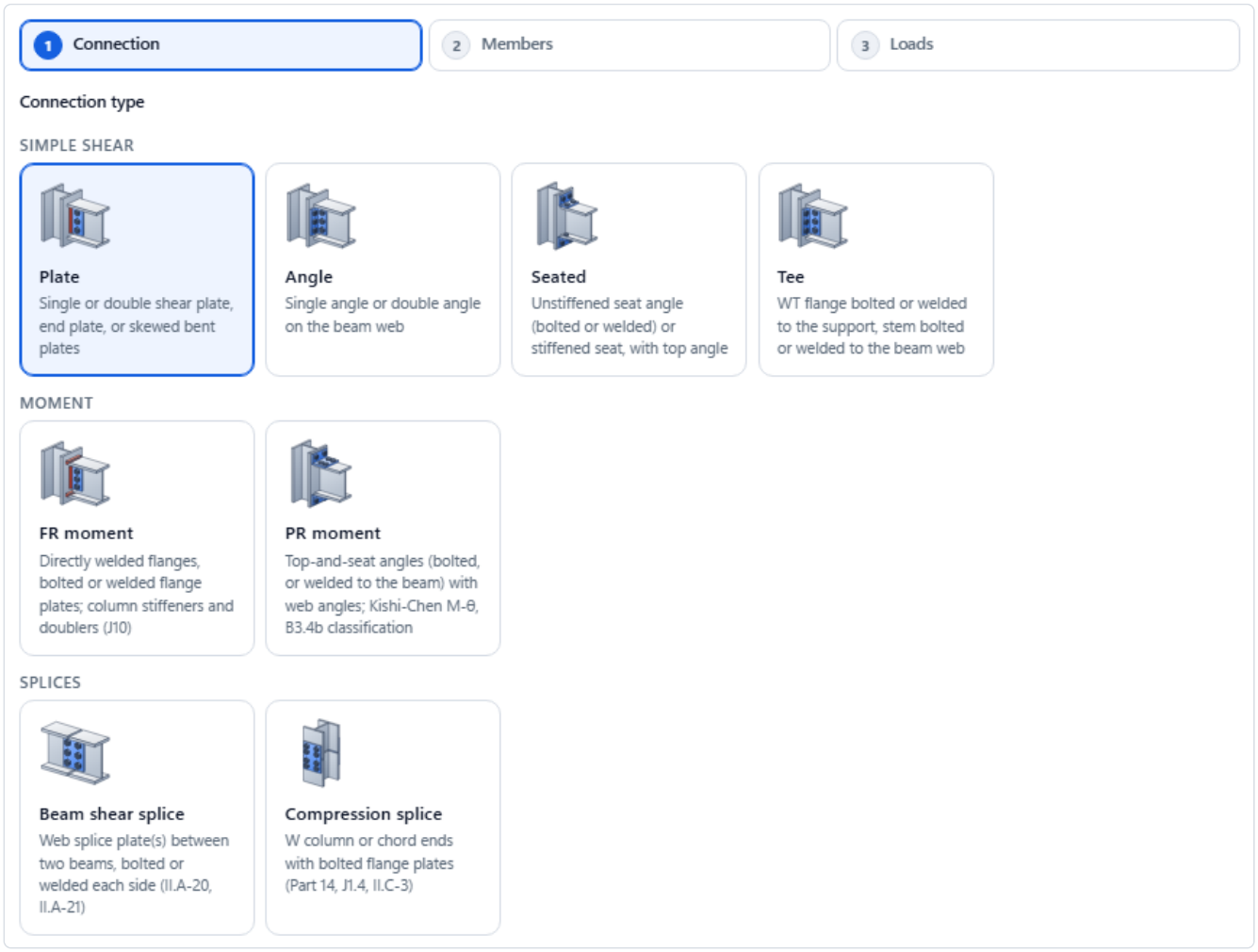
CHAPTER 4

# Designing a single-plate connection

A single-plate connection (shear tab) is a plate shop-welded to the support and bolted or welded to the beam web. This page also describes the editor workflow and the loads, bolts, copes and auto-design that the simple shear connection types share.

## Choosing the connection type

Select **New connection…** in a project's menu. The wizard's first step, **Connection type**, groups the types:




| Group | Type | Mark | Page |
|---|---|---|---|
| **Simple shear** | **Plate**, then **Plate type**: **Single plate** (shear tab, conventional or extended, or all-welded plate) or **Double plate** | ST- | this page |

| Group | Type | Mark | Page |
|---|---|---|---|
| | **Plate**, then **Plate type**: **End plate** (shear end plate, shop-welded to the beam end, bolted to the support) | EP- | Shear end plate |
| | **Plate**, then **Plate type**: **Skewed bent plate** (skewed beam to a girder web) | SK- | Skewed bent plate |
| | **Angle**, then **Angle type**: **Single angle** (bolted to the support, bolted or welded to the beam web) | SA- | Single angle |
| | **Angle**, then **Angle type**: **Double angle** (all-bolted, bolted / welded or all-welded) | DA- | Double angle |
| | **Seated**: unstiffened (bolted or welded) or stiffened seat, with a top angle | SE- | Seated |
| | **Tee**: shear tee, WT flange to the support, stem to the beam web | TE- | Shear tee |
| **Moment** | **FR moment**: directly welded flanges, or bolted or welded flange plates; column stiffeners and doublers | MC- | FR moment |
| **Column bases** | **Base plate**: column base plate on a pier or footing, with anchor rods | BP- | Base plate |

1. Select the type card. Its tooltip gives the short description. **Plate** and **Angle** cover several types: their first option row, **Plate type** (Single plate, Double plate, End plate, Skewed bent plate) or **Angle type** (Single angle, Double angle), picks the type. Outside the wizard (file tree, editor, reports, batch) each type keeps its own name, for example *Shear end plate* or *Double angle*.
2. Select the **Joint type** (not for FR moment): **Beam to column flange**, **Beam to column web**, **Beam to girder web** or **Beam to HSS column**. A type that cannot use a joint does not offer it, for example a single angle on an HSS column or a seat on a girder web.
3. Select the **Configuration** card of the type. Cards that cannot be built on the chosen joint are greyed out, with the reason in the tooltip.
4. Continue with **Members** and **Loads**, as described in Projects and connections.

Every option card shows a small picture of the choice (plate type, joint, configuration, member or layout); only method choices, such as the instantaneous-center or elastic method of a bolt or weld group, have none.

Switching type in the wizard keeps the joint, the members and the loads entered so far, and starts the rest from that type's defaults (and the project defaults for that type). **Base plate** opens its own wizard (column, pier, rods). The editor, the drawings, the checks, the report, the drawing sheet, batch design, compare, templates and project defaults work for every type.

## The editor workflow

The editor designs the connection live. Every change re-runs the full design, about a second after you stop typing, and updates the status, drawings and checks.

1. **Set up.** Choose **LRFD** or **ASD** and **US** or **Metric** in the toolbar.
2. **Configuration.** Choose the configuration and, for Weld / Bolt, the procedure.
3. **Loads.** Enter the loads.
4. **Members.** Pick the beam and the support.
5. **Details.** Set the bolts, the plate, copes and welds. Start from the defaults; ILX flags what needs to change.
6. **Fix and finish.** Read the status bar and fix what fails (see Checks and compliance). Use **Auto-design** to let ILX search for a design.

7. **Produce the outputs.** Create the report and drawing sheet.

The sections below follow the order of the input panel.

## Choosing the configuration

1. Select one of the two configuration buttons. The first word is the support side, the second the beam side.
2. For Weld / Bolt, choose the **Procedure**.

| Configuration | Element | Support side | Beam side |
|---|---|---|---|
| **Weld / Bolt** | single plate (shear tab) | shop-welded, both sides of the plate (or one side, see One-sided support welds) | bolted |
| **Weld / Weld** | single plate | welded | welded to the web (C-shape, vertical or top and bottom) |

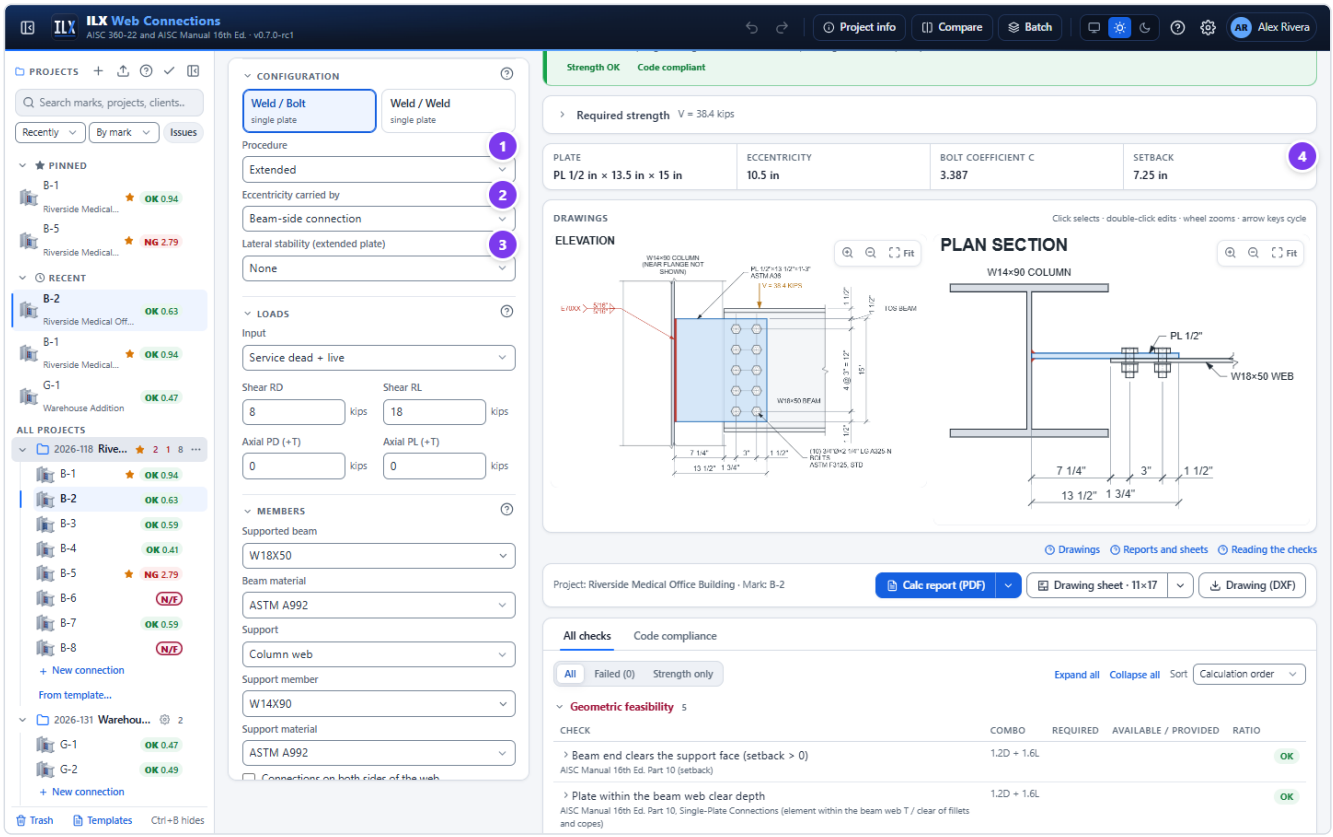
In the wizard, **Shear tab** and **Extended shear tab** are Weld / Bolt with the conventional or extended procedure, and **All-welded plate** is Weld / Weld.

Single angles (an angle bolted to the support) are a connection type of their own; see Designing a single-angle connection.

### Conventional or extended (Weld / Bolt)

| | Conventional (Table 10-9) | Extended |
|---|---|---|
| Basis | Manual Part 10, Table 10-9 as corrected by Muir and Thornton (2011) | Muir and Hewitt (2009), Manual Part 10 |
| Bolts | 2 to 12 rows, one vertical row, s = 3 in. | Any number of rows, up to 3 vertical rows |
| Holes | STD or SSL horizontal only | Any hole type |
| Distance a | a ≤ 3½ in. | Any |
| Eccentricity | Per Table 10-9 (a/2 or a) | Carried by the beam side, the support side or both |
| Plate thickness | tmax by Table 10-9 | tmax by Mmax (with the Muir and Hewitt exemption), plate buckling, stabilizer requirement |
| Edge distance | Leh ≥ 2d on the plate and beam web | Only part of the tmax exemption |

Use the extended procedure when the plate must reach past a column flange into a column web, or past a girder flange, or when the conventional limits cannot be met.




1. **Procedure**: **Extended**.
2. **Eccentricity carried by**: **Beam-side connection** (the usual assumption), **Support-side connection** or **Both (conservative)**. This field also appears for the Weld / Weld configuration.
3. **Lateral stability (extended plate)**: **None**, **Top flange restrained**, **Both flanges restrained** or **Stabilizer plates**. This feeds the stabilizer requirement of the extended procedure.
4. **Beam end braced (Spec. Appendix 6.3)**. Ticked (default): plate flexure in Manual Eq. 10-8 uses Cb = 1.84, which the Manual allows only when the beam end is braced per Spec. Appendix 6, Section 6.3. Cleared: the Manual gives no Cb, so ILX shows **Cb, plate flexure with the beam end not braced**, your input (default 1.0, Spec. F1: a cantilever with an unbraced free end). The report reads "Cb = 1.00 (engineer input; beam end not braced per App. 6.3)" and the input table lists it. ILX also adds a recommendation quoting the Manual (design check 5): for unbraced beams the plate strength and the beam's lateral-torsional buckling strength interact (Cheng and Yura 1988), and ILX does not check that interaction. The report states which Cb was used and why. Conventional procedure: the setting applies only when there is axial load (Manual Part 12, p. 12-7, ties Cb = 1.84 to a braced beam end for single plates under shear and axial force); with shear only the conventional plate keeps Cb = 1.84 (Design Example II.A-17B).
5. The key geometry strip shows the resulting plate, eccentricity, bolt coefficient C and setback.

## Double plate

A double plate puts one plate on each face of the beam web. Each plate is welded to the support with fillets on both of its faces, and the bolts pass through plate, beam web and plate, so they are in double shear.

1. In the wizard, under **Plates**, choose **Double plate**. In the editor, set **Plates** to **Double plate** in the Configuration section. The configuration must be Weld / Bolt.

2. **Fit-up tolerance**: the clear gap between the plates is the beam web thickness plus this tolerance (default 1/16 in. in total). A filler, if entered, is added to the gap.
3. A double plate cannot be used on an HSS wall, with a one-sided support weld or with a welded beam side. The editor removes the HSS wall from the support list and tells you why.

The AISC Manual has no table or procedure for double plates. ILX uses the eccentric design of the extended single plate, and the report states this design basis:

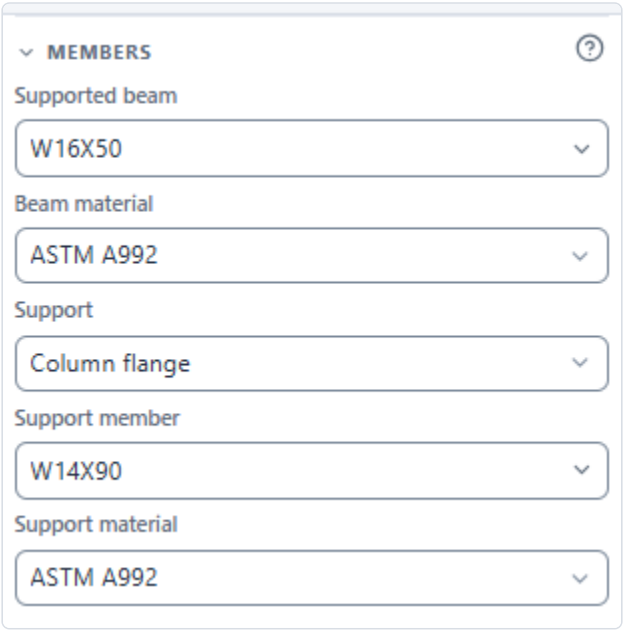
| Item | Double-plate design basis |
|---|---|
| Bolts | Double shear (two shear planes). Bearing and tearout on the beam web for the full force and on each plate for half. The bolt group carries the full eccentricity, from the weld line to the bolt-group centroid (instantaneous center method). |
| Not used | The Table 10-9 conventional shortcuts and the tp ≤ d/2 + 1/16 (tmax) ductility rule. The stiffer double plate needs the eccentric design. |
| Plates | Each plate takes half the force: shear yielding and rupture, block shear, flexural yielding and rupture with the shear interaction, buckling and lateral stability, at the bolt line and at the support face. |
| Welds to the support | Fillets on both faces of each plate, designed for strength for half the force with the eccentric moment. The (5/8) tp rule is not applied. |
| Support and beam | The full force: support shear rupture over the combined weld groups, web punching and yield line over both plates, beam web shear, block shear and coped-beam checks. |

The checks of each plate end in "(each plate, R/2)". Four detailing items are reported for a double plate:

- **Gap between the plates**: the tw + tolerance gap, and a note to shim when the gap is larger.
- **Inside fillets**: the two inside fillets must fit in the gap (2w ≤ gap) and must be reachable for welding. Confirm the access with the fabricator.
- **Beam end clears the inside fillets**: the setback must be larger than the weld size.
- **Erection** (a note, not a warning): the plates project past the beam end, so the bottom flange may land on them as the beam is lowered. The beam can often be worked into place. If not, cope the bottom flange over the plates, slide the beam in along its axis, or ship one plate loose and field-weld it. The note gives the cope length needed.

The drawings show both plates: the elevation calls out "2 PL", the plan section shows the plates on both faces of the web with the bolts through all three plies, and a **Section A-A** (on screen and in the report) shows the plates, the gap and the welds on both faces of each plate.

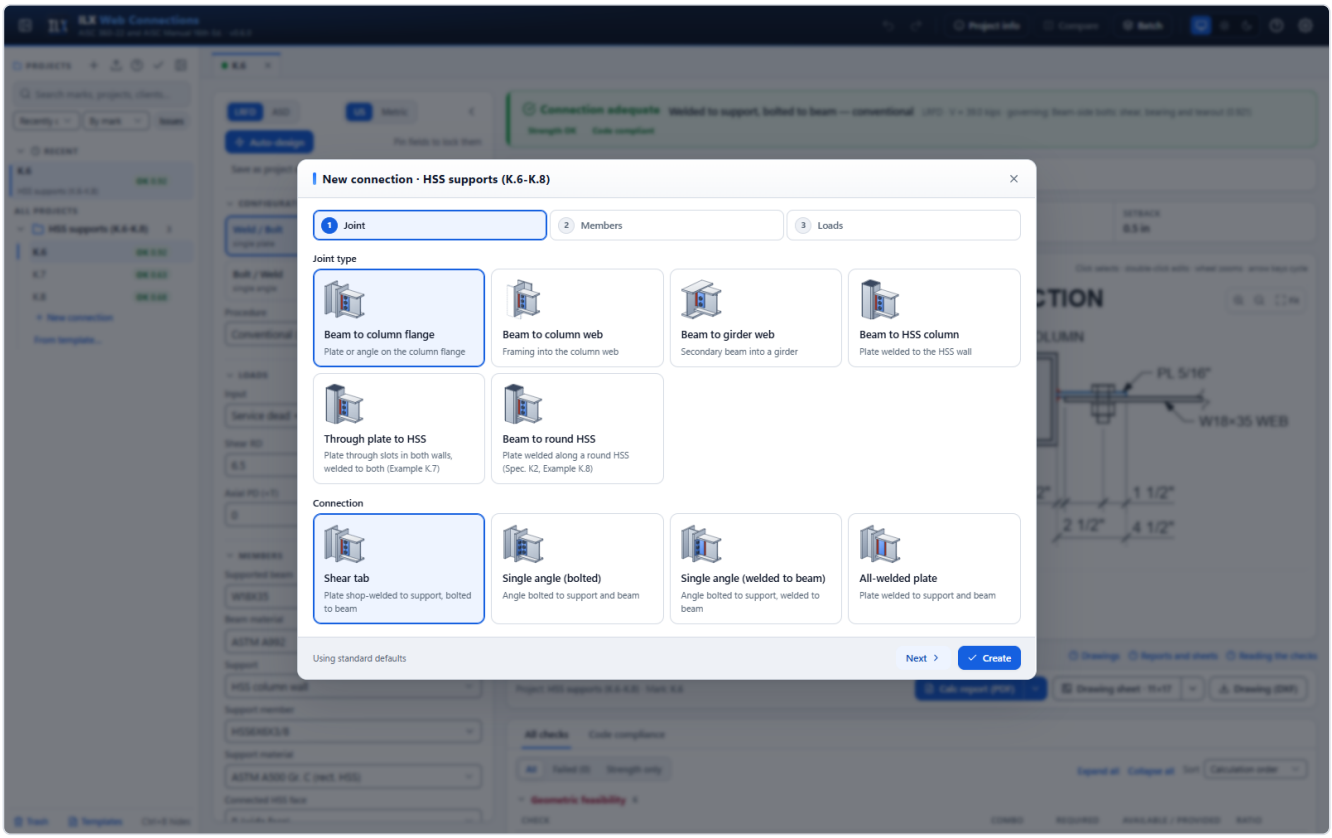
## Members and supports




1. **Supported beam.** Type to search the AISC W shapes (`w16x50`, `W 16 X 50` and `W16×50` all work). Recently used shapes are listed first. In metric, the metric designations are shown.
2. **Beam material**: ASTM A992, A572 Gr. 50, A36, A1085 and so on.
3. **Support**:
   - **Column flange**;
   - **Column web**, drawn with the flanges hidden;
   - **Girder web**, drawn as the girder cross-section;
   - **HSS column wall**, welded configurations only: a plate on the face, a through plate, or a round HSS (see HSS columns).
4. **Support member** and **Support material**. For an HSS wall, choose the HSS shape and an HSS material (A500 Gr. C rectangular or round, A1085, A53 Gr. B), and the **Connected HSS face** where offered.
   When you change **Support**, ILX keeps the input valid and tells you what it changed in a notification, as the new-connection wizard does:
   - **To HSS column wall:** a W-shape member becomes an HSS8X8X1/2 in A500 Gr. C (rectangular).
   - **From HSS to a column or girder:** the HSS becomes a W14X90 (column) or W24X68 (girder), and an HSS material becomes A992.
   - **Between a column and a girder:** the member changes only while it is still the other type's default (W14X90 or W24X68).
   - **Round HSS:** picking a round HSS member switches A500 Gr. C rectangular to round, and back.
5. **Girder TOS − beam TOS (+ = beam lower)**, girder supports only. ILX checks that the beam's top flange clears the girder flange and fillet. If it does not, it proposes a cope as a suggested fix.
6. **Connections on both sides of the web**, for welded connections to a column web or girder web. Tick it when another connection frames into the other face. ILX then uses half the web thickness for this connection (Manual Eq. 9-6 becomes 6.19 D/Fu).

7. **Connection near the column end**, column web or column flange. Cleared (default): the connection is not near the column end, the base case of the Manual and the Specification. Ticked: enter **Column end to the connection force**.
   - Column web: when the distance is less than √(8wab/(a + b)), ILX halves the Eq. 9-45 yield-line strength (Manual p. 9-18).
   - Column flange with an axial force: column web local yielding uses Eq. J10-3 (2.5k + lb) when the distance is ≤ d, and web crippling uses Eqs. J10-5a/b when it is < d/2 (AISC 360-22 J10.2, J10.3). Otherwise Eqs. J10-2 and J10-4.
   - The report shows the limit, the distance and which equation was used, and the input table lists the column-end condition. The 85 % rule of Manual p. 9-17 (yield line noncritical when the load spans more than 85 % of the web width) is applied to column webs only.

## HSS columns: face plate, through plate and round HSS

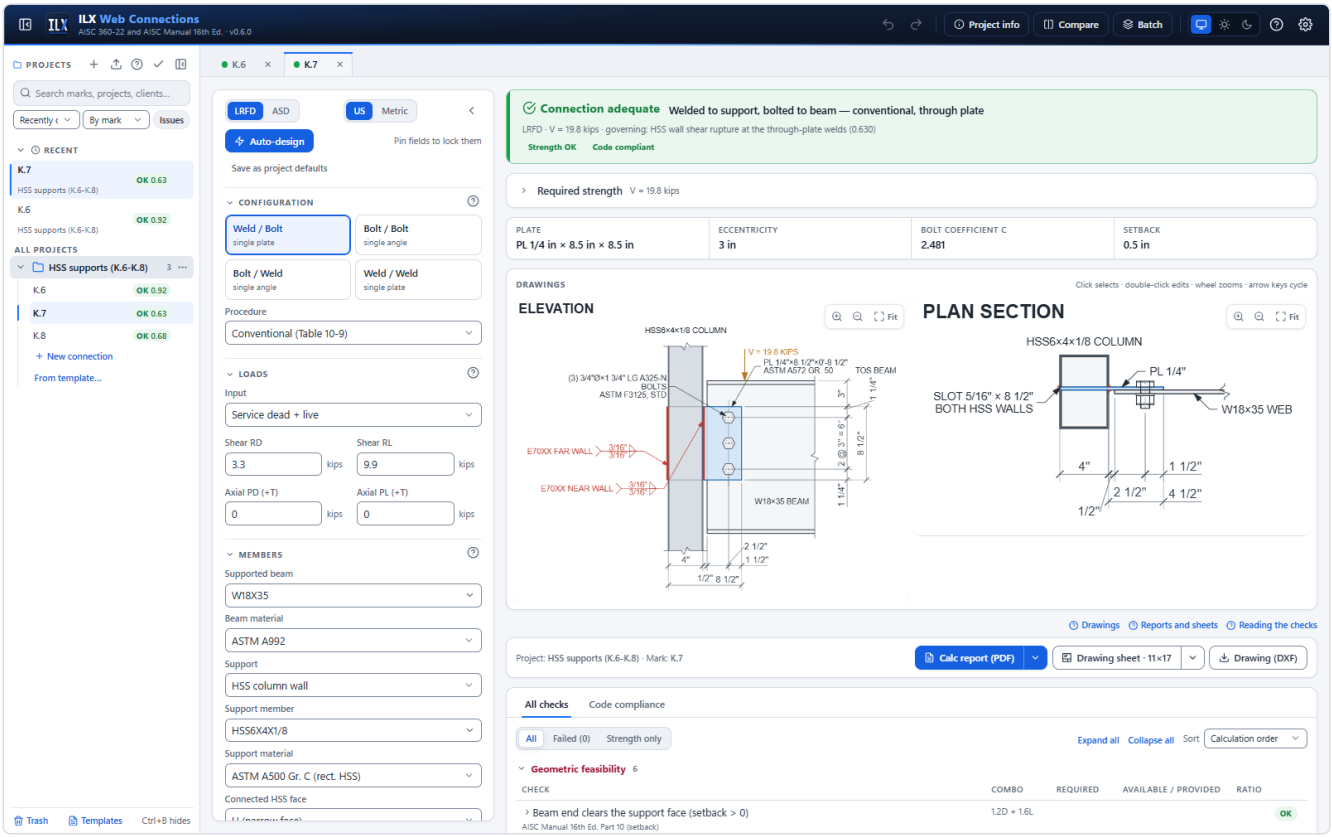



An HSS column takes a welded plate (Weld / Bolt or Weld / Weld). The wizard offers three HSS joint types, and the editor reaches the same three through **Support**, **Support member** and **HSS connection**:

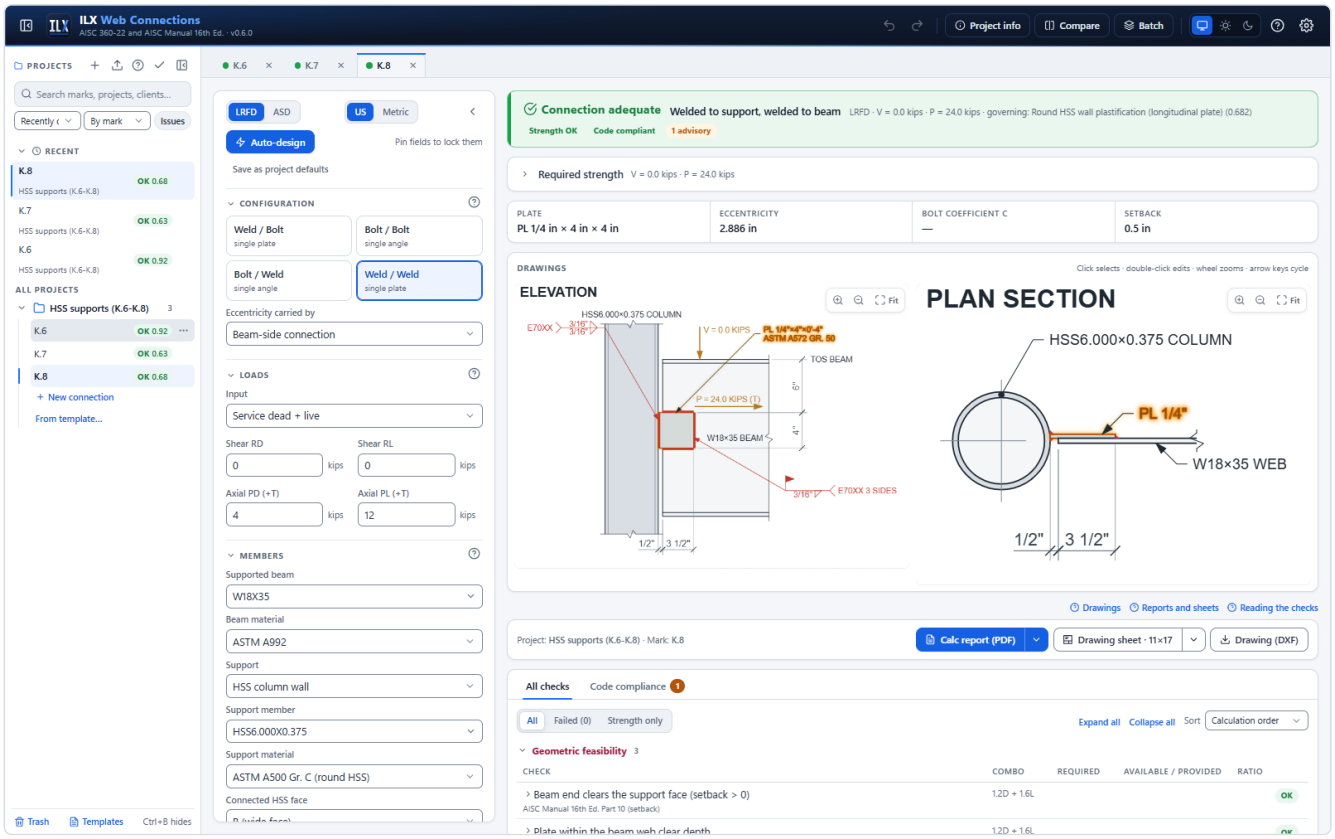
| Joint type | How to get it in the editor | Checks added for the HSS (AISC Design Example) |
|---|---|---|
| **Beam to HSS column**: plate welded to one face of a rectangular HSS | Support **HSS column wall**, a rectangular member, **HSS connection: Plate welded to the HSS face** | b/t ≤ 1.40√(E/Fy) (advisory); tp ≤ Fu t / Fyp (advisory: earlier Manuals; AISC 360-22 Commentary K2.3 says it was removed from the Specification); punching shear Ru e ≤ φ Fu t lp² / 5 (Manual Eq. 10-10, e = HSS face to the bolt group); tmin = 3.09 D/Fu and HSS shear rupture along the welds; wall plastification when there is axial load or the wall is slender (K.6) |

| Joint type | How to get it in the editor | Checks added for the HSS (AISC Design Example) |
|---|---|---|
| **Through plate to HSS**: the plate passes through slots in both walls and is welded to both | Same, with **HSS connection: Through plate (both walls)** | The beam side is a single plate. The near-wall welds and the HSS wall take R (B + a) / B, where B is the HSS dimension the plate crosses (moments about the far wall). ILX checks the weld size, HSS shear yielding and HSS shear rupture at that reaction, and, under axial load, the yield lines of both walls. The face-plate punching and tmin rules do not apply. There are notes on the slot size (tp + 1/16 in.) and on the cost of through plates (K.7) |
| **Beam to round HSS**: plate welded along a round HSS | Pick a round member, for example HSS8.625X0.500. The grade becomes A500 Gr. C (round). | Table K2.1A limits: D/t ≤ 0.11E/Fy, D/t ≤ 50 under axial load, Fy ≤ 52 ksi and Fy/Fu ≤ 0.8 (A500 Gr. C is accepted). Plastification Rn = 5.5 Fy t² (1 + 0.25 lb/D) Qf (Eq. K2-2a) under axial load, with Mn = 0.8 lb Rn. A note gives the minimum distance from the plate to the HSS end, D (1.25 − β/2) (K.8) |

- **Connected HSS face** picks the face of width B or H on a rectangular tube. For a through plate, the plate crosses the other dimension.
- A through plate needs a rectangular HSS. If you pick a round member while a through plate is selected, ILX switches back to a plate welded to the wall and says so.
- Qf = 1.0 throughout, because the HSS axial stress is not an input. Check a column in heavy compression separately.
- A500 Gr. C round HSS uses Fy = 50 ksi, as in ASTM A500-21, Manual Table 2-4 and Design Example K.8.



In the drawings, the part of a through plate inside the tube is drawn hidden in the elevation. The near-wall and far-wall welds each have their own symbol (**NEAR WALL**, **FAR WALL**). The plan section shows the plate running through both walls, with a fillet on each wall and a **SLOT … BOTH HSS WALLS** callout. A round HSS is drawn in plan as a circle section with its centerlines.

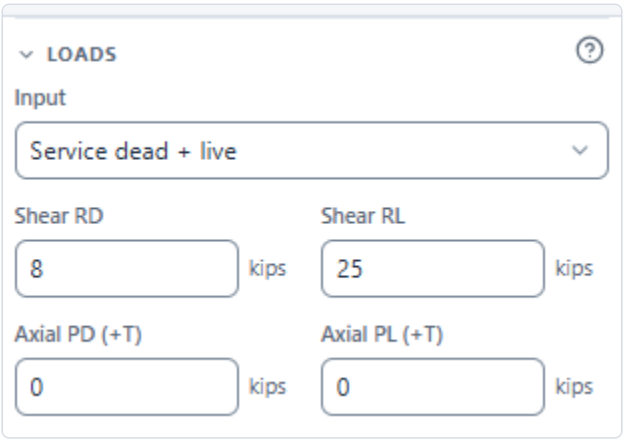



All three joint types are reproduced end to end from AISC Design Examples K.6, K.7 and K.8, in LRFD and ASD, on the Verification page.

## Loads and load combinations

Choose how you enter the loads with **Input** in the **Loads** section. Axial loads are positive in tension (+T). The resulting required strength is shown in the **Required strength** card above the drawings; open it to see the combination worked out.

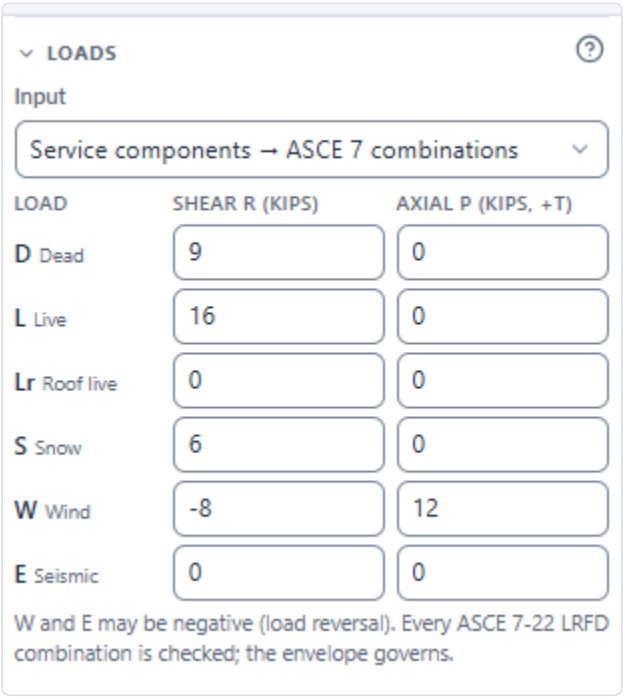
| Input | Enter | Combinations used |
|---|---|---|
| **Service dead + live** (default) | Shear RD, RL; axial PD, PL | LRFD: 1.2D + 1.6L and 1.4D. ASD: D + L. |
| **Service components → ASCE 7 combinations** | D, L, Lr, S, W and E, each as shear R and axial P | Every ASCE 7-22 LRFD (2.3) or ASD (2.4) combination; the envelope governs |
| **Required strength (Ru, Pu)** | The factored shear and axial (Ra, Pa in ASD) | None (used as entered) |
| **Factored load cases (table)** | Named cases, each with V and P | Each case is checked; the envelope governs |

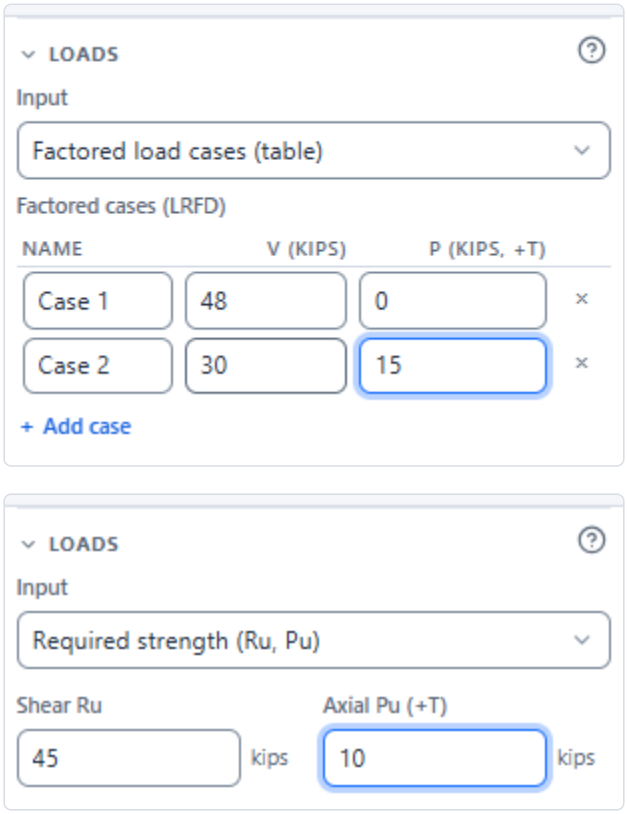



**Required strength** V = 49.6 kips

$$R_u = 1.2\,R_D + 1.6\,R_L = 1.2 \times 8.0 \text{ kips} + 1.6 \times 25.0 \text{ kips} = \mathbf{49.6 \text{ kips}}$$

*ASCE 7 load combination 2*






**With ASCE 7 combinations or load cases**

- **Combination table.** A **Load combinations** table under the output bar lists every combination with its V, P, ratio and status, and marks the one that **governs**. See Load combinations in the results.
- **Reversal.** W and E may be negative to model load reversal. A combination with net compression is checked as compression, with the effective length factor **K (plate in compression)**, which defaults to 1.2.
- **Combo column.** Each check shows the combination that governs it in the **Combo** column.

**Axial load**

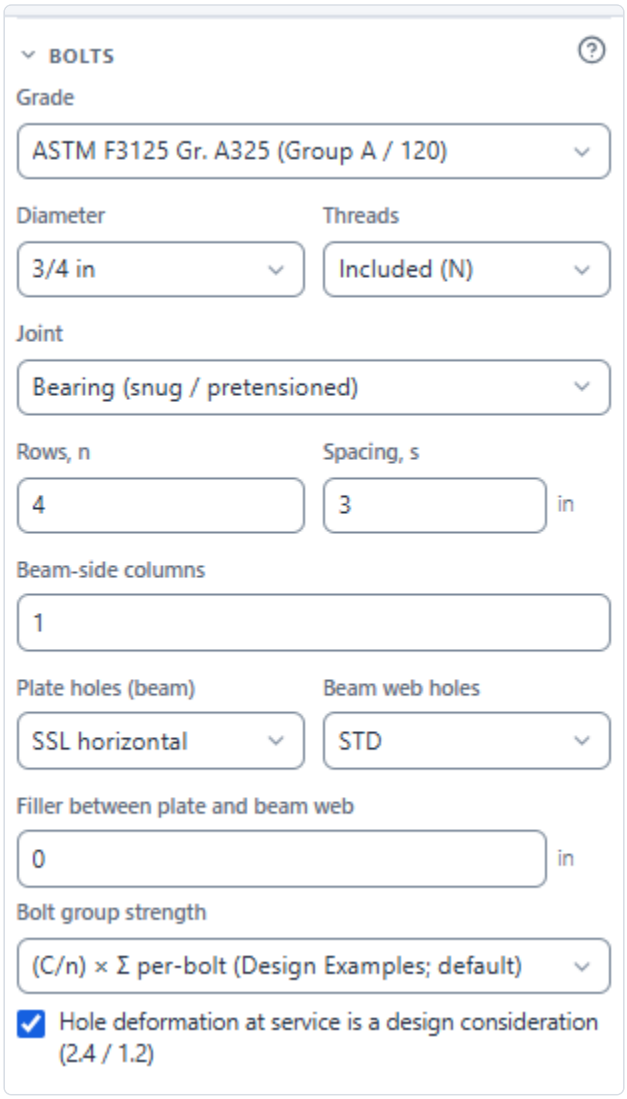
Axial tension adds these checks:

- combined tension and shear on the bolts (J3.7);
- the plate in tension and block shear (U and L shapes);
- the beam in tension (D2/D3);
- the interaction equations of Manual Part 12.

Compression adds plate compression with Lc = K a, and the column-web checks. **K (plate in compression)** appears when the net axial load is compressive.

The wizard's loads step offers only service D + L and required strength. Use the editor for the other modes.

## Bolts and holes




| Field | Options and notes |
|---|---|
| **Grade** | ASTM F3125 Gr. A325 and F1852 (Group A / 120), Gr. A490 and F2280 (Group B / 150), F3043 / F3111 Gr. 2 (Group C / 200), A307 Gr. A, and F1554 Gr. 36 / 55 / 105 or A36 threaded rod |
| **Diameter** | 1/2 in. to 1-1/2 in. (Group C: the sizes the standard lists) |
| **Threads** | **Included (N)** or **Excluded (X)**. A307 is N only. |
| **Joint** | **Bearing (snug-tight)**, **Bearing, pretensioned** or **Slip-critical** (see Bolt rules for every type) |
| **Surface** (slip-critical) | **Class A (μ = 0.30)** or **Class B (μ = 0.50)** |
| **Number of fillers** (slip-critical) | 0 to 4, for the hf factor |
| **Rows, n** and **Spacing, s** | The vertical bolt rows and their pitch |
| **Beam-side columns** and **Gage** | 1 in the conventional procedure; up to 3 in the extended procedure |
| **Plate holes (beam)**, **Beam web holes** | **STD**, **OVS**, **SSL horizontal**, **SSL vertical**, **LSL horizontal**, **LSL vertical**. The conventional procedure allows STD and SSL horizontal only. |

| Field | Options and notes |
|---|---|
| **Filler between plate and beam web** | Applies the AISC 360 J5.2 bolt-shear reduction when thicker than 1/4 in. |
| **Bolt group strength** | See below |
| **Hole deformation at service is a design consideration** | Ticked: bearing 2.4 d t Fu and tearout 1.2 lc t Fu. Cleared: 3.0 and 1.5 (J3.11). |

"Horizontal" and "vertical" give the slot direction on the drawing. Under vertical shear, a horizontal slot is transverse to the load.

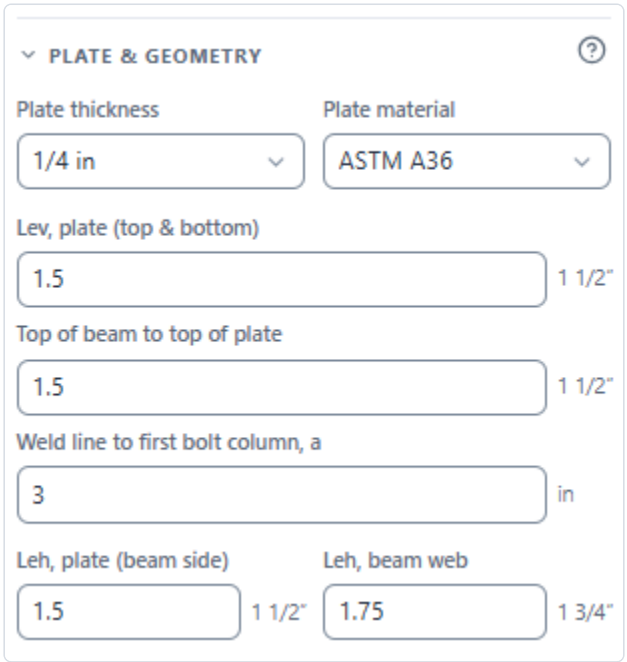
**Slip-critical joints.** ILX checks slip resistance with Du = 1.13, hf from the number of fillers, and φ (or Ω) by hole type per J3.9. It also checks bearing at the strength load. Oversized holes, and slots parallel to the load, require slip-critical bolts. ILX flags them in the code-compliance list if the joint is bearing-type.

### Bolt group strength

| Option | Strength of the eccentric group | When to use it |
|---|---|---|
| **(C/n) × Σ per-bolt (Design Examples; default)** | Each bolt takes the least of shear, bearing and tearout; the sum is multiplied by C/n | The method of AISC Design Examples v16 II.A-17A and II.A-18 |
| **C × weakest bolt (Muir & Thornton; conservative)** | C times the strength of the weakest bolt | The papers' method; conservative |
| **C·rnv shear + Σ bearing/tearout (split)** | Bolt shear as C rnv, bearing and tearout summed concentrically | Matches some commercial programs; not a Manual method |

C comes from a general instantaneous-center solution (Manual Part 7) at the true load angle, verified against Tables 7-6 and 7-7. Tearout uses lc measured along the line of action of the resultant (Design Example II.A-1B).

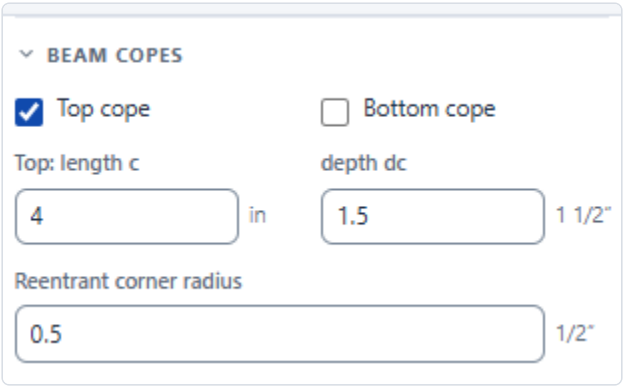
## Plate and geometry



| Field | Meaning |
|---|---|
| **Plate thickness**, **Plate material** | 3/16 in. to 1-1/4 in.; A36, A572 Gr. 50 and so on |
| **Lev, plate (top & bottom)** | Vertical edge distance at the top and bottom bolts. With bolts, the plate depth follows from n, s and Lev. |
| **Plate depth, L** | Weld / Weld only, where there are no bolts |
| **Top of beam to top of plate** | Sets where the plate sits on the web; it must clear the flange, fillet and any cope |
| **Weld line to first bolt column, a** | The distance a of Manual Part 10 |
| **Leh, plate (beam side)**, **Leh, beam web** | Horizontal edge distances on the plate and the beam web |
| **Beam setback** (welded beam side) | The gap between the support face and the beam end |
| **Plate lap on beam web** | Weld / Weld: how far the plate overlaps the beam web |

The setback and plate width follow from a, Leh and the configuration, and are shown in the key-geometry strip under the status bar.

## Beam copes

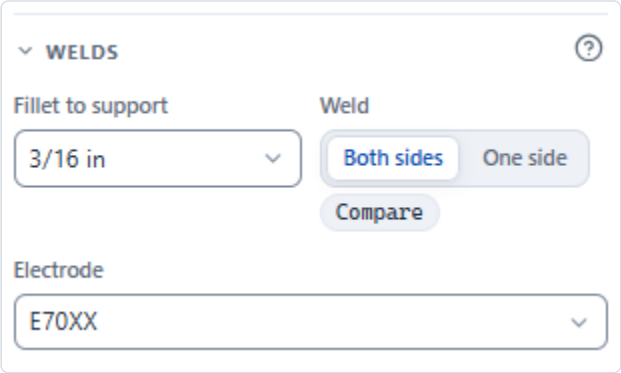



1. Tick **Top cope**, **Bottom cope** or both.
2. Enter each cope's length **c** and depth **dc**.
3. Enter the **Reentrant corner radius**. AISC 360-22 M2.2 (thermal cutting) requires a curved transition at reentrant corners; the field's hint and the check both cite it. (J1.6 covers weld access holes, not cope radii.) ILX checks:
   - that the radius is at least 3/8 in. (the M2.2 User Note);
   - that the radius fits within the cope;
   - with **Cyclic loading** ticked, the Appendix 3 radius, and the fatigue stress range at the cope corner (Table A-3.1 Case 1.3); see Fatigue (Appendix 3).

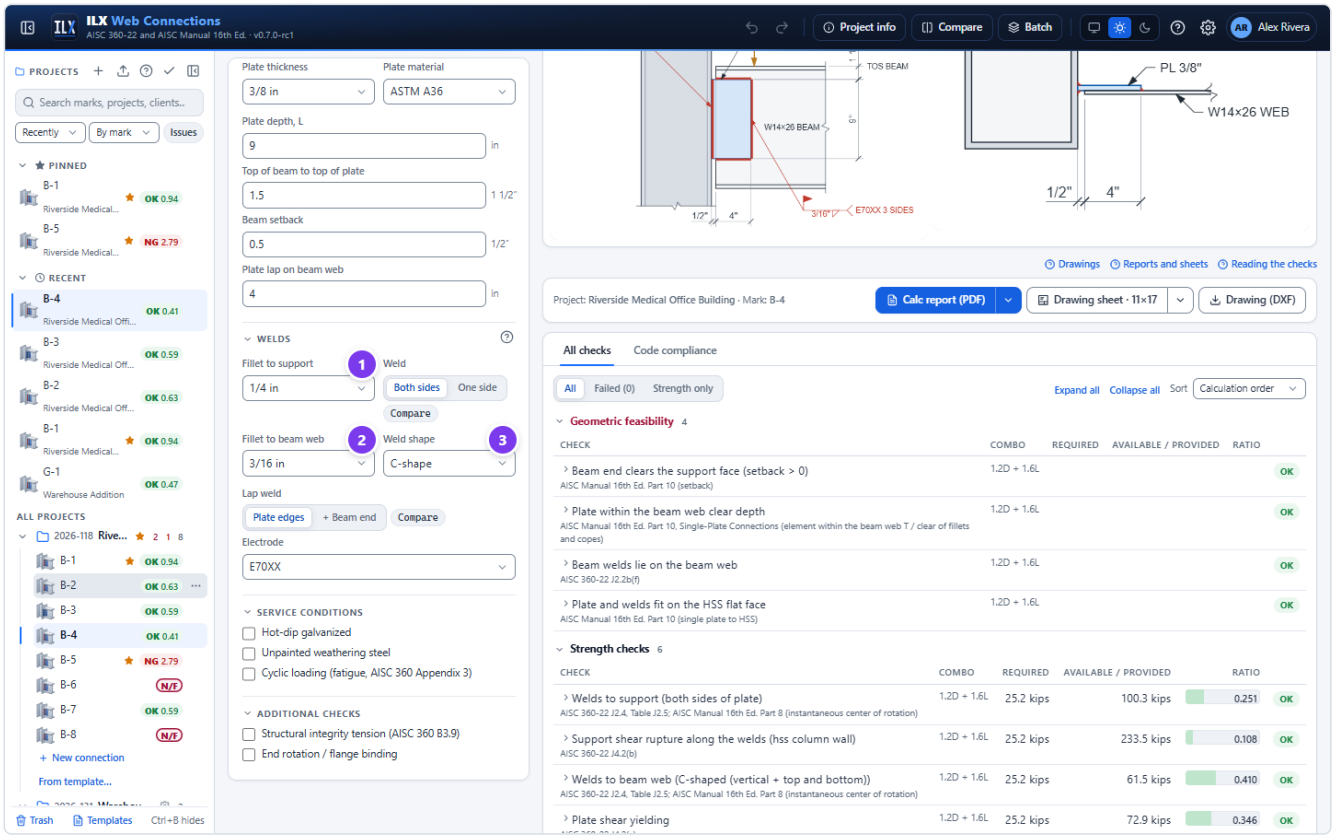
Single copes are checked by Dowswell (2018), the method of the 16th Ed. Manual. Double copes are checked by Dowswell and Whyte (2014), Manual Eqs. 9-21 and 9-22.

The coped section is also checked for shear yielding, shear rupture and block shear. **Hot-dip galvanized** and **Unpainted weathering steel** add the related cope and preheat notes.

## Welds to the support and the beam




| Field | Meaning |
|---|---|
| **Fillet to support** | The fillet weld size at the support. ILX checks w ≥ (5/8) tp for ductility, the minimum and maximum size, the minimum length, the base metal (tmin = 3.09 D/Fu per weld line, Manual Eq. 9-6) and support shear rupture along the welds. |
| **Weld** (support) | **Both sides** (default) or **One side** of the plate. See below. |
| **Fillet to beam web** | The fillet at the beam (Weld / Weld). |
| **Weld shape** | **C-shape**, **Vertical** or **Top & bottom** (welded beam side). Eccentric weld groups use the instantaneous-center method of Manual Part 8. |
| **Lap weld** | **Plate edges** (default: fillets along the plate edges on the near face of the web) or **+ Beam end** (adds the fillet along the beam end on the far face of the plate, in the same weld group). Weld / Weld only. |
| **Electrode** | E70XX or E80XX |




1. **Fillet to support**.
2. **Fillet to beam web**.
3. **Weld shape**. The beam-side weld symbol on the drawing reads, for example, **3 SIDES** for a C-shape.

## One-sided support welds

**Weld** sets whether the plate is welded to the support on both faces or on one face only. **Compare** next to it designs the other option and shows both governing ratios, for example *both: 0.72 OK · one: 1.31 NG*.

With **One side**, ILX:

- checks one fillet line instead of two, with kds = 1.0 (AISC 360-22 J2.4, "all other conditions");
- treats the weld as an eccentric weld group (IC method, Manual Part 8) with an out-of-plane eccentricity tp/2 + w/3, combined with any in-plane eccentricity, even in the conventional procedure. AISC publishes no formula for this; it is ILX's application of the Part 8 method;
- shows w ≥ 2 × (5/8) tp as an advisory, not a failure. This is ILX's derivation from the Manual's both-sides 5/8 tp rule;
- fails the connection when it carries axial tension or the B3.9 structural-integrity tension, because tension across a one-sided fillet is not permitted (Manual Part 8, Spec Commentary J2.2b);
- adds an advisory that the Manual Part 10 procedures assume both faces are welded;
- uses tp + w as the plate footprint for the web yield-line, punching and HSS checks.

The weld symbol is drawn on the arrow side only, and the plan section shows one fillet. With a vertical-only lap weld, axial tension and **Plate edges**, the check on the lapped ends (J2.2b(f)) fails and suggests **+ Beam end**.

## Skewed plate and sloped beam

The **Skew and slope** section covers the two non-rectangular cases in Manual Part 10, "Non-Rectangular Simple Shear Connections" (pp. 10-86 to 10-95).

### Skewed single plate

A skewed single plate is welded to the support at the beam's skew angle (Manual Fig. 10-38). It is available for a single plate welded to the support on both faces and bolted to the beam. The Manual says it is designed like a square single plate (p. 10-90), so every single-plate check still runs. The skew changes the welds to the support and adds the checks below.

| Input | Default | Meaning |
|---|---|---|
| **Skew of the plate from square** | 0° | 0 to 45°. 0 is the square plate. |
| **Plate end at the support** | Square edge | **Square edge** is Fig. 10-38(a), the preferred detail. It leaves a root opening tp sin(skew) on the obtuse side. **Beveled to the skew** is Fig. 10-38(b), with no root opening. |
| **Obtuse-side fillet** | Same as acute | When skewed, **Fillet to support** is the fillet in the acute angle and this is the fillet in the obtuse angle. |

**What changes:**

- **Weld strength.** Each fillet gets the effective throat of a skewed T-joint (AWS D1.1 Annex II; Spec J2.2a): te = (w − R) / (2 sin(Ψ/2)).
  - Ψ = 90° − skew on the acute side and 90° + skew on the obtuse side.
  - Root openings under 1/16 in. are ignored.
  - Below Ψ = 60° the AWS Table 2.2 Z loss is deducted.
  - The weld checks use the comparable 90° fillet size, w = (te,acute + te,obtuse) / (2 × 0.707), wherever the square plate uses the fillet size. That covers the weld group, the 5/8 tp rule, the support base metal and the integrity tie.
- **Root opening.** With a square edge the obtuse root opening must not exceed 3/16 in. (Manual p. 10-89; AWS 5.22.1). If it does, bevel the edge.
- **Skew over 30°.** The Manual limits fillet-welded skewed plates to 30° (6-15/16 in 12) and refers to AWS. Beyond 30° with a square edge, ILX shows an advisory. Table 10-14C gives beveled-edge fillets and a BTC-P4 groove alternate up to 45°.
- **Table 10-14C.** For 5/16, 3/8, 1/2 and 5/8 in. plates, ILX shows the Manual's weld detail for the skew and whether your welds match it. This is advisory. Every Table 10-14C detail passes the throat rule above (tested).
- **Obtuse side over 100°.** A note gives the AWS Fig. 3.11 Detail C preparation.
- **Bolt clearance on the acute side.** The distance from the first bolt column to the support face, measured square to the bolt at the outer face of the nut, is compared with the tightening clearance C1 (Manual Tables 7-15/7-16). The Manual says skewed single plates "should be analyzed" for clearance; this layout check is ILX's.
- **Beam end.** A note gives the clearance for a beam end cut parallel to the support, and for a square cut at the acute flange tip.

The support weld symbol shows both sizes, with the acute fillet on the arrow side, and the skew in the tail.

### Sloped beam

**Beam slope** is the vertical angle of the beam flanges (Manual Fig. 10-34(b)). Positive means the beam rises away from the support. It is available on the single plate, the single angle and the double angle.

The Manual says a sloped connection is designed as the rectangular one, "with consideration of the geometry of the connection to establish the location of fittings and fasteners" (p. 10-90). ILX therefore keeps every strength check unchanged, with the vertical reaction R; R cos(slope) would be the beam shear. It adds:

- **Fit check.** The plate or angles must stay between the sloped flanges, clear of the fillets (kdes / cos slope) or the copes, over their whole length along the beam. The flanges move run × tan(slope) over that length.
- **Note.** The holes in the beam web are on vertical lines matching the square connection material (Fig. 10-43). For a girder support, the note gives how far the beam top moves over the girder flange.

**Not covered:**

- the cope checks still use square-framing geometry, so verify cope depth and length on the layout;
- the elevation is drawn square, with a "BEAM SLOPED" note;
- the axial component of a sloped beam's reaction is not added; enter it as axial load if it applies;
- canted beams (Fig. 10-34(c)) and compound skew-and-slope (Fig. 10-34(d)).

## Service conditions and additional checks

| Option | Effect |
|---|---|
| **Hot-dip galvanized** | Adds the galvanizing notes for copes and reentrant corners |
| **Unpainted weathering steel** | Adds the related detailing notes |
| **Cyclic loading (fatigue, AISC 360 Appendix 3)** | Designs the connection for fatigue (AISC 360-22 Appendix 3) and requires the Appendix 3 cope radius. See Fatigue (Appendix 3). |
| **Structural integrity tension (AISC 360 B3.9)** | Runs the structural-integrity tension check as an independent case |
| **End rotation / flange binding** | Checks that the beam can rotate the required amount before its flange binds on the support. Enter the **Required rotation**, or use the presets **0.03** or **0.025 (341)**. The 0.025 rad preset is for a gravity connection in a braced-frame bay (AISC 341). |

## Fatigue (Appendix 3)

Tick **Cyclic loading** to design the connection for fatigue by AISC 360-22 Appendix 3. These inputs appear:

| Field | Meaning |
|---|---|
| **Stress-range cycles nSR (0 = indefinite life)** | The number of stress-range fluctuations in the design life (Commentary Eq. C-A-3-2: cycles per day × 365 × years). **0** (default) designs for an indefinite life: the allowable range is the threshold FTH, and the root-crack categories C′ and C″, which have no threshold, use 2 × 10^8 cycles (the User Note). 20,000 cycles or fewer: no evaluation is required (Section 3.1) and the result says so. |
| **Cyclic load range** | **Fraction of the service live load** (default) or **Range of shear entered**. |
| **Fraction of the live load that cycles** | The live shear that cycles, as a fraction of RL. **1.0** (default): the full live load comes and goes; **2.0**: full reversal. |
| **Cyclic range of shear** | The range of the required shear between its cyclic extremes, entered directly (kips or kN). |

How the stress range is found: ILX computes each detail's stress at the governing design load by elastic analysis (Section 3.2, no stress-concentration factors; weld groups by the elastic method, with the eccentricity), then multiplies it by k = range / required shear. The other load components (axial) are taken to cycle in proportion. With direct, case or ASCE 7 load input there is no service live load, so the **Fraction** option uses the design load as the range (k = 1, conservative) and every fatigue check says so; enter the range instead for a full check.

Each detail is a check of its own, in a **Fatigue (Appendix 3)** group on screen and a **Fatigue (Appendix 3)** table in the report's Summary of Checks: the detail, its Table A-3.1 case and stress category, the stress range fsr, the allowable range FSR and the result. A range below FTH needs no further evaluation; the calculation says so. The single plate's details:

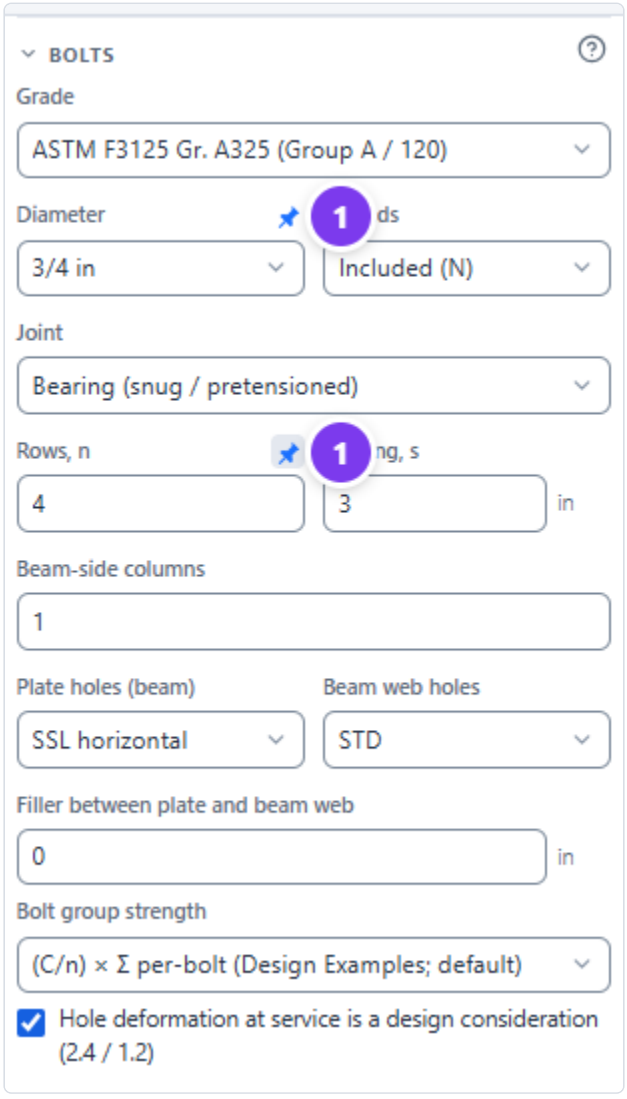
| Detail | Table A-3.1 | Stress range from |
|---|---|---|
| Plate and beam web at the bolt holes | Case 2.1 (slip-critical, B, gross section), 2.2 (pretensioned, B, net section); snug-tight joints are not in Section 2 and are taken as Case 1.5 open holes (D), conservative | V or P over the section along the bolt line |
| Plate-to-support fillet welds | Case 8.2 (F, shear on the throat, Eq. A-3-2) | Elastic weld group, V at the bolt-group eccentricity |
| Plate at the support welds | Case 5.7: the lesser of the toe crack (C) and the root crack (C″, Eq. A-3-5 with RFIL, Eq. A-3-6) | 6 V e / (t L²) + P / (t L) |
| Plate-to-beam fillet welds (welded beam side) | Case 8.2 (F) | Elastic weld group |
| Beam at the cope corner | Case 1.3: C for R ≥ 1 in. ground or reamed (setting), E′ for R ≥ 3/8 in. | M c / I of the coped section, M = V (setback + c) |

The grinding of cope corners and the testing of CJP welds are engineer settings in **Settings → Design rules** (see Settings); their defaults are the Specification's conservative choices. Bolts in shear are not checked for fatigue (Section 3.4(a) checks the connected material; the Commentary notes no fatigue failures of high-strength bolts in shear splices). Not checked: the 0.66Fy limit on the peak stress (the strength checks govern), distortion-induced fatigue and corrosive or hot (over 300 °F) service. RCSC 4.2(3) requires pretensioned or slip-critical bolts in joints under fatigue load; the pretension check flags a snug-tight joint.

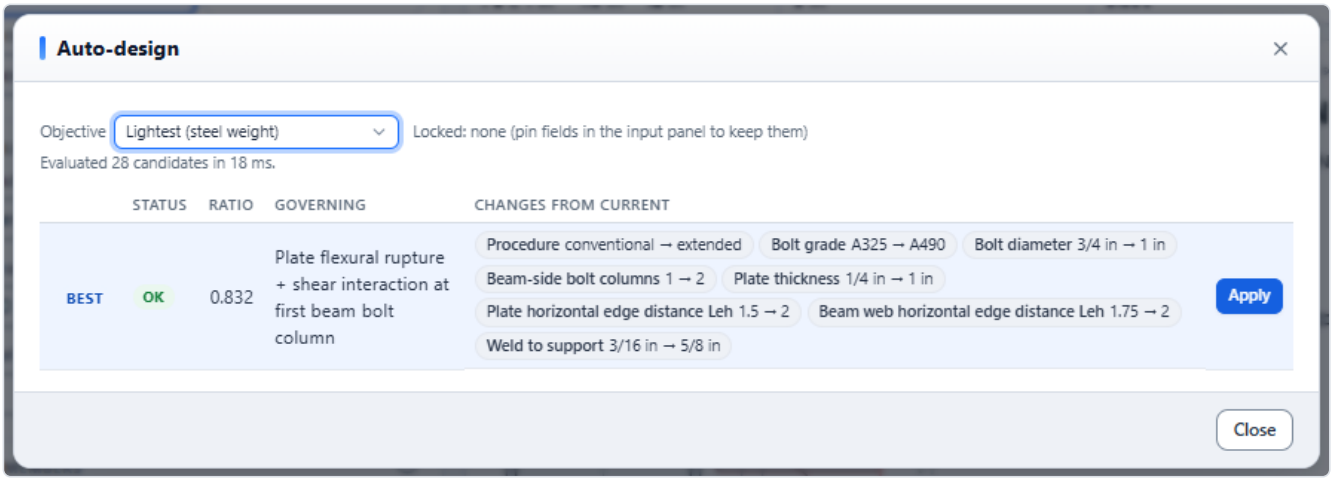
# Auto-design, compare and batch

## Auto-design

Auto-design searches bolt count, diameter and grade, plate thickness, weld size and the geometry, and returns the best adequate designs.




1. **Pin** the fields you want to keep. Hover over a field label and select the pin icon. It stays blue while pinned. Pinned fields are listed in the auto-design dialog as **Locked**.
2. Select **Auto-design** in the toolbar.
3. Choose the **Objective**:
   - **Lightest (steel weight)**;
   - **Fewest bolts**;
   - **Lowest cost (fabrication estimate)**.
4. Review the candidates. Each row shows its status, ratio, governing check and the **Changes from current**.
5. Select **Apply** on the design you want. The inputs change; select **Undo** (Ctrl+Z) to go back.



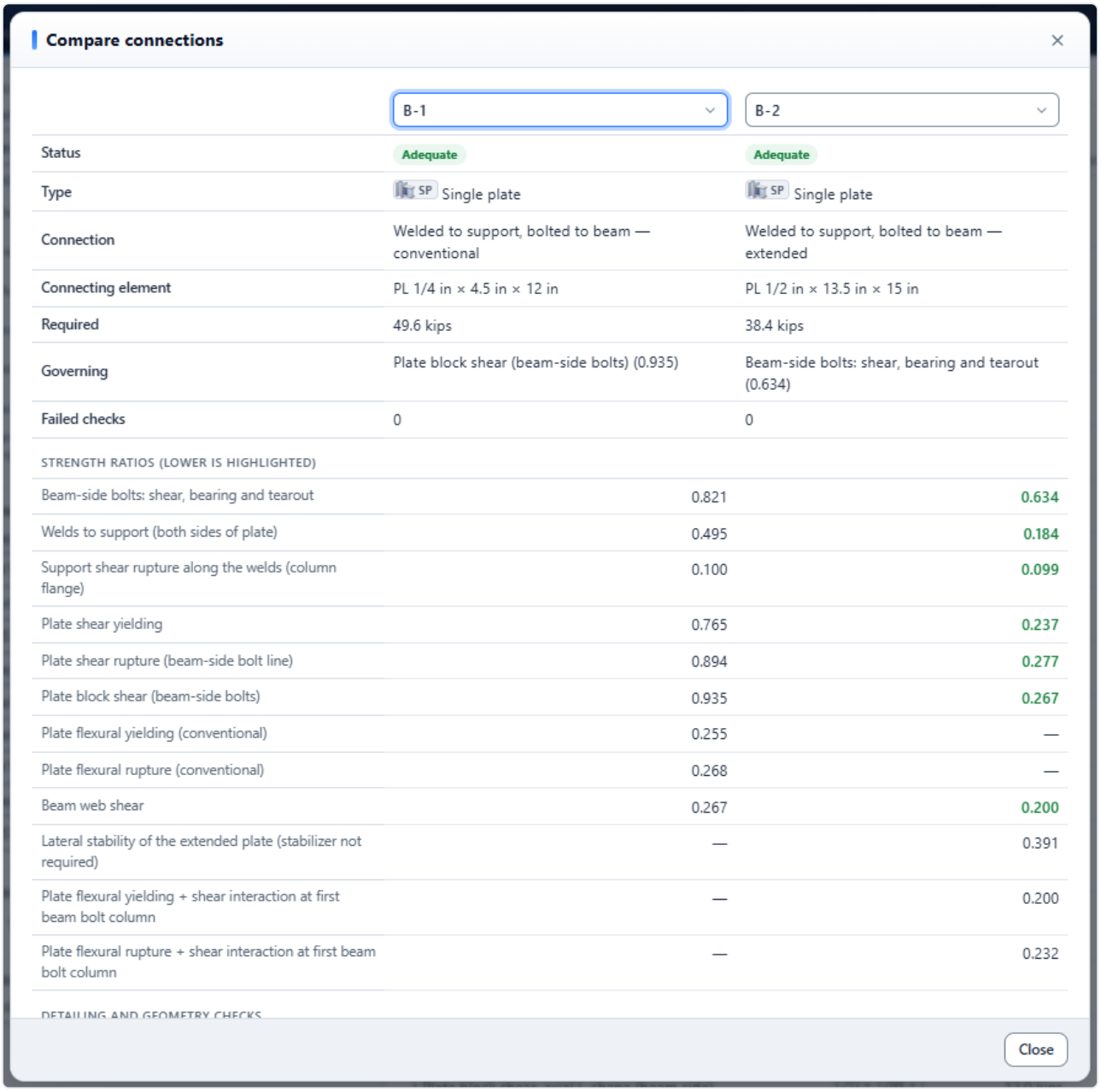

Auto-design can change the procedure (for example from conventional to extended) unless you pin **Procedure**. Check the changes before you apply them. To auto-design a connection as you create it, tick **Auto-design the connection** in the last wizard step.

## Compare

1. Open two or more connections in tabs.
2. Select **Compare** in the header.
3. Pick **Connection A** and **Connection B**.

The table lists the status (**Adequate**, **Not adequate** or **NOT FEASIBLE**), element, required strength, governing check and number of failed checks of each connection. Then come two sections:

- **Strength ratios.** Every strength check side by side. The lower ratio of each pair is highlighted.
- **Detailing and geometry checks.** The detailing and feasibility checks that fail in either connection, as OK / NG / NOT FEASIBLE, with a count of those that pass in both.



Compare connections

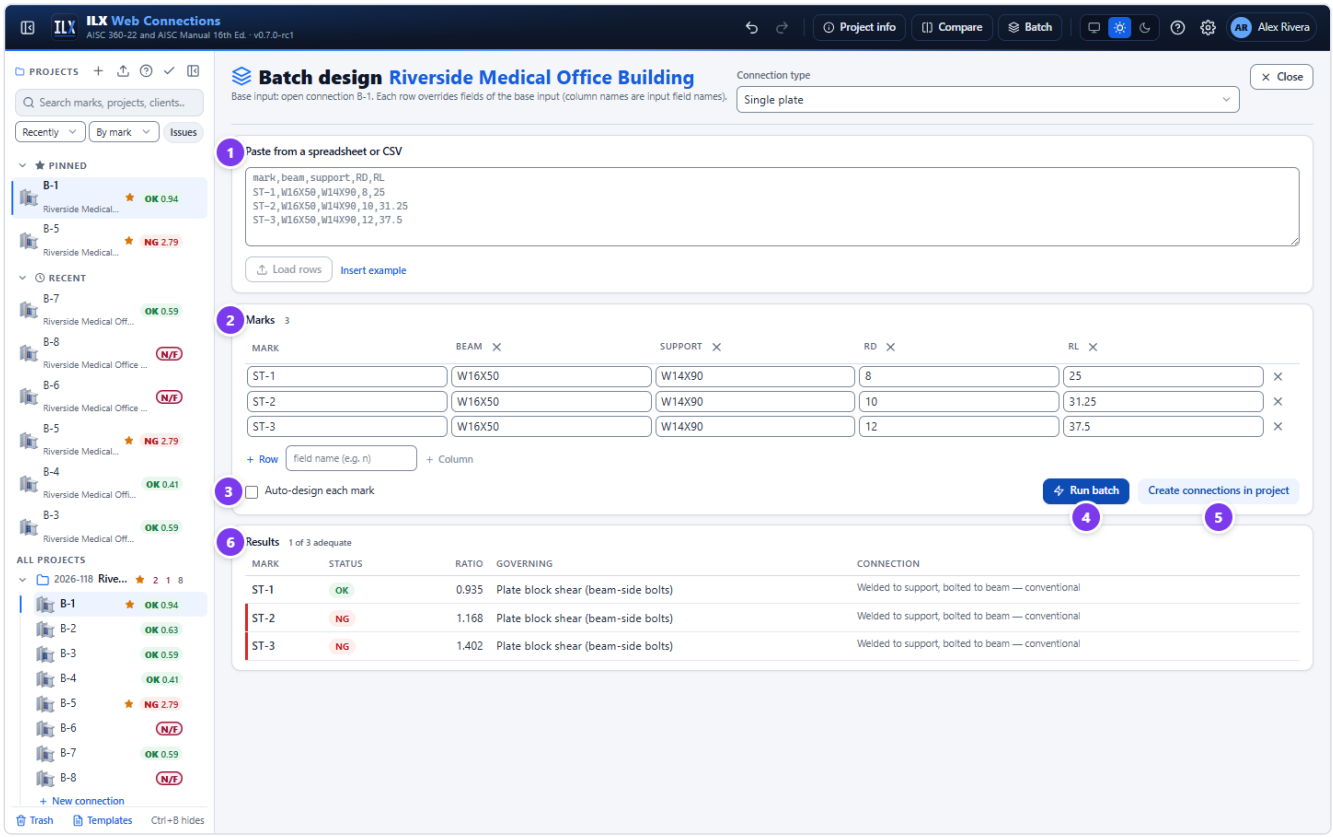
| | B-1 | B-2 |
|---|---|---|
| Status | Adequate | Adequate |
| Type | SP Single plate | SP Single plate |
| Connection | Welded to support, bolted to beam — conventional | Welded to support, bolted to beam — extended |
| Connecting element | PL 1/4 in × 4.5 in × 12 in | PL 1/2 in × 13.5 in × 15 in |
| Required | 49.6 kips | 38.4 kips |
| Governing | Plate block shear (beam-side bolts) (0.935) | Beam-side bolts: shear, bearing and tearout (0.634) |
| Failed checks | 0 | 0 |
| STRENGTH RATIOS (LOWER IS HIGHLIGHTED) | | |
| Beam-side bolts: shear, bearing and tearout | 0.821 | 0.634 |
| Welds to support (both sides of plate) | 0.495 | 0.184 |
| Support shear rupture along the welds (column flange) | 0.100 | 0.099 |
| Plate shear yielding | 0.765 | 0.237 |
| Plate shear rupture (beam-side bolt line) | 0.894 | 0.277 |
| Plate block shear (beam-side bolts) | 0.935 | 0.267 |
| Plate flexural yielding (conventional) | 0.255 | — |
| Plate flexural rupture (conventional) | 0.268 | — |
| Beam web shear | 0.267 | 0.200 |
| Lateral stability of the extended plate (stabilizer not required) | — | 0.391 |
| Plate flexural yielding + shear interaction at first beam bolt column | — | 0.200 |
| Plate flexural rupture + shear interaction at first beam bolt column | — | 0.232 |
| DETAILING AND GEOMETRY CHECKS | | |

Close

## Batch design

Batch design runs a schedule of marks through the same base connection, and creates the connections in one step.

Open it with **Batch** in the header, or **Batch design…** in a project's menu. The base input is the open connection when it belongs to that project; otherwise it is the project defaults. The page header says which.




1. **Paste the schedule.** Paste rows from a spreadsheet or CSV and select **Load rows**. The first row may be a header; column names are input field names, such as `mark`, `beam`, `support`, `RD`, `RL`, `n` or `plate_t`. **Insert example** shows the format.
2. **Edit the grid.** Add or remove rows, or add a column by typing a field name and selecting **+ Column**. Invalid numbers are outlined in red. US lengths accept fractions.
3. **Auto-design (optional).** Tick **Auto-design each mark** to size every mark for its own loads.
4. **Run batch.**
5. **Create the connections.** Select **Create connections in project** to add every mark to the project.
6. **Results.** Each mark's status (**OK**, **NG**, **NOT FEASIBLE** or **Error**), ratio, governing check and connection description. Input errors and notes appear under the governing check.

> **Load columns and the load mode.** Each load mode reads its own columns: `RD`, `RL` (and `PD`, `PL`) for **Service dead + live**, the ASCE 7 components for **ASCE 7 combinations**, and `V`, `P` for **Required strength**. Load columns are never ignored silently:
>
> - If a row's load columns belong to another mode than the base connection's, the row switches to that mode, and the result notes *Load mode switched from Required strength to Service D + L to use column(s) RD, RL*.
> - If the columns mix modes (for example `V` and `RD`), or contradict a `load_mode` column, the row is rejected with an error that names the columns.

CHAPTER 5

# Designing a single-angle connection

A single-angle connection is a simple shear connection made of one angle on one side of the beam web. One leg, the **support leg**, is bolted to the support. The other leg, the **outstanding leg** (OSL), is bolted or welded to the beam web.

The calculation is the single-plate engine's angle path. Every check, field and output described here is shared with the single-plate connection, and only the angle-specific parts differ.

## When to use a single angle

- Use a single angle for light to moderate reactions where a one-sided, field-bolted connection is wanted, for example a beam framing into a girder web that must be erected from one side.
- Use a single plate when the element can be shop-welded to the support, or when the support is an HSS column (an angle bolted to an HSS wall is available, but the element is typically shop-welded to an HSS: Manual p. 10-96).
- Use a double angle for larger reactions or a symmetric connection.
- Use a shear tee when the element should be bolted to the support on both sides of the beam web line.

## Creating a single angle in the wizard

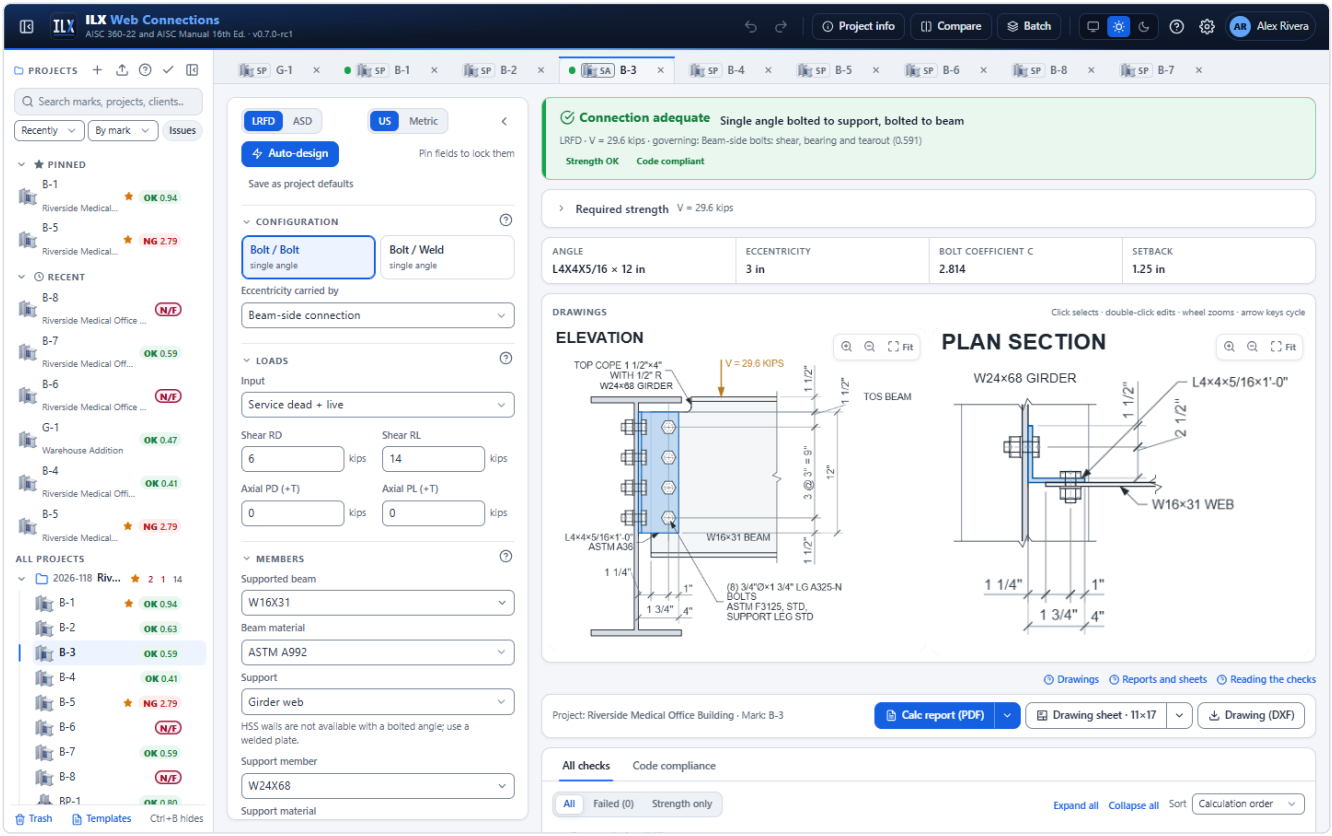
1. In the wizard's **Connection type** step, select the **Angle** card in the **Simple shear** group, then **Single angle** under **Angle type**.
2. Under **Joint type**, select **Beam to column flange**, **Beam to column web**, **Beam to girder web** or **Beam to HSS column**. On an HSS column the angle's support leg is bolted to the HSS wall (see Bolted to an HSS wall); the card that welds the angle to the support is greyed out.
3. Under **Configuration**, select a card:

| Card | Sets | Basis |
|---|---|---|
| **All-bolted angle** | Support leg bolted, outstanding leg bolted to the beam web (Bolt / Bolt) | Manual Part 10, single-angle connections; Design Example II.A-28 |
| **Bolted / welded angle** | Support leg bolted, outstanding leg welded to the beam web (Bolt / Weld) | Manual Part 10, single-angle connections |

1. **Members.** Enter the **Connection mark** (new marks are numbered SA-1, SA-2 and so on), the **Supported beam**, the **Column** or **Girder**, and the **Angle**.
2. **Loads.** Choose **Service D + L** (Dead RD, Live RL) or **Required strength** (Shear Ru or Ra, and **Axial**, + for tension). The other load modes are in the editor.
3. Tick **Auto-design the connection (bolts, angle, weld) for these loads** to size the connection as it is created. The wizard keeps the beam, the support, the joint type and the configuration.
4. Select **Create**.

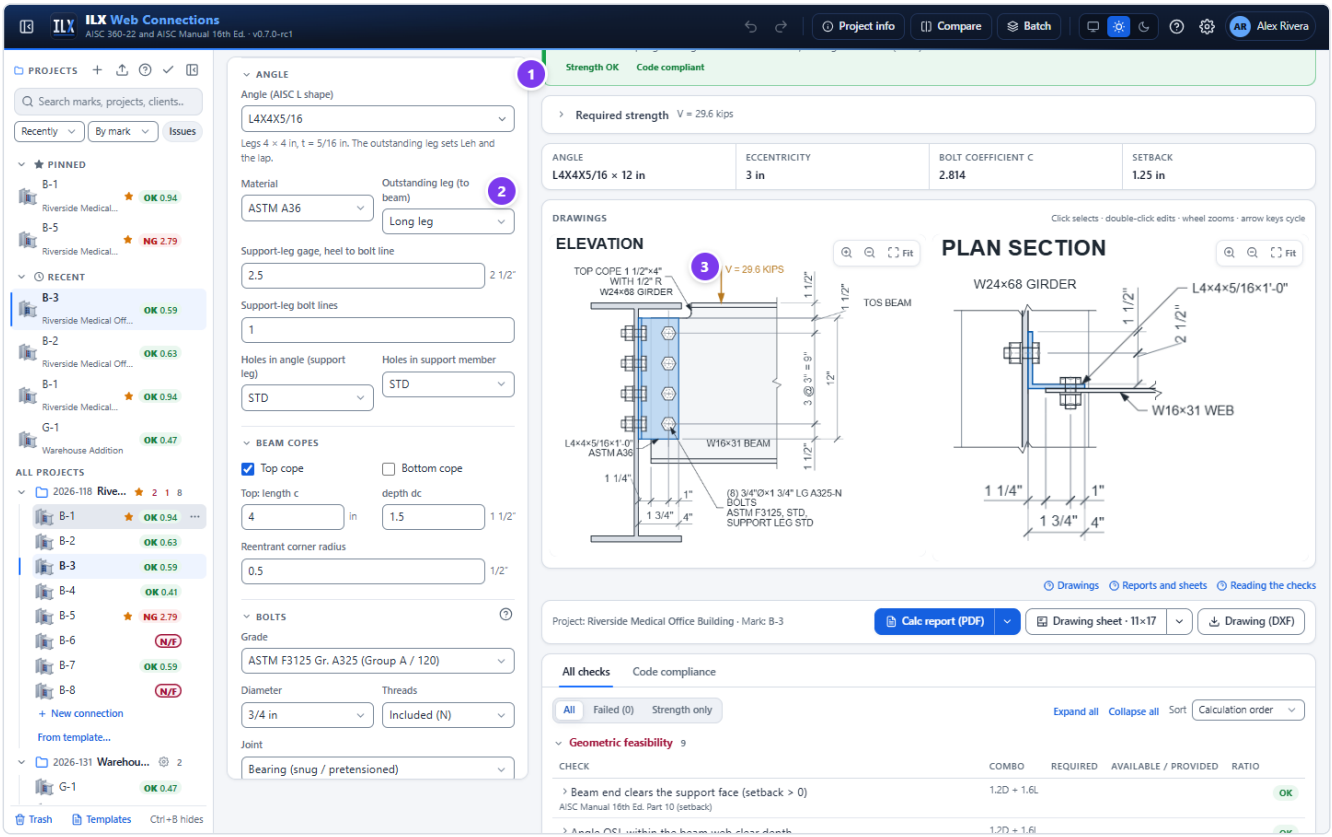
Saved single-plate connections that used an angle (Bolt / Bolt or Bolt / Weld) in earlier versions open as single-angle connections. Their inputs and results are unchanged.

## Inputs in the editor




The single-angle editor is the single-plate editor on the angle path. The sections, in order, are **Configuration**, **Loads**, **Members**, **Angle**, **Beam copes**, **Bolts**, **Angle geometry**, **Welds**, **Service conditions** and **Additional checks**. This page covers what differs from the single plate; the shared sections link to the single-plate page.

## Angle configuration




1. The **Angle** section holds the angle fields (see below).
2. **Outstanding leg (to beam)**: **Long leg** or **Short leg**.
3. The drawing shows the angle's support leg edge-on. A top cope is drawn with its radiused reentrant corner.

The **Configuration** section has two buttons, **Bolt / Bolt** and **Bolt / Weld**. The first word is the support side, the second the beam side; both read *single angle*.

The section also shows **Eccentricity carried by**. For an angle it does not change the design: ILX always takes the eccentricity on the beam side (see Eccentricity).

## Members and support

The **Members** section is the single plate's (see Members and supports), with two differences:

- **Support** also offers **HSS column wall** when the support leg is bolted; **Bolts into the HSS** and, for blind bolts, the manufacturer's strength then appear (see Bolted to an HSS wall). With the angle welded to the support, HSS walls are not offered.
- **Connections on both sides of the web** is not shown.

**Girder TOS − beam TOS** works as for the single plate: ILX checks that the beam's top flange clears the girder flange and proposes a cope if it does not.

**Beam end braced (Spec. Appendix 6.3)** with **Cb, plate flexure with the beam end not braced** (Configuration section) and **Connection near the column end** (column web or column flange) work as for the single plate: see Connection type and Members and supports.

### The angle

| Field | Meaning |
|---|---|
| **Angle (AISC L shape)** | Any AISC L shape. The line under it gives the legs and the thickness, and reminds you that the outstanding leg sets Leh and the lap. |
| **Material** | The angle material (A36, A572 Gr. 50 and so on) |
| **Outstanding leg (to beam)** | **Long leg** or **Short leg**. For an unequal-leg angle, the drawings call it LLO or SLO. |
| **Support-leg gage, heel to bolt line** | g, from the heel of the angle to the first support-leg bolt line |
| **Support-leg bolt lines** | 1 or 2 vertical bolt lines in the support leg |
| **Line spacing** | The spacing of the two support-leg lines (2 lines only) |
| **Holes in angle (support leg)**, **Holes in support member** | **STD**, **OVS**, **SSL** or **LSL**, horizontal or vertical. Manual p. 10-59 requires standard holes in the angle leg to the support ("to facilitate erection and provide torsional resistance"): any other hole type there fails the detailing check **Single angle: standard holes in the leg to the support**. For the leg to the beam the Manual says the holes "can be standard holes or horizontal short slots": other holes there (**Angle holes (beam leg)**) get the recommendation **Single angle: standard or horizontal short-slotted holes in the leg to the beam**. |

The beam-leg holes (**Angle holes (beam leg)**, **Beam web holes**) are in the **Bolts** section.

### Beam copes

As for the single plate; see Beam copes.

### Bolts and holes for the angle

The **Bolts** section is the single plate's (see Bolts and holes): grade, diameter, threads, bearing or slip-critical, **Rows, n** and **Spacing, s**, the bolt group method and the hole-deformation option. The same rows and spacing apply to both legs.

- **Bolt / Bolt: Beam-side columns** (1 to 3) and **Gage**, **Angle holes (beam leg)** and **Beam web holes**, and **Filler between plate and beam web**.
- **Bolt / Weld:** only the support-leg bolts. The beam-side fields are hidden.

### Angle geometry

| Field | Meaning |
|---|---|
| **Lev, angle (top & bottom)** | Vertical edge distance at the top and bottom bolts. The angle length follows from n, s and Lev. |
| **Top of beam to top of angle** | Where the angle sits on the web. It must clear the flange, fillet and any cope. |
| **Heel to first beam bolt column, a** | Bolt / Bolt: the distance from the heel to the first beam-side bolt column |
| **Leh, beam web** | Bolt / Bolt: horizontal edge distance on the beam web. The angle's beam-side Leh follows from the leg width and a. |
| **Angle heel to beam end (setback)** | Bolt / Weld: the gap between the support face and the beam end |

| Field | Meaning |
|---|---|
| **K (plate in compression)** | Shown when the net axial load (PD + PL + P) is compressive |

There is no thickness field: the thickness is the angle's.

### Welds (Bolt / Weld)

| Field | Meaning |
|---|---|
| **Fillet, angle to beam web** | The fillet weld from the outstanding leg to the beam web |
| **Weld shape** | **C-shape**, **Vertical** or **Top & bottom**. The eccentric weld group uses the instantaneous-center method of Manual Part 8. |
| **Lap weld** | **Plate edges** or **+ Beam end**. **Compare** designs the other option and shows both governing ratios. |
| **Electrode** | E70XX or E80XX |

There is no weld to the support: the support leg is always bolted.

### Loads, service conditions and additional checks

The **Loads** section is the shared one; see Loads and load combinations. All four input modes are supported, with axial tension or compression. Axial tension puts the support-leg bolts in tension, and ILX checks them for combined tension and shear with prying.

**Service conditions** and **Additional checks** (structural integrity, end rotation) are as for the single plate; see Service conditions and additional checks. **Cyclic loading** designs the angle for fatigue (Fatigue (Appendix 3)): the angle legs and the beam web at the bolt holes, the angle-to-web welds when welded, the angle-to-support welds when welded, and a cope corner.

## Checks for single angles

The checks are the single-plate checks with the angle legs as the connected elements. Read them as described in Checks and compliance.

### Eccentricity

- **Beam side (Bolt / Bolt).** The beam-side bolt group carries the full eccentricity, with the load at the support line (the heel). **Eccentricity carried by** is ignored for angles.
- **Support leg.** The reaction acts in the plane of the beam web, so the support-leg bolt group also carries an in-plane eccentricity of g + tw/2.

Both bolt groups are solved by the instantaneous-center method (Manual Part 7) at the true load angle.

### Bolt checks

| Check | Reference |
|---|---|
| **Beam-side bolts: shear, bearing and tearout** (Bolt / Bolt) | Manual Part 7 (instantaneous center); AISC 360-22 J3.11 |

| Check | Reference |
|---|---|
| **Support-leg bolts: shear, bearing and tearout**, with the support ply (column flange, column web or girder web) | Manual Part 7; J3.11 |
| **Support-leg bolts: tension with shear and prying** (axial tension) | J3.7, Table J3.2; J3.8; Manual Part 9 (prying action) |
| Slip resistance (slip-critical joints) | J3.9 |

## Angle leg checks

| Check | Reference |
|---|---|
| **Angle OSL shear yielding**, **shear rupture** and **block shear** (beam-side bolts) | J4.2(a), J4.2(b), J4.3 |
| Angle OSL block shear under axial load (U-shape, L-shape) and its interaction with shear | J4.3; Design Example II.A-1B |
| Angle OSL tension yielding and rupture (U = 1.0) | J4.1(a), J4.1(b) |
| **Angle OSL flexural yielding + shear interaction** at the first beam bolt column (Bolt / Bolt) or at the beam end (Bolt / Weld) | Manual Part 12, Eqs. 12-2 and 12-3; Manual Part 10, Eq. 10-8 (welded beam side) |
| Angle OSL flexural rupture + shear interaction (Bolt / Bolt) | J4.5; Manual Part 9; Manual Part 12 |
| Angle OSL flexural buckling (shown as not controlling when $\lambda \leq 0.7$) | Manual Part 9 |
| **Angle OSL tp ≤ tmax** (beam-side bolts, ductility) | Manual Part 10, extended configuration (tmax) |
| **Angle support leg shear yielding**, **shear rupture** and **block shear** | J4.2(a), J4.2(b), J4.3 |
| Fy Z ≤ Fu Znet (advisory) | Manual Part 10 (rotational ductility) |

## Weld checks (Bolt / Weld)

- **Welds to beam web** by the IC method (J2.4, Table J2.5; Manual Part 8).
- Minimum and maximum fillet size, lap ≥ 5 × the thinner part and ≥ 1 in., horizontal weld length ≥ 4w, effective length and terminations (J2.2b).
- **Beam web thickness develops the welds** and **Angle OSL thickness develops the beam welds** (Manual Part 9, minimum supporting thickness).

## Beam and support checks

- Beam web shear (G2.1), or the coped section in shear yielding and rupture and block shear (J4.2, J4.3).
- Beam tension yielding and rupture under axial load (D2, D3).
- Coped beam local flexure (Dowswell 2018) and flexural rupture through the bolt holes (J4.5).
- Girder web supports: **Support web punching shear** and **Support web yield-line strength** under axial load (J10.10; Manual Part 9).

## Detailing and feasibility checks

- Bolt spacing (J3.4, J3.6) and edge distances on the angle legs and the beam web, including the support leg's vertical and toe edge distances (J3.5, Table J3.4).
- **Support-leg gage clears the angle fillet (g ≥ k + d)** and **Bolt tightening clearance to the support** (Manual Part 7, Tables 7-15 and 7-16).

- The angle sits within the beam web clear depth, the bolts lie in the angle legs and on the support, and the holes stay inside the angle. Any failure makes the connection **NOT FEASIBLE**.
- **Angle OSL depth ≥ T/2** (beam stability), at least two bolts, element thickness ≥ 1/4 in. and setback ≥ ½ in. (recommendations).

## Sloped beam

**Beam slope** (**Skew and slope** section, default 0) works as for the single plate: the strength checks are those of the square connection (Manual p. 10-90). ILX adds a check that the angle fits between the sloped flanges, and a note. See Skewed plate and sloped beam. A skewed single angle is not a Manual variant; use the skewed bent-plate type.

## Bolted to an HSS wall

With **Support** = **HSS column wall** and the support leg bolted (**All-bolted angle** or **Bolted / welded angle**), the support-leg bolts go into the wall of a rectangular HSS. This is the less common case: with HSS columns the connecting element is typically shop-welded to the HSS and bolted to the beam (Manual 16th Ed. p. 10-96), so consider a single plate first. An angle welded to an HSS wall is not available.

**Inputs** (Members section, shown only for a bolted support leg on an HSS wall)

| Field | Default | Notes |
|---|---|---|
| **Bolts into the HSS** | **Through-bolts** | **Through-bolts (snug-tight, J3.11b)**: long bolts through the whole tube, both walls (Manual p. 7-12). **Blind bolts (manufacturer, J3.12)**: special fasteners installed from one side (Manual p. 7-13). |
| **Blind bolt available strength per fastener (manufacturer)** | 0 | Blind bolts only: the manufacturer's available shear strength per fastener in the design method you use (φRn for LRFD, Rn/Ω for ASD), in kips or kN. 0 fails the design until you enter it. |

### Checks

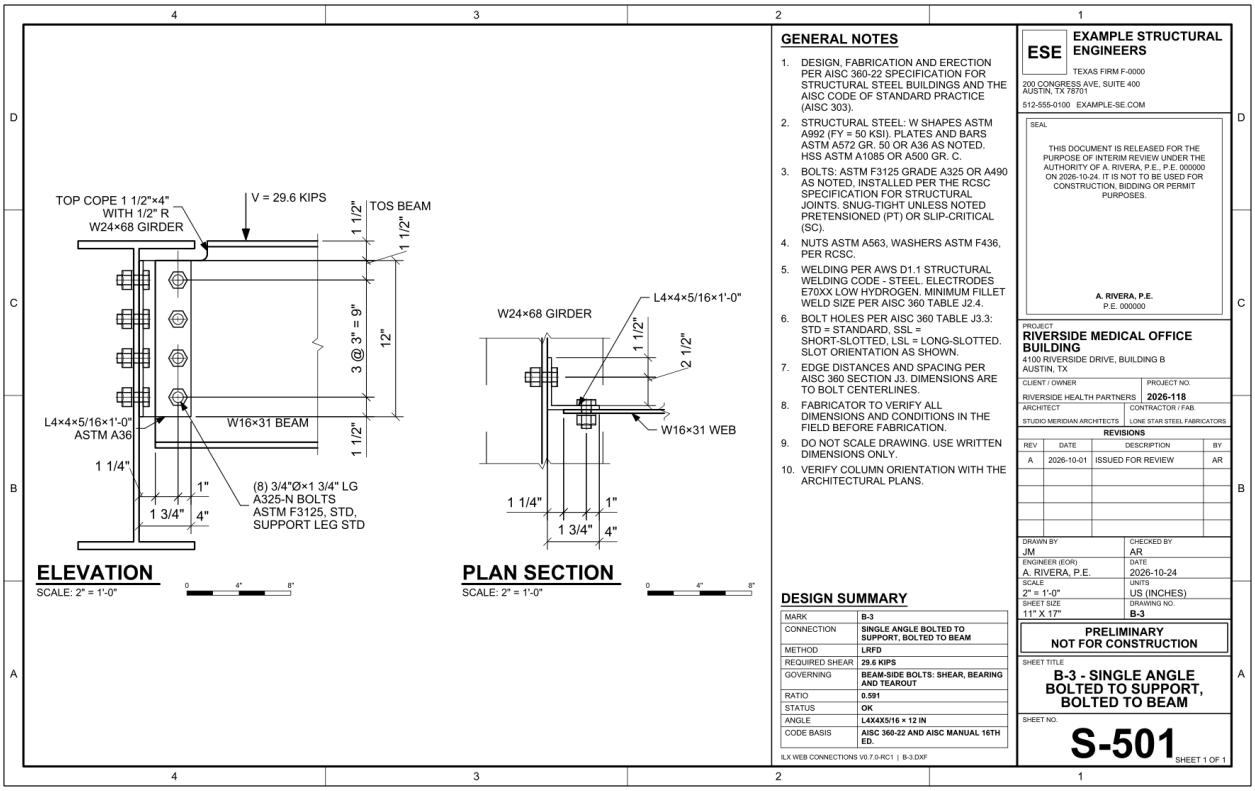
- Through-bolts: each support-leg bolt takes the least of the bolt shear (Table J3.2, single shear at the angle/HSS interface), the angle bearing and tearout (J3.11a), and the HSS wall bearing and tearout of Spec J3.11b: bearing Rn = 1.8 Fy d t (J7, Eq. J7-1, φ = 0.75, Ω = 2.00) and tearout 1.2 lc t Fu (Eq. J3-6g; 1.5 lc t Fu, Eq. J3-6h, when deformation is not a design consideration; 1.0 lc t Fu, Eq. J3-6i, for long slots perpendicular to the force). t is the HSS design wall thickness, Fy and Fu those of the HSS material; the wall tears out only between holes (the column is continuous). The group then takes the eccentricity g + tw/2 as before.
- Through-bolts are snug-tight only: the flexibility of the HSS walls precludes pretensioning (Manual p. 7-12). A **Pretensioned** or **Slip-critical** joint fails.
- The bolt grip is the angle leg plus the whole HSS depth across the connected face. Long bolts may need ASTM A449 (Manual p. 7-12).
- Blind bolts: each bolt takes the least of your manufacturer value, the HSS wall bearing and tearout (J3.11a with the design wall thickness) and the angle bearing and tearout, with the bolt value labelled as manufacturer's data (J3.12).
- Both kinds: every hole must lie on the flat of the connected HSS face, or the hole check fails (NG). A bolted leg whose toe runs past the flat gets a recommendation (it would sit on the corner radius). Axial tension through the bolts into the HSS wall fails: Spec J3.13 needs a rational analysis of the wall, which ILX does not model.

**Gaps.** The HSS-wall limit states of a blind bolt (tear-out of the bolt in tension, wall distortion) are the manufacturer's: confirm them with the product literature. Through-bolts pass through both walls: coordinate them with any connection on the opposite face. Round HSS is not available (a flat leg cannot bolt to a curved wall).

The drawing shows through-bolts crossing the tube with the nut on the far wall; the bolt callout gives the support-leg bolt length and adds *THROUGH-BOLTS, SNUG-TIGHT (AISC 360 J3.11b)* or *BLIND BOLTS PER MANUFACTURER (AISC 360 J3.12)*.

## Single-angle drawings, report and sheet

The drawings are the single plate's: an elevation, with the support leg edge-on, and a plan section. The angle is called out with its size and length, plus the leg orientation (LLO or SLO) for an unequal-leg angle. The report is titled **Single-Angle Shear Connection**, and the sheet's design summary labels the element **ANGLE**.




See Drawings and Reports.

## Auto-design for single angles

Auto-design uses the single-plate search on the angle path. It searches:

- the bolt count, diameter and grade;
- the angle (AISC L shapes with 3 to 6 in. legs, lightest first) and, for unequal legs, the outstanding leg;
- the beam weld (Bolt / Weld);
- the distance a and the support-leg gage, fitted to the leg widths.

Holes, spacing, members and loads are never changed. The support and beam sides stay as chosen, so an angle never becomes a plate. The dialog, objectives and pins work as for the single plate; see Auto-design, compare and batch. Batch design (default columns `beam`, `support`, `RD`, `RL`) and compare also work.

## Limitations and documented deviations

- **HSS supports: bolted support leg only.** On a rectangular HSS wall the support leg is through-bolted or blind-bolted, without axial tension; see Bolted to an HSS wall. A welded single plate is the usual choice.
- **The support leg is always bolted.** An angle welded to the support and bolted to the beam is not modelled. The **Bolted / welded angle** card is the reverse: bolted to the support, welded to the beam web.
- **Verification.**
  - Design Example II.A-28A (all-bolted angle to a girder web) is reproduced, OK in LRFD and ASD.
  - In Design Example II.A-29, the angle is welded to the column flange and bolted to the beam, so only its angle element checks (shear yielding, shear rupture, block shear) are compared.

  See Verification.
- **Eccentricity (II.A-28A).** ILX applies e = g + tw/2 on the support leg and e = a on the beam leg. The example neglects the beam-leg eccentricity because it is less than 3 in. (Manual Fig. 10-15). ILX's support-leg value is 10 % higher and its beam-leg value 20 % lower, and the governing strength agrees within 0.3 %. This is an open decision.
- **Beam-web edge bolt (II.A-28A).** The example conservatively uses le = 1¼ in. instead of the drawn 1½ in., so ILX's per-bolt value is higher.
- **Prying of angle support legs in tension** is computed by Manual Part 9, but no published example has been found to verify it.
- **Bolt / Weld** has no published example reproduced end to end.

CHAPTER 6

# Designing a double-angle connection

A double-angle connection is a simple shear connection made of two angles, one on each side of the beam web. Each side can be bolted or welded, so it covers Manual Tables 10-1 (all-bolted), 10-2 (bolted / welded) and 10-3 (all-welded).

## When to use it

- Use a double angle for a symmetric, flexible shear connection with good capacity, a beam that frames between column flanges or into a girder web, or a reaction with axial tension or compression.
- Use a single plate when the support side should be shop-welded and only one element is wanted.
- Use a single angle for light reactions where a one-sided connection is acceptable.
- Use a shear end plate when the element should be shop-welded to the beam end.

The two legs have their own names in the app. The **beam legs** lie against the beam web. The **outstanding legs** lie against the support. **Welds A** join the beam legs to the beam web. **Welds B** join the outstanding legs to the support.

## Creating one in the wizard

1. In the wizard's **Connection type** step, select the **Angle** card in the **Simple shear** group, then **Double angle** under **Angle type**.
2. Under **Joint type**, select **Beam to column flange**, **Beam to column web**, **Beam to girder web** or **Beam to HSS column**. On an HSS column the outstanding legs can be welded or bolted to the HSS (see Bolted to an HSS wall).
3. Under **Configuration**, select a card. The first word is the beam legs, the second the outstanding legs:

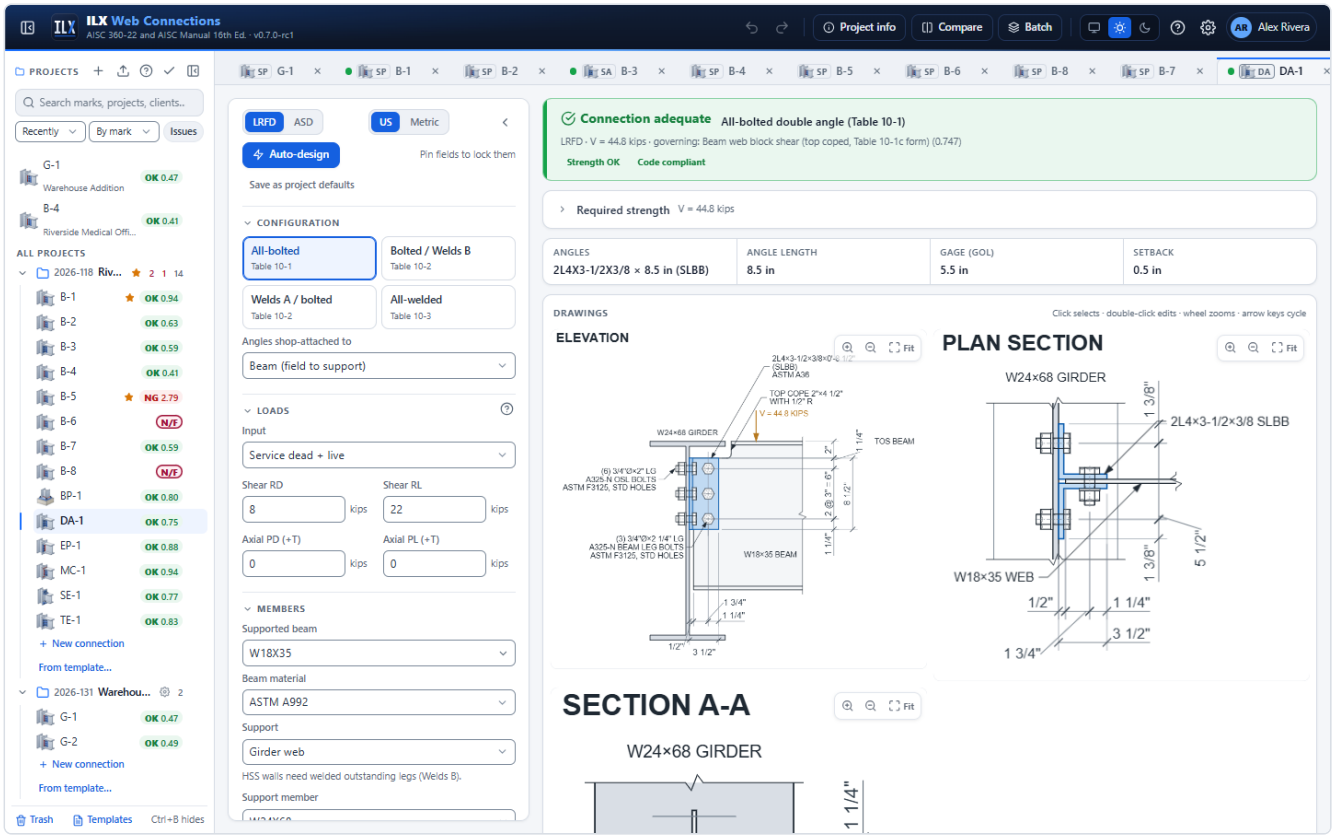
| Card | Sets | Manual basis |
|---|---|---|
| **All-bolted** | Beam legs bolted, outstanding legs bolted | Table 10-1 |
| **Bolted / Welds B** | Beam legs bolted, outstanding legs welded to the support | Table 10-2 |
| **Welds A / bolted** | Beam legs welded to the beam web, outstanding legs bolted | Table 10-2 |
| **All-welded** | Welds A and Welds B | Table 10-3 |

On an HSS column all four cards are available. **All-bolted** and **Welds A / bolted** bolt the outstanding legs to the HSS wall with through-bolts by default.

1. **Members.** Enter the **Connection mark** (new marks are numbered DA-1, DA-2 and so on), the **Supported beam**, the **Column**, **Girder** or **HSS column**, and **Angles (each)**, the angle shape used for both angles.
2. **Loads.** Choose **Service D + L** (Dead RD, Live RL) or **Required strength** (Shear Ru or Ra, and **Axial** with + for tension). The other load modes are in the editor.

3. Tick **Auto-design the connection (bolts, angles, welds) for these loads** to size the connection as it is created. The wizard keeps the beam, the support, the joint type and the configuration, and uses the **Lightest (steel weight)** objective.
4. Select **Create**.

## Inputs in the editor




The input panel has these sections, in this order. Sections that do not apply to the configuration are hidden: **Bolts** for the all-welded configuration, and **Welds** for the all-bolted one. The key-geometry strip under the status shows **Angles** (for example 2L4X3-1/2X3/8 × 8-1/2 in (SLBB)), **Angle length**, **Gage (GOL)** or **Beam leg**, and **Setback**.

### Configuration

| Field | Options and notes |
|---|---|
| Configuration buttons | **All-bolted** (Table 10-1), **Bolted / Welds B** (Table 10-2), **Welds A / bolted** (Table 10-2), **All-welded** (Table 10-3). On an HSS wall the bolted-support buttons bolt the outstanding legs to the HSS; see Bolted to an HSS wall. Choosing welded outstanding legs resets **Connection on the other side of the web** to **None**. |
| **Angles shop-attached to** | **Beam (field to support)** (default) or **Support (beam knifed in)**. With **Support**, ILX checks that the bottom flange can pass the beam legs during erection, and proposes a bottom cope if it cannot. The choice also sets which welds the drawing marks as field welds. |

## Loads and axial force

The **Loads** section is the shared one; see Loads and load combinations. All four input modes are supported: **Service dead + live**, **Service components → ASCE 7 combinations**, **Required strength (Ru, Pu)** and **Factored load cases (table)**. Axial tension and compression are both supported. The demand on the beam-leg bolts and on Welds A is the resultant of the shear and the axial force.

## Members and support

| Field | Options and notes |
|---|---|
| **Supported beam**, **Beam material** | A W, M, S or HP shape |
| **Support** | **Column flange**, **Column web**, **Girder web**, and **HSS column wall (rectangular)**. On an HSS wall the outstanding legs are welded (Welds B) or bolted with through-bolts or blind bolts. |
| **Support member**, **Support material** | A W, M, S or HP shape, or a rectangular HSS for an HSS wall. For an HSS wall, the materials are limited to A500 and A1085. Round HSS is rejected because it has no flat face. |
| **Connected HSS face** | HSS wall only: **B (wide face)** or **H (narrow face)** |
| **Bolts into the HSS** | HSS wall with bolted outstanding legs only: **Through-bolts (snug-tight, J3.11b)** (default) or **Blind bolts (manufacturer, J3.12)**. See Bolted to an HSS wall. |
| **Blind bolt available strength per fastener (manufacturer)** | Blind bolts only. Default 0, which fails until you enter the manufacturer's value. |
| **Girder TOS − beam TOS (+ = beam lower)** | Girder web only. ILX checks the beam flanges against the girder flanges and fillets, and the angles against the girder fillets. |
| **Connection on the other side of the web** | Column web or girder web with bolted outstanding legs: **None**, **Bolts shared (II.A-8)** or **Bolts staggered** |
| **Opposite reaction Ru** (or Ra), **Opposite rows sharing bolts (0 = n)** | Shared bolts only: the reaction of the opposite connection, and how many of its rows share these bolts (0 means n) |
| **Vertical stagger of the opposite bolts** | Staggered bolts only. The opposite connection's holes also pass through these outstanding legs and reduce their net section. |
| **Required stagger for wrench clearance (0 = ILX approx.)** | Staggered bolts only. Your value for the offset the wrench needs to clear the neighbouring head or nut. The Manual gives none for bolts on both sides of a web (its Table 7-15 / 7-16 staggered tables cover a bolt in each leg of an angle). 0 (default) uses the ILX approximation C1 + heavy-hex head across corners/2 + 1/16 in. The report states which was used. |
| **Connection near the column end**, **Column end to the connection force** | Column web or column flange. Cleared (default): the connection is not near the column end. Ticked: enter the distance from the column end to the force. Column web: when it is less than $\sqrt{(8wab/(a + b))}$, ILX halves the Eq. 9-45 yield-line strength (Manual p. 9-18). Column flange with an axial force: web local yielding uses Eq. J10-3 when the distance is ≤ d, and web crippling Eqs. J10-5a/b when it is < d/2 (AISC 360-22 J10.2, J10.3). The report shows the limit, the distance and which equation was used. |

When you change **Support**, the editor does not change the support member for you. Pick a matching **Support member** (an HSS for an HSS wall, a W shape otherwise). Until you do, the field shows an error.

## Angles

| Field | Options and notes |
|---|---|
| **Angles (2L)** | An AISC L shape. The line under it gives the legs, the thickness, and which legs go on the beam web. |
| **Angle material** | A36 (default), A572 Gr. 50 and so on |
| **Orientation** | **SLBB (short legs on the beam)** or **LLBB (long legs on the beam)** |
| **Top of beam to top of angles** | Places the angles on the web. They must stay within the web clear depth, below any cope. |
| **Lev, angles (top & bottom)** | Configurations with bolts. The angle length is L = (n − 1) s + 2 Lev. |
| **Angle length, L** | All-welded only |

## Beam copes

Tick **Top cope**, **Bottom cope** or both, then enter **Top cope length c** and **Top cope depth dc** (and the same for the bottom cope), and the **Reentrant corner radius**. The radius rules are those of the single plate (M2.2, at least 3/8 in., within the cope, 1 in. with cyclic loading); see Beam copes.

## Bolts and holes

| Field | Options and notes |
|---|---|
| **Bolt grade**, **Bolt diameter**, **Threads** | As for the single plate; see Bolts and holes |
| **Joint** | **Snug-tight**, **Pretensioned** or **Slip-critical** |
| **Surface**, **Number of fillers** | Slip-critical only: **Class A (μ = 0.30)** or **Class B (μ = 0.50)**; 0 to 4 fillers for hf |
| **Rows, n**, **Spacing, s** | 1 to 20 rows, the same rows in both legs |
| **Heel to beam-leg bolts**, **Beam-leg bolt lines** | Bolted beam legs: the gage from the heel, and 1 or 2 bolt lines |
| **Beam-leg line spacing** | Two beam-leg bolt lines only |
| **Holes in angle beam legs**, **Holes in beam web** | **STD**, **OVS**, **SSL** or **LSL**, horizontal or vertical |
| **Filler between each angle and the beam web** | Applies the AISC 360 J5.2 bolt-shear reduction when thicker than 1/4 in. |
| **Outstanding-leg gage (cross-centers, GOL)**, **Bolt lines per outstanding leg** | Bolted outstanding legs: the gage between the bolt lines of the two angles, and 1 or 2 lines in each leg |
| **Outstanding-leg line spacing** | Two lines per outstanding leg only |
| **Holes in outstanding legs**, **Holes in support** | As above |

| Field | Options and notes |
|---|---|
| **Include the eccentricity on the beam-leg bolts (IC method)** | Bolted beam legs. Manual p. 10-10 lets you neglect the beam-side eccentricity only for a single vertical row of bolts in standard or short-slotted holes with a (heel to the bolt line) ≤ 3 in. In that case the box is your choice: cleared (default) neglects it, ticked multiplies the group strength by C/n from the instantaneous-center method, with the load at the support face. With a > 3 in., two bolt lines, or other holes, the box is replaced by a note and the eccentricity is always included. The bolt check states which case applies. |
| **Hole deformation at service is a design consideration (2.4 / 1.2)** | Ticked: bearing 2.4 d t Fu and tearout 1.2 lc t Fu. Cleared: 3.0 and 1.5 (J3.11). |

## Geometry

| Field | Options and notes |
|---|---|
| **Beam setback** | Support face to the beam end. The default is 1/2 in. |
| **Beam underrun (strength checks)** | 1/4 in. by default (AISC practice). It is deducted from the beam-end distances, the weld returns and the shear-lag lengths in the strength checks, as in the Design Examples. |
| **K (angles in compression)** | Shown when the dead, live or required axial load is compressive. It sets Lc = K × (connection line − t). The default is 1.2. |

## Welds A and Welds B

| Field | Options and notes |
|---|---|
| **Welds A (to beam web)**, **Welds A shape** | Welded beam legs: the fillet size, and **C-shaped (toe + returns)** or **Toe only** |
| **Welds B (to support, at the toes)**, **Welds B top return** | Welded outstanding legs: the fillet along the toe of each outstanding leg, and its return at the top (about 2w; it is not counted in the strength) |
| **Electrode** | E70XX or E80XX |

Weld the outstanding legs along the toes only. A weld across the top or bottom of the angles, or along the heel, stops the legs from flexing and the connection from rotating. ILX adds this as a note.

## Service conditions and additional checks

**Hot-dip galvanized**, **Unpainted weathering steel**, **Cyclic loading (fatigue, AISC 360 Appendix 3)**, **Structural integrity tension (AISC 360 B3.9)** and **End rotation / flange binding** (with **Required rotation** and the presets **0.03** and **0.025 (341)**) work as for the single plate; see Service conditions and additional checks. Weathering steel sets the maximum spacing to 14t ≤ 7 in.

**Fatigue (Appendix 3).** With **Cyclic loading** ticked the connection is designed for fatigue, with the cycle count and the cyclic load range as for the single plate; see Fatigue (Appendix 3). The details: the angles and the beam web at the bolt holes (Table A-3.1 Case 2.1 slip-critical or 2.2 pretensioned, Category B; snug-tight joints Case 1.5, D), Welds A (an elastic weld group per angle with V/2 at the heel line, Case 8.2, F), Welds B (each angle a line of length L with V/2 at the outstanding-leg width, $q = V/(2L)\sqrt{(1 + 36\, e^2/L^2)}$, Case 8.2, F) and the beam at a cope corner (Case 1.3).

## The checks it runs

See Checks and compliance for how to read the results. Each check cites the reference listed here.

### Bolts

- Beam-leg bolts: shear transfer at bolt holes, Σ min(bolt shear, bearing and tearout on both angles, bearing and tearout on the web), in double shear (J3.7, Table J3.2; J3.11; Manual Table 10-1, 10-2 or 10-3). Tearout uses lc along the line of action of the resultant. The beam-end edge includes the underrun and the copes over the bolt line.
- Outstanding-leg bolts: shear transfer at bolt holes, single shear, both angles, for the shear only (same references).
- Slip resistance: 2 slip planes on the beam legs, 1 on the outstanding legs, with ksc under tension (J3.9).
- Outstanding-leg bolts under axial tension: tension with shear and prying (J3.7; J3.8; Manual Part 9 prying action). On a column flange, the flange is also checked for prying at the bolts (Manual Part 9; AISC 360-22 J10.1).
- Long joints, A307 grips and fillers reduce the bolt shear (Table J3.2 notes, J5.2).
- Shared bolts: double shear, and support-web bearing and tearout under both reactions (Manual Part 10; Design Examples II.A-8 and II.A-9).

### Welds

- Welds A: instantaneous-center method on the C-shaped (or toe-only) group of one angle, at the true load angle, times two angles (J2.4, Table J2.5; Manual Part 8; Table 10-2 basis, Table 8-8).
- Welds B: both toe welds lumped, Table 8-4 at 0° with k = 0 and ex = the outstanding-leg width (J2.4; Manual Part 8; Table 10-2 basis). With axial load, an elliptical shear and axial interaction.
- Base metal: the beam web develops Welds A on both faces, 6.19 D/Fu; the support develops Welds B, 3.09 D/Fu, or 6.19 D/Fu with a connection on both faces (Manual Part 9). On an HSS, Fu is that of the HSS material.
- Support shear rupture along Welds B on a column or girder web (J4.2(b)).
- Minimum fillet size (J2.2b, Table J2.4), length ≥ 4w (J2.2b(c),(d)), the Welds B top return, and weld terminations (J2.2b(e)).

### Angles

- Shear yielding of both angles (J4.2(a)); shear rupture of each bolted leg, or of the gross section when both legs are welded (J4.2(b)).
- Block shear of each bolted leg (J4.3). Under tension, the U-shape and L-shape axial blocks of the beam legs and their interaction with the shear block (J4.3; Design Example II.A-1B).
- Tension yielding and rupture (J4.1); compression of the legs between the support and the beam connection (J4.4).
- Thickness limits: t ≤ 5/8 in. for flexibility, t ≥ 1/4 in. (recommendations), and t ≥ w + 1/16 in. along an edge welded with a fillet of 1/4 in. or more (J2.2b(b)).
- Length ≥ T/2 for beam stability (Manual Part 10, recommendation).

### Beam

- Web shear (G2.1), or shear yielding of the coped section (J4.2(a)).
- Web shear rupture only when the beam is double-coped. Otherwise a flange lies in the rupture path, and ILX says so.
- Web block shear: top-coped bolted webs in the Table 10-1c form; around Welds A when top-coped (II.A-5) or under tension (II.A-2B) (J4.3).

- Coped-beam flexural local buckling: single copes by Dowswell (2018), double copes by Dowswell and Whyte (2014), with e = c + setback (Manual Part 9); rupture through the bolt holes inside the cope (J4.5).
- Tension yielding and rupture with shear lag (D2 and D3, Table D3.1).

**Support under axial load**

- Column or girder web: punching shear (J10.10; Manual Part 9).
- Column web: yield line by Manual Eq. 9-45. The column-web yield line (Manual Eq. 9-45) is halved when the force is closer to the column end than √(8wab/(a + b)) (your input, Manual p. 9-18). When the load is spread over more than 85 % of the web width w, the check is reported as noncritical (Manual p. 9-17). The 85 % rule is not applied to girder webs. Girder web: approximate yield line, flagged in the message.
- Column flange: web local yielding (J10.2); web crippling in compression (J10.3).
- HSS wall: plastification by an approximate yield line (J10.10).

**Detailing and feasibility**

- Spacing (J3.4) and maximum spacing and edge distance (J3.6); minimum edge distances (J3.5, Table J3.4); slip-critical requirements for oversized holes and parallel slots, and long slots in one ply only (J3.3); pretension (J3.2); washers (RCSC Section 6); bolt grip (RCSC 2.3); entering, tightening and fillet clearances (Manual Tables 7-15 and 7-16).
- Setback ≥ 1/2 in., cope limits, framing between the column flanges, girder flange and fillet clearance, knifed erection and the OSHA 1926.756(c) note for double connections.
- NOT FEASIBLE when the beam end runs into the support, the beam legs do not reach past the beam end, the angles leave the web clear depth or run into a cope, holes overlap or lie outside a ply, the outstanding legs do not fit on the column flange, between the column fillets or on the HSS flat (K.3), or staggered holes overlap.

**Structural integrity (B3.9).** With the option ticked, the tie force T = max(2/3 Vu or Va, 10 kips) is checked alone, with nominal strengths: bolt shear and tension, web bearing and tearout, angle bearing, yielding and rupture, web block shear and prying (Design Example II.A-1C), or the welds in tension for welded legs.

## Sloped beam

**Beam slope** in **Geometry** sets the vertical angle of the beam flanges, + rising away from the support (default 0). The Manual designs a sloped double angle "essentially similar to ... rectangular double-angle connections" (p. 10-90, Fig. 10-41). Every strength check is therefore unchanged, using the vertical reaction.

ILX adds two items:

- **Fit check.** The angles must stay between the sloped flanges, clear of the fillets or copes, over the beam legs' length.
- **Note.** For a girder support, it gives how far the beam top moves over the girder flange. Verify the copes on the layout; the cope checks use square framing.

The elevation is drawn square. See Skewed plate and sloped beam.

## Bolted to an HSS wall

With **Support** = **HSS column wall (rectangular)** and bolted outstanding legs (**All-bolted** or **Welds A / bolted**), the outstanding-leg bolts go into the HSS wall. This is the less common case: with HSS columns the connecting element is typically shop-welded to the HSS and bolted to the beam (Manual 16th Ed. p. 10-96).

### Inputs

| Field | Default | Notes |
|---|---|---|
| **Bolts into the HSS** | **Through-bolts** | **Through-bolts (snug-tight, J3.11b)**: long bolts through the whole tube, both walls (Manual p. 7-12). **Blind bolts (manufacturer, J3.12)**: special fasteners installed from one side (Manual p. 7-13). |
| **Blind bolt available strength per fastener (manufacturer)** | 0 | Blind bolts only: the manufacturer's available shear strength per fastener in the design method you use (φRn for LRFD, Rn/Ω for ASD), in kips or kN. 0 fails the design until you enter it. |

### Checks

- Through-bolts: each bolt takes the least of the bolt shear (Table J3.2, single shear at the angle/HSS interface), the angle bearing and tearout (J3.11a), and the HSS wall bearing and tearout of Spec J3.11b: bearing Rn = 1.8 Fy d t (J7, Eq. J7-1, φ = 0.75, Ω = 2.00) and tearout 1.2 lc t Fu (Eq. J3-6g; 1.5 lc t Fu, Eq. J3-6h, when deformation is not a design consideration; 1.0 lc t Fu, Eq. J3-6i, for long slots perpendicular to the force). t is the HSS design wall thickness, Fy and Fu those of the HSS material. The HSS column is continuous, so the wall tears out only between holes.
- Through-bolts are snug-tight only: the flexibility of the HSS walls precludes pretensioning (Manual p. 7-12). A **Pretensioned** or **Slip-critical** joint fails.
- The bolt grip is the outstanding leg plus the whole HSS depth across the connected face. Long bolts may need ASTM A449 (Manual p. 7-12); the threads-excluded check uses this grip.
- Blind bolts: each bolt takes the least of your manufacturer value, the HSS wall bearing and tearout (J3.11a with the design wall thickness) and the angle bearing and tearout. The calculation labels the bolt value as manufacturer's data (J3.12).
- Both kinds: every hole must lie on the workable flat of the connected HSS face (corner radius 2.25 tnom, Manual Table 1-12), or the hole check fails (NG). A bolted leg whose toe runs past the flat gets a recommendation (it would sit on the corner radius).
- Axial tension through the bolts into the HSS wall fails: Spec J3.13 needs a rational analysis of the wall (yield line, pull-out), which ILX does not model. Compression and shear are designed.

**Gaps.** The HSS-wall limit states of a blind bolt (tear-out of the bolt in tension, wall distortion) are the manufacturer's: confirm them with the product literature. Through-bolts pass through both walls: coordinate them with any connection on the opposite face. Round HSS is not available (a flat leg cannot bolt to a curved wall).

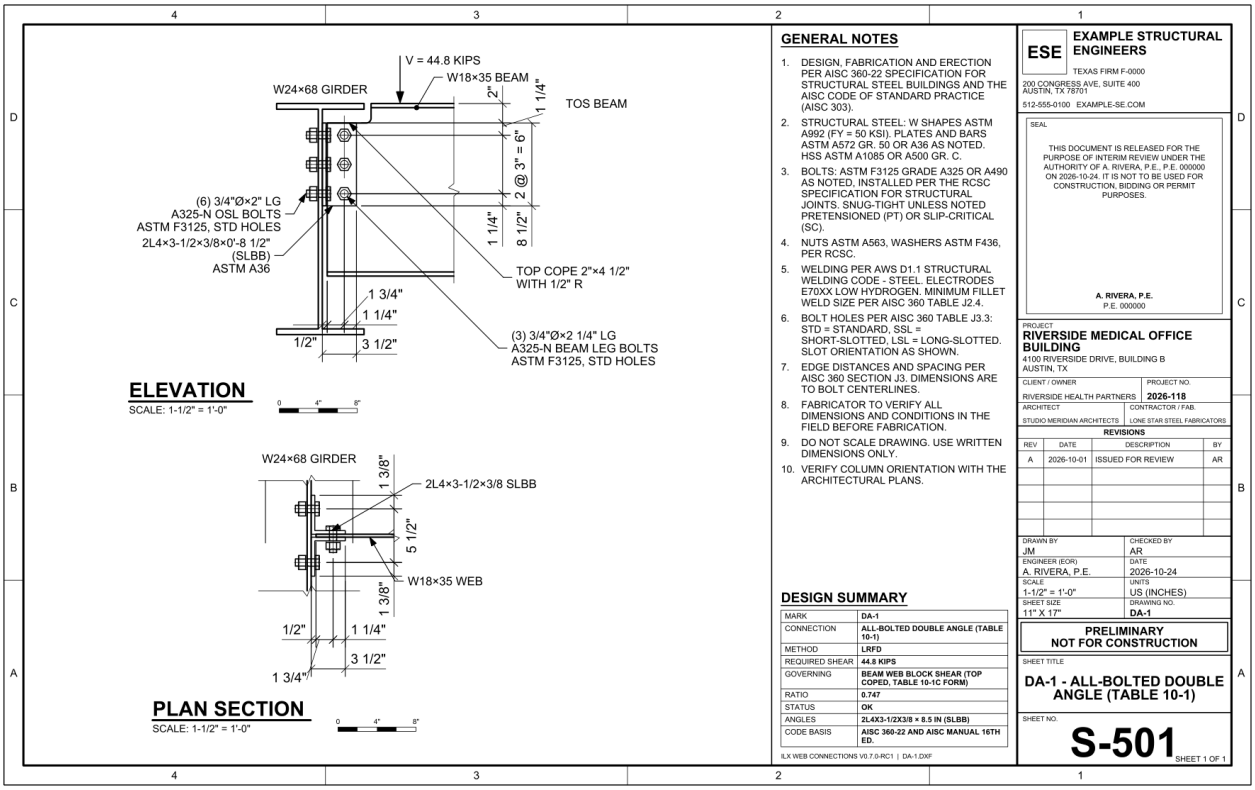
The drawing shows through-bolts crossing the tube with the nut on the far wall, and the bolt callout adds *THROUGH-BOLTS, SNUG-TIGHT (AISC 360 J3.11b)* or *BLIND BOLTS PER MANUFACTURER (AISC 360 J3.12)*.

## Drawings, report and sheet

The editor shows three views:

- **ELEVATION**: the beam web face-on with the near angle, the bolts, Welds A and B, the copes and the support profile.
- **PLAN SECTION**: both angles straddling the beam web, with the support in plan.
- **SECTION A-A**: the support face seen from the beam, with both outstanding legs, the gage GOL and Welds B.

Every bolt is drawn with its head, washers and nut. The beam-leg and outstanding-leg bolt callouts each give their own bolt length. The Welds A symbol reads **3 SIDES** (C-shaped) or **TOE**. The Welds B symbol reads **TOES** with the return.




The drawing sheet carries the elevation and the plan section with the title block. Section A-A is on screen and in the calculation report. See Drawings and Reports.

## Auto-design, compare and batch

Auto-design searches:

- the bolt diameter: 3/4, 7/8 or 1 in., when there are bolts;
- the angle: 13 standard angles from L3X3X1/4 to L6X4X5/8, lightest first;
- the number of rows: from the smallest count that reaches T/2 up to 12. For the all-welded configuration, it searches the angle length in 1-1/2 in. steps instead;
- the sizes of Welds A and Welds B: 3/16 to 3/8 in., no larger than t − 1/16 in.

For each angle, it keeps the shortest length that passes. It never changes the configuration, the members or the support. Pinned fields are kept. The objectives are **Lightest (steel weight)**, **Fewest bolts** and **Lowest cost (fabrication estimate)**. For the dialog, see Auto-design, compare and batch.

Compare works for double angles. The default batch columns are `beam`, `support`, `RD` and `RL`. When batch auto-designs, it keeps the beam, the support, the support type and the configuration.

## Limitations and documented deviations

### Scope

- The beam is a W, M, S or HP shape. The support is a W, M, S or HP shape, or a rectangular HSS (outstanding legs welded, or bolted with through-bolts or blind bolts; no axial tension when bolted).
- Each leg takes 1 or 2 bolt lines, with 1 to 20 rows.
- Back-to-back connections (shared or staggered) need a column or girder web and bolted outstanding legs.

### Simplifications

- The outstanding-leg bolts carry the shear only. Axial tension goes to bolt tension with prying, and compression to bearing of the legs on the support (Design Example II.A-2B).
- The eccentricity on the beam-leg bolts is neglected unless you tick the IC option, and only where Manual p. 10-10 permits it (one bolt line, STD or short-slotted holes, a ≤ 3 in.).
- The Welds B return is not counted in the strength. The axial interaction of Welds B is engineering practice, not a Manual equation. Out-of-plane bending of the outstanding legs about the toe welds is covered only by the flexibility note.
- The girder web yield line and the HSS plastification under axial load are approximate. Both are flagged in the check.
- The tightening clearance C1 and fillet clearance C3 come from Manual Table 7-15 (Table 7-16 for F1852 and F2280 twist-off bolts, with the tool and socket set in **Settings > Design rules**; 1/2 in., Group 200 (F3043), A307 and threaded-rod bolts use your C1 from the same tab, as the Manual does not tabulate them). The staggered-bolt clearance on both sides of a web is your input, with the ILX approximation as the default: the Manual's staggered-bolt tables cover bolts in the two legs of an angle.
- Letter-size sheets can overflow with the text-dominated views. Use 11 × 17.

**Verification.** ILX matches 124 published LRFD and ASD values of Design Examples II.A-1A, 1B, 1C, 2A, 2B, 3, 4, 5, 8 and K.3 within 0.81 %. Other rows of Tables 10-1 to 10-3 were not benchmarked. See Verification.

### Documented deviations

| Example | Published | ILX | Why |
|---|---|---|---|
| II.A-2B, Welds A | 175.5 kips (Table 8-8 entered at 30°) | 180.2 kips (+2.7 %) | ILX uses the actual load angle, 38.7°. The example rounds down to the 30° table row, which is conservative. At 30°, ILX gives C = 2.689, as the table does. |
| II.A-1B, block-shear interaction | 0.0531 (with the larger L-shape axial block) | 0.0543 | ILX combines the shear block with the smaller of the U-shape and L-shape axial blocks, which is conservative (+2.3 %). |

CHAPTER 7

# Designing a shear end-plate connection

A shear end plate is a plate shop-welded to the end of the beam web, with a fillet weld on each face of the web, and field-bolted to the support with two vertical lines of bolts. ILX designs it to AISC Manual Part 10, Shear End-Plate Connections (Table 10-4), and Design Examples II.A-11A to II.A-11C.

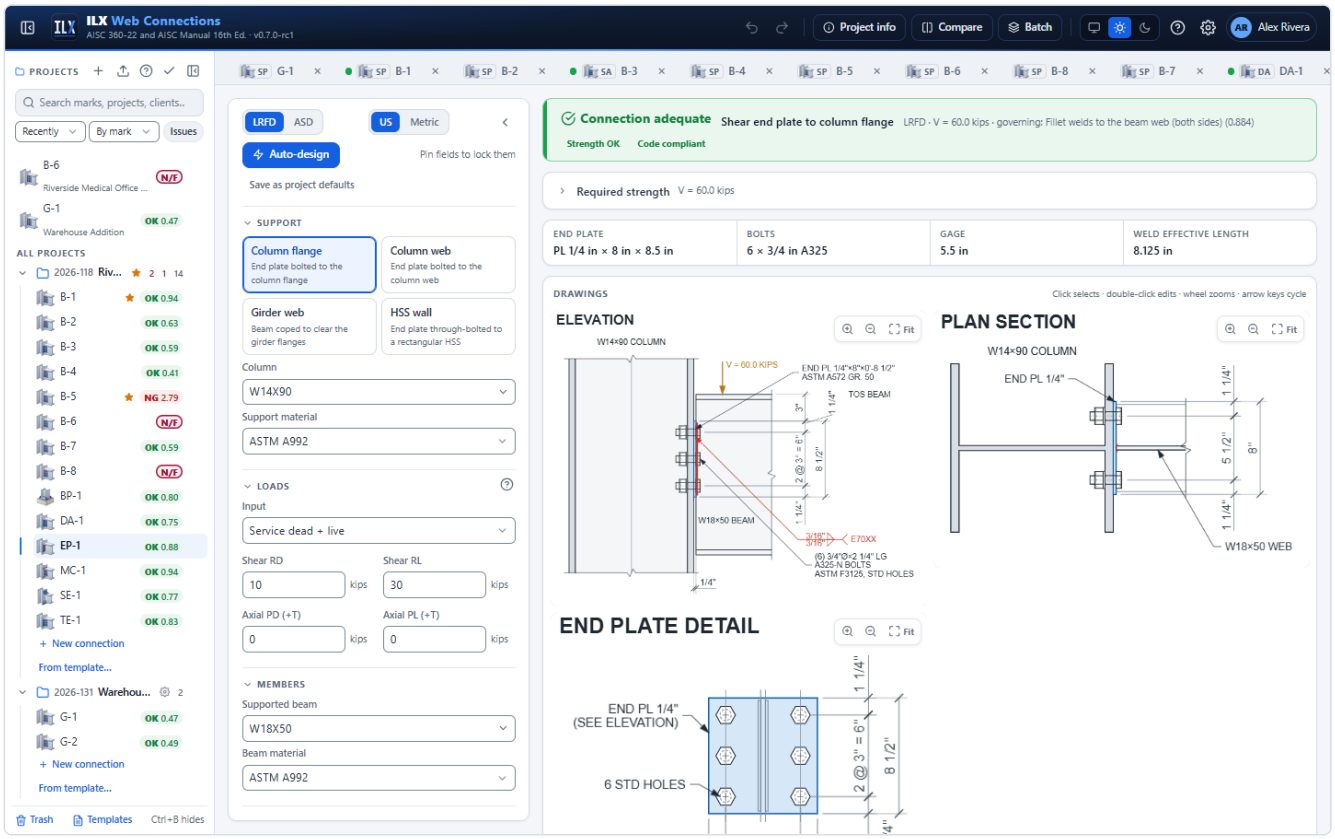
## When to use a shear end plate

- Use it when you want all the welding done in the shop and a simple bolted field connection, and the beam length can be held to the end-plate tolerance.
- The end plate must fit on the beam web flat, between the fillets or copes. A plate that projects past the beam flanges is an extended end plate, which is a moment connection, not this type.
- For a shear tab welded to the support, use the single plate. For a tab that is bolted on both sides, or when the beam length is uncertain, see double angles, the single angle and the shear tee.

## Creating a shear end plate in the wizard

1. On the project row, open the menu (**⋯**) and choose **New connection…**.
2. **Connection** step:
   - **Connection type**: **Plate** in the **Simple shear** group, then **End plate** under **Plate type** ("Plate shop-welded to the beam end, field-bolted to the support").
   - **Joint type**: **Beam to column flange**, **Beam to column web**, **Beam to girder web** or **Beam to HSS column**. All four are offered. The cards read "End plate bolted to the support", or "End plate through-bolted to the HSS" for the HSS column.
   - There are no configuration cards: the end plate is always welded to the beam and bolted to the support.
3. **Members** step: **Connection mark** (prefix **EP**, for example EP-1), **Supported beam**, and **Column**, **Girder** or **HSS column**. Choosing the HSS joint changes the support to an HSS member.
4. **Loads** step: **Service D + L** (Dead RD, Live RL) or **Required strength** (Shear Ru or Ra, and **Axial (+T)**). The other load modes are in the editor.
5. Tick **Auto-design the connection (bolt rows, end plate, weld) for these loads** to size the connection before it is created. The beam, the support and the joint type are kept; the objective is the lightest design.
6. Select **Create**.

## Editor inputs




The toolbar has **Design method** (**LRFD** or **ASD**), **Units** (**US** or **Metric**), **Auto-design** and **Save as project defaults**. The key-geometry strip under the status bar shows the **End plate** (for example PL 1/4 × 8 1/2 × 8 1/2), the **Bolts**, the **Gage** and the **Weld effective length** per side.

The sections below follow the order of the input panel.

### Support and framing

| Field | Options and notes |
|---|---|
| Support cards | **Column flange**, **Column web**, **Girder web** ("Beam coped to clear the girder flanges"), **HSS wall** ("End plate through-bolted to a rectangular HSS") |
| **Column**, **Girder** or **HSS column** | The support member. Switching to **HSS wall** sets HSS8X8X1/2 in A500 Gr. C; switching back sets W14X90 (W24X68 for a girder) in A992. |
| **Support material** | HSS walls offer the A500, A1085 and A53 Gr. B grades only |
| **Connected HSS face** | HSS wall only: **B** or **H** face. Rectangular HSS only; the bolts pass through both walls. |
| **Girder TOS − beam TOS (+ = beam lower)** | Girder web only. Sets the bolt and plate position on the girder web and the flange interference check. Without a top cope, the editor reminds you that beams framing into a girder web usually need one. |
| **Connections on both sides of the web** | Column web and girder web only |
| **Back-to-back end plates share the bolts** | Shown when both sides are ticked. The bolts pass through both end plates and the web. |

| Field | Options and notes |
|---|---|
| **Connection near the column end**, **Column end to the connection force** | Column web or column flange. Cleared (default): the connection is not near the column end. Ticked: enter the distance from the column end to the force. Column web: when it is less than √(8wab/(a + b)), ILX halves the Eq. 9-45 yield-line strength (Manual p. 9-18). Column flange with an axial force: web local yielding uses Eq. J10-3 when the distance is ≤ d, and web crippling Eqs. J10-5a/b when it is < d/2 (AISC 360-22 J10.2, J10.3). The report shows the limit, the distance and which equation was used. |
| **Opposite connection shear (0 = same)** | Shared bolts only. Sets this connection's share of the web bearing, V / (V + V other). |

## Load modes and axial force

All four load modes are available: **Service dead + live**, **Service components → ASCE 7 combinations**, **Required strength** and **Factored load cases (table)**. Axial load is supported, positive in tension. See Loads and load combinations.

- Axial tension adds bolt tension with shear, prying on the end plate (and on a column flange), beam tension and the support checks under axial load.
- Axial compression is assumed to bear through the end plate. ILX checks the support under the plate bearing area and, conservatively, assigns the resultant to the welds. There is no plate buckling check and no **K** field.

## Supported beam and material

| Field | Notes |
|---|---|
| **Supported beam** | W, M, S or HP shape |
| **Beam material** | ASTM A992, A572 Gr. 50, A36 and so on |

## Copes and reentrant corners

| Field | Notes |
|---|---|
| **Top cope**, **Bottom cope** | Tick one or both |
| **Top: length c**, **depth dc** / **Bottom: length c**, **depth dc** | The length c is measured from the beam end, that is, the end-plate face |
| **Reentrant corner radius** | Shown when a cope is ticked. Checked to AISC 360 M2.2: at least 3/8 in., and it must fit within the cope. |

A cope moves the top (or bottom) of the web flat to the cope line, so the end plate and its weld must stay below it.

## Bolts and bolt pattern

| Field | Options and notes |
|---|---|
| **Bolt grade** | The bolt grades of the app (Group A, B, C, A307 and rods) |
| **Bolt diameter** | Limited to the sizes the grade offers |
| **Threads** | **Included (N)** or **Excluded (X)** |

| Field | Options and notes |
|---|---|
| **Joint** | **Snug-tight (bearing)**, **Pretensioned (bearing)** or **Slip-critical**. Grades that cannot be pretensioned offer snug-tight only. |
| **Surface** | Slip-critical only: **Class A (μ = 0.30)** or **Class B (μ = 0.50)** |
| **Rows, n** and **Spacing, s** | Bolt rows (1 to 20) and their pitch. There are always two bolt lines, so the connection has 2n bolts. |
| **Gage, g (two bolt lines)** | Manual Table 10-4 uses 3-1/2 to 5-1/2 in.; 5-1/2 in. clears column-flange fillets |
| **End-plate holes**, **Support holes** | **STD**, **OVS**, **SSL horizontal**, **SSL vertical**, **LSL horizontal**, **LSL vertical** |
| **Bolt group strength** | **Σ least of shear / bearing per bolt (default)** or **n × weakest bolt (conservative)** |
| **Hole deformation at service is a design consideration (2.4 / 1.2)** | Ticked: bearing 2.4 d t Fu, tearout 1.2 lc t Fu. Cleared: 3.0 and 1.5. |
| **Filler between end plate and support** | Applies the AISC 360 J5.2 bolt-shear reduction when thicker than 1/4 in. It adds to the grip and to the rotation gap. |

## End-plate size and position

| Field | Notes |
|---|---|
| **Thickness** | 3/16 to 1-1/4 in. The editor shows the flexibility range, 1/4 to 3/8 in. (AISC Manual Part 10). |
| **End-plate material** | A572 Gr. 50 by default |
| **Lev (top & bottom)** | Vertical edge distance. The plate length is L = 2 Lev + (n − 1) s. |
| **Leh (plate edge to bolt line)** | Horizontal edge distance. The plate width is W = g + 2 Leh. |
| **Top of beam to top of plate** | Places the plate on the beam web. It must keep the plate on the web flat. |

## Weld to beam web

| Field | Notes |
|---|---|
| **Fillet to beam web (both sides)** | 1/8 to 5/8 in. One fillet on each face of the web. |
| **Electrode** | E70XX or E80XX |
| **Directional strength increase (AISC 360 J2.4)** | Ticked: (1 + 0.50 sin^1.5 θ) at the angle of the resultant of shear and axial force. With shear only, θ = 0 and there is no increase. |

The effective weld length per side is the part of the plate on the web flat (kdes or the cope line to d − kdes) minus 2w: the welds stop one weld size short of each end.

## Service conditions and integrity

| Option | Effect |
|---|---|
| **Hot-dip galvanized** | Adds the M2.2 note: grind copes to bright metal |
| **Unpainted weathering steel** | Maximum bolt spacing 14t ≤ 7 in. instead of 24t ≤ 12 in. |

| Option | Effect |
|---|---|
| **Cyclic loading (fatigue, AISC 360 Appendix 3)** | Designs the connection for fatigue (Fatigue (Appendix 3)), and requires the Appendix 3 cope radius, the pretension requirement for bolts in tension and the cut-surface roughness note. The details: the end plate at the bolt holes (Table A-3.1 Section 2, or Case 1.5 for snug-tight joints), the end-plate-to-web fillets (Case 8.2, F), and with axial tension the beam web at the welds (Case 5.7: the lesser of the toe crack C and the root crack C″ with RFIL) and the bolts in tension (Case 8.5, G: 20 % of the cyclic tension for pretensioned bolts, Section 3.4(b)); the beam at a cope corner (Case 1.3). |
| **Structural integrity tension (AISC 360 B3.9)** | Runs the integrity checks as an independent case (see below) |
| **Axial in the column braced by this beam (B3.9(c), 0 = none)** | Integrity only. Adds the B3.9(c) column-bracing check. |
| **End rotation / flange binding** | Checks the rotation before the bottom flange bears on the support. The gap is the end plate plus filler. Enter **Required rotation** or use the presets **0.03** or **0.025 (341)**. |

## Checks the end plate runs

Every check carries its clause, and failing checks give a fix hint. See Checks and compliance for how to read the results.

### Feasibility gate

These checks mark the connection NOT FEASIBLE:

- a round HSS support (a flat end plate cannot bear on a curved wall);
- the end plate outside the beam depth, or no web flat for the welds (effective length ≤ 0);
- holes that overlap or break out of the plate (J3.4), and a net section ≤ 0 (B4.3b);
- bolt heads or nuts on the web weld: g/2 − tw/2 − w must be at least half the heavy-hex width across corners (Manual Part 7);
- bolts off the support: past the column flange edge or in the column web fillet (k1), plate wider than the column web flat T, plate off the girder web flat, or holes off the HSS flat;
- copes that leave no web;
- on a column web, beam flanges that do not fit between the column flanges (bf + 1/2 in. ≤ d − 2tf) unless both are coped past the flange tips;
- on a girder web, a beam flange that hits the girder flange or fillet. ILX proposes the cope as a suggested fix.

### Bolts and bearing at the holes

| Check | Reference |
|---|---|
| Shear transfer at the bolt holes: each bolt takes the least of bolt shear, end-plate bearing or tearout and support bearing or tearout; Rn = Σ rn (or 2n × weakest) | J3.7, Table J3.2; J3.11 (J3.7 User Note, Design Example II.A-11A) |
| Bolt shear reductions: long joint (> 38 in.), A307 grip, filler | Table J3.2 notes b and c; J5.2 |
| Slip resistance, μ Du hf Tb with Du = 1.13 and φ or Ω by hole type, reduced by ksc under tension | J3.9 (Eqs. J3-5a, J3-5b) |

| Check | Reference |
|---|---|
| Bolt tension combined with shear, F'nt (skipped when frv ≤ 30 % of the available shear stress) | J3.8 (Eqs. J3-3a, J3-3b) |
| End-plate bending with prying action: the end plate as a tee flange with the beam web as the stem, b = (g − tw)/2, p = min(s, L/n) | Manual Part 9 (prying action), Eqs. 9-23 to 9-40; Design Example II.A-11B |

The bolts push the plate up, so the top row tears out toward the top edge of the plate. The support has no free edge below the bolts.

### End-plate strength

| Check | Reference |
|---|---|
| Shear yielding on two planes, 0.60 Fy (2 L tp) | J4.2(a) |
| Shear rupture on two planes through the holes, 2 [L − n (hv + 1/16)] tp | J4.2(b) |
| Block shear on two L-shaped blocks, Ubs = 1.0 | J4.3 |
| Thickness ≤ 5/8 in. for flexibility, as for double angles (compliance; advisory when axial tension is present) | Manual 16th Ed. Part 10, shear end plates: flexibility |
| Thickness ≥ 1/4 in., gage 3-1/2 to 5-1/2 in., rows 2 to 12 (advisories) | Manual Part 10; Table 10-4 |
| Length L ≥ T/2 for erection stability (advisory) | Manual Part 10, Shear End-Plate Connections |

### Welds and beam web

| Check | Reference |
|---|---|
| Fillet welds to the beam web: Fnw × 2 × 0.707 w lw against the resultant of shear and axial | J2.4, Table J2.5; Manual Part 10 |
| Minimum fillet size and minimum length 4w | J2.2b, Table J2.4 |
| Beam web shear yielding and shear rupture along the weld length | J4.2; Manual Part 9 |
| Beam web develops the welds, tmin = 6.19 D/Fu for E70 (advisory) | Manual Eq. 9-7 |
| Weld near the k-area (note) | J10.8 Commentary |

### Beam strength

| Check | Reference |
|---|---|
| Web shear on the gross section, or shear yielding of the coped section | G2.1; J4.2(a) |
| Coped-beam flexure, single cope (Dowswell 2018) or double cope (Dowswell & Whyte 2014), with e = end plate + filler + c | Manual Part 9 |
| Cope limits, radius, fatigue radius, galvanizing and cut-surface notes | Manual Part 9; M2.2; Appendix 3 |
| Tension yielding (reduced at copes) and rupture at the welds, An = lw tw with U = 1.0 | D2 and D3 (Design Example II.A-11B) |

### Support under axial load

| Support | Checks | Reference |
|---|---|---|
| Column flange | Flange bending with prying (tension); web local yielding; web crippling (compression) | Manual Part 9; J10.2; J10.3 |
| Column web | Web yield line (c = g, l = (n − 1) s in tension; W × L in compression), halved near the column end (your input) and noncritical when c > 0.85 w; punching shear (tension) | J10.10; Manual Eq. 9-45, pp. 9-17 and 9-18 |
| Girder web | Approximate yield-line mechanism, flagged; punching shear (tension) | J10.10; Manual Part 9 |
| HSS wall | Approximate wall plastification, flagged; b/t ≤ 1.40 √(E/Fy) advisory | J10.10 / Chapter K commentary |

Support bearing and tearout are part of the bolt checks for every support.

### Detailing

- Spacing ≥ 2⅔d (3d preferred), gage ≥ 2⅔d (J3.4); maximum spacing (J3.6).
- Edge distances Lev and Leh on the plate, and (bf − g)/2 on a column flange: Table J3.4 plus the Table J3.5 increments (J3.5). Maximum edge distance 12t ≤ 6 in. (J3.6).
- Oversized holes and vertical slots require slip-critical bolts; long slots in one ply only (J3.3).
- Pretensionable grade (J3.2); pretension required for Group B or C bolts in tension, or bolts in tension with cyclic loading (RCSC 4.2).
- Washers (RCSC Section 6), bolt grip and length (RCSC Section 2.3).
- Wrench clearances to the web weld, the beam flanges and the support (C1 from Manual Tables 7-15 and 7-16).
- Shared bolts: erection-safety note (OSHA 29 CFR 1926.756(c)(1)). HSS: through-bolt note.

### Structural integrity (B3.9)

With **Structural integrity tension (AISC 360 B3.9)** ticked, ILX checks T = max(2/3 Vu or Va, 10 kips) against nominal strengths: bolt tension, end-plate prying (and column-flange prying), the welds at θ = 90°, and the beam web rupture at the weld (Design Example II.A-11C). It also reports the largest column force the connection can brace (150 Tn LRFD, 100 Tn ASD), and checks B3.9(c) when a column axial is entered.

## Skewed end plate

A beam that frames into the support at a plan angle can use a skewed end plate: the end plate stays flat on the support and the beam web meets it at the skew (Manual 16th Ed. Part 10, Skewed Connections, pp. 10-89 and 10-90, Fig. 10-39; Table 10-14B (continued), p. 10-104). Enter the skew in the **Skew** group; with a skew of 0 the connection is designed exactly as a square end plate.

| Field | Default | Notes |
|---|---|---|
| **Skew from square, φ** | 0° | Plan angle of the beam from square, 0 to 45°. A bevel of r in 12 is atan(r/12): 3 in 12 is 14.04°, 6-15/16 in 12 is about 30°. |
| **Beam web end** | Square-cut | Square-cut (Fig. 10-39(a)): the acute corner bears on the plate and the obtuse side opens a root R = tw sin φ. Beveled (Fig. 10-39(b)): the web end is cut to the skew, no root opening and no web thickness limit. |
| **Web to acute-side bolt line, H1** | 0 = auto | Auto: the Table 10-14B H1 for the beam web thickness and the bevel (bolt entering and tightening clearance); outside the table, g/2. The obtuse-side line is g − H1. |

| Field | Default | Notes |
|---|---|---|
| **Obtuse-side fillet** | 0 = auto | Auto: s + A, with s the **Fillet to beam web** (acute side) and A from Table 10-14B. A = 0 for a beveled web. Where the table shows a dash, or outside the table, A is the root opening rounded up to 1/16 in. (0 below 1/16 in.). |

### What changes with a skew

- **Bolts.** The reaction acts in the plane of the beam web, which meets the plate at e = H1 − g/2 from the bolt-group centroid. The Manual says this eccentric loading "should be considered" (p. 10-90). ILX uses the eccentric bolt group coefficient C (Manual Part 7, instantaneous center of rotation) and multiplies the shear transfer at the bolt holes by C/n. Slip resistance uses C in place of n. The steps show e and C.
- **End plate.** The two sides of the web carry unequal shares. The Manual gives no rule for the split, so ILX uses the simple-beam analogy of Design Example II.A-10: the bolt line nearer the web carries R × (far distance)/g. Shear yielding, shear rupture and block shear are checked on one plane for the heavier side.
- **Welds.** The two fillets meet the plate at dihedral angles 90° − φ (acute, leg s) and 90° + φ (obtuse, leg s + A, less the root opening). Each effective throat comes from AWS D1.1:2000 Annex II, (w − R)/(2 sin(Ψ/2)), with root openings under 1/16 in. ignored and the Z loss below 60°. The weld strength and the beam web at the weld (tmin) use the equivalent size of two 90° fillets with the same total throat (Annex II comparable fillet size; Spec J2.2a). The minimum fillet size applies to the measured legs.
- **Prying** (axial tension) uses b = max(H1, g − H1) − tw/2, the longer side.
- **Feasibility.** The web and both fillets must lie between the bolt lines, clear of the holes. The bolt head clearance uses the tighter side.

### Skew checks

| Check | Result | Reference |
|---|---|---|
| Bevel ≤ 6-15/16 in 12 (about 30°) with fillet welds | Compliance. Larger skews need groove welds (AWS D1.1) or bent plates: use the skewed bent plate type. | Manual p. 10-89 |
| Square-cut web: root opening tw sin φ ≤ 3/16 in. | Compliance. Fix: set **Beam web end** to Beveled. | AWS D1.1 5.22.1; Manual p. 10-89 |
| Square-cut web in a Table 10-14B dash cell | Compliance. Fix: bevel the web. | Table 10-14B (continued) |
| H1 ≥ Table 10-14B H1 | Compliance (entering and tightening clearance) | Table 10-14B (continued) |
| Obtuse-side line ≥ 1-3/4 in. from the web (H1 with shared bolts) | Advisory | Table 10-14B (continued) |
| Bolts ≤ 7/8 in., end plate ≤ 3/8 in., supporting web ≤ 3/4 in., tw and bevel inside the table | Advisory: outside these limits check the entering and tightening clearances by layout | Table 10-14B; Manual Tables 7-15, 7-16 |
| Beam flange tip on the acute side clears the support for a square-cut beam end | Advisory: cut the flanges parallel to the support or cope them | Manual Fig. 10-39 (geometry) |
| Skewed T-joint welds: dihedral angles, throats, equivalent size | Note | AWS D1.1 Annex II |

The drawings show the skew in the plan section (the web at its angle and offset, the square-cut or beveled end, both fillets), the skew, bevel, web end and H1 on the web callout, and both fillet sizes (s and s + A) on the weld symbol. The report's input table lists the skew, H1, e, C and both fillets.

**Gaps (not checked; verify separately)**

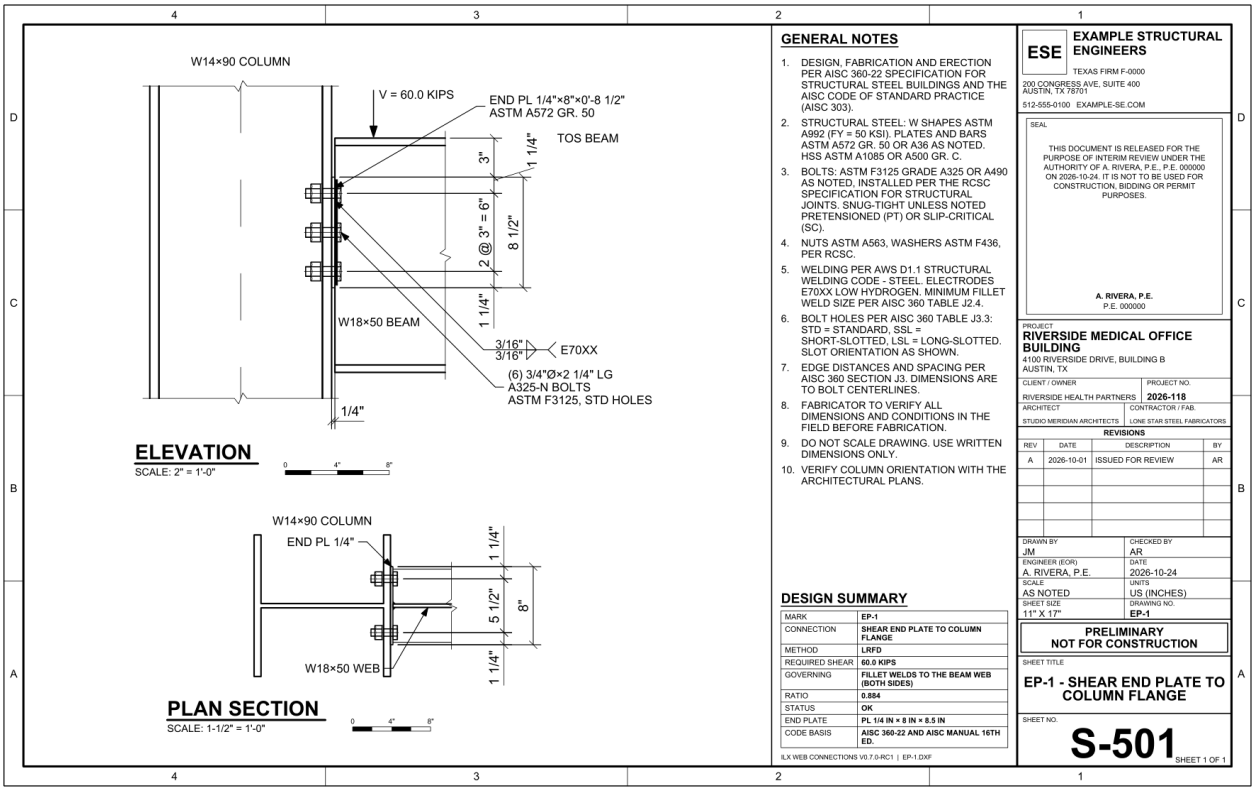
- The axial force is taken normal to the support. Its component parallel to the support face (P sin φ) and the bending from the web offset are not covered by the Manual and are not checked; a note is shown when there is axial force.
- The end-plate share by the simple-beam analogy is an ILX choice (the Manual gives no rule).
- Coped-beam flexure and the rotation check use the square-framing geometry.
- Skews over 6-15/16 in 12 (groove welds per AWS D1.1) are not designed with this type.

## End-plate drawings, report and sheet

On screen the editor shows three views:

- **Elevation**: the support in profile, the end plate edge-on, the beam with radiused copes, the bolts in side view through the plies, the vertical bolt chain, the weld symbol (both sides, shop), and the bolt, plate and cope callouts.
- **Plan**: a section looking down, with the end plate, the beam web and both fillet welds, the bolts, the gage chain and the plate width.
- **End**: the end-plate detail seen from the support side, with the holes, bolt heads, pitch and gage chains, and the web and welds behind as hidden lines.

Select an element in a drawing to open its fields. The bolt callout gives the bolt length for its grip.




The drawing sheet carries the elevation and the plan. The end-plate detail is on screen and in the report only, because a third view would force every view to a smaller scale. The report has the input table, every check with its calculation and the three views. See Drawings and Reports.

## Auto-design for end plates

Auto-design uses the shared dialog: see Auto-design, compare and batch. For a shear end plate it searches, from the lightest hardware up:

- **Bolt diameter**: 3/4, 7/8 and 1 in.;
- **Rows, n**: from 2 up to the number that fits on the web flat (12 at most);
- **Thickness**: 1/4, 5/16 and 3/8 in. first; 7/16 to 3/4 in. only when the loads include axial tension;
- **Fillet to beam web**: 3/16 to 3/8 in., the smallest that passes;
- **Top of beam to top of plate**: moved to the top of the web flat when the current position does not fit;
- **Gage**: falls back to 5-1/2 in. when the bolts do not fit the support.

Pinned fields are kept. The bolt grade, spacing, edge distances and materials are not searched. The objectives are **Lightest (steel weight)** (end-plate weight), **Fewest bolts** and **Lowest cost (fabrication estimate)** (bolts, welds and plate).

Compare and batch work for this type. The batch columns are `mark`, `beam`, `support`, `RD` and `RL`.

## Limitations and deviations

**Scope**

- Rectangular HSS only. A round HSS support is NOT FEASIBLE.
- Sloped beams, seismic (AISC 341) demands and moments are not modeled. The out-of-plane eccentricity is neglected, as the Manual does for shear end plates. Skewed beams are covered up to 6-15/16 in 12 with fillet welds; see Skewed end plate for what is and is not checked.
- Blind bolts on HSS walls are noted only.

**Simplifications**

- The girder-web and HSS-wall checks under axial load use approximate yield-line mechanisms, flagged in the results; the Manual gives no equation for these cases.
- With connections on both sides of a web, opposite axial forces are not netted (conservative).
- Prying on a column flange uses the end plate's tributary length (conservative for a continuous flange).
- The slip resistance uses hf = 1.0; the filler thickness does not change it.
- The 3/8-in. flexibility limit becomes advisory when axial tension is present. This is an ILX rule, following Design Example II.A-11B, which uses a 1/2-in. plate.

**Verification**

- II.A-11A (shear end plate to a girder web), LRFD and ASD: all 19 values match, within 0.6 %.
- II.A-11B (axial tension with prying): 31 of 32 values match, within 0.8 %. The required weld size differs: AISC reads Manual Table 8-4 at 30°, conservatively, and gets 2.08 sixteenths; ILX applies the directional increase at the true angle, 38.7°, and gets 1.96. The 3/16-in. weld is adequate either way.
- II.A-11C (structural integrity): all 12 values match, within 0.9 %.
- Manual Table 10-4: the values printed in the examples match; the other rows are reproduced from the table's stated basis.

See How ILX was verified.

CHAPTER 8

# Designing a seated connection

A seated connection carries the beam reaction on a seat below the bottom flange, with a top angle (or a side angle) for stability. ILX designs unstiffened seats (a seat angle bolted or welded to the support, Manual Tables 10-5 and 10-6) and stiffened seats (a seat plate on a stiffener plate, welded to the support, Manual Table 10-8).

## When to use a seated connection

- Use a seated connection when the beam must be set on the support during erection, for example a beam framing into a column web where a framed connection is hard to erect, or when the connection should sit below the beam.
- Use an unstiffened seat angle for moderate reactions. Use a stiffened seat when the reaction is too large for a seat angle.
- Seated connections carry vertical shear only. For axial force, use a framed connection: a single plate, a double angle or a shear end plate.
- Seats frame to a column flange, a column web or an HSS column wall. They do not frame to a girder web.

## Creating one in the wizard

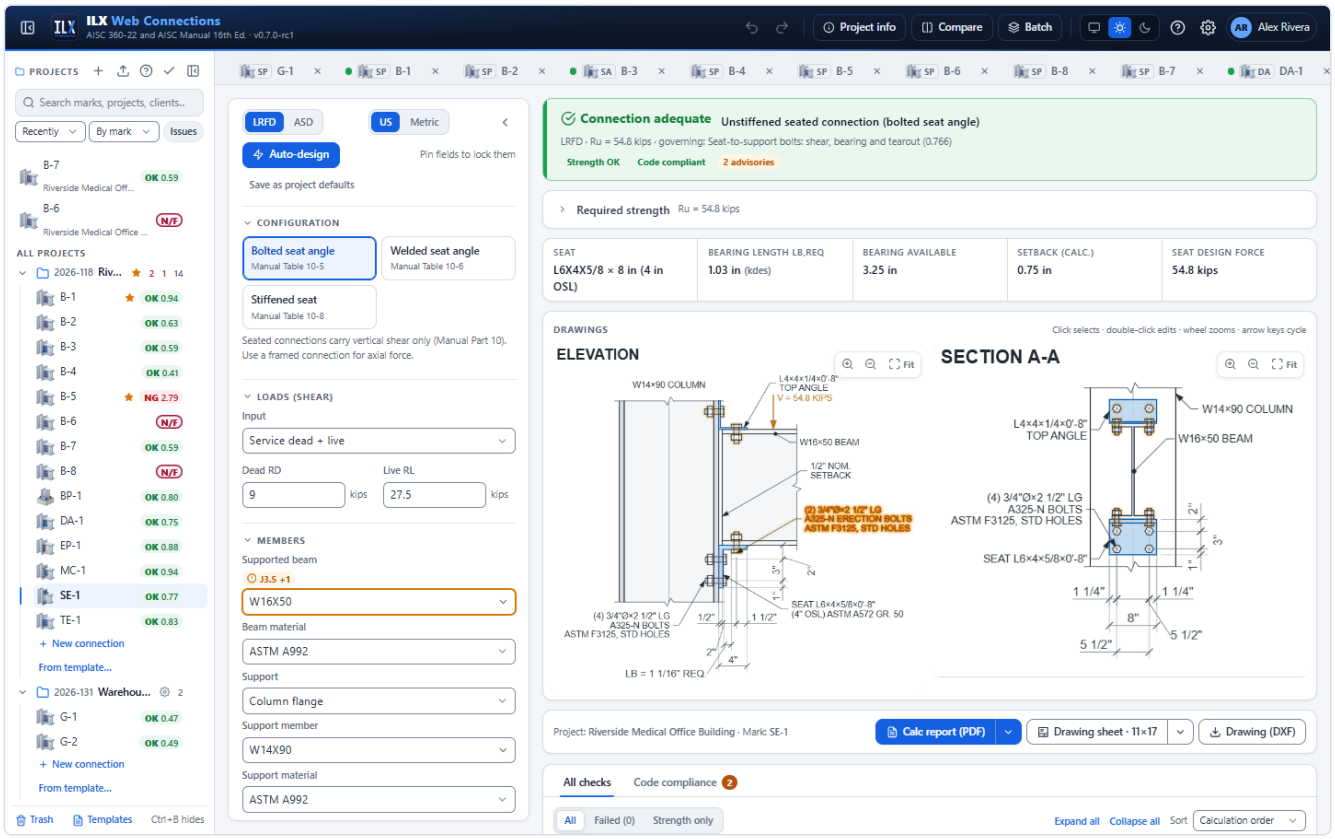
1. In the wizard's **Connection type** step, select the **Seated** card in the **Simple shear** group.
2. Under **Joint type**, select **Beam to column flange**, **Beam to column web** or **Beam to HSS column**. **Beam to girder web** is not offered. If you come from another type with a girder web, the joint becomes a column flange. On an HSS column a bolted seat angle is bolted to the HSS wall (see Bolted to an HSS wall).
3. Under **Configuration**, select a card:

| Card | Sets | Manual basis |
|---|---|---|
| **Bolted seat angle** | Seat angle bolted to the support; beam bolted to the seat | Table 10-5 |
| **Welded seat angle** | Seat angle welded to the support along both vertical edges | Table 10-6 |
| **Stiffened seat** | Seat plate and stiffener welded to the support | Table 10-8 |

On an HSS column all three cards are available; **Bolted seat angle** uses through-bolts by default.

1. **Members.** Enter the **Connection mark** (new marks are numbered SE-1, SE-2 and so on), the **Supported beam**, the **Column** or **HSS column**, and, for a seat angle, the **Seat angle**.
2. **Loads.** Choose **Service D + L** (Dead RD, Live RL) or **Required strength** (Shear Ru or Ra). There is no axial field; the step notes *Seated connections carry vertical shear only.* The other load modes are in the editor.
3. Tick **Auto-design the connection (seat, bolts, welds) for these loads** to size the connection as it is created. The wizard keeps the beam, the support, the joint type and the configuration (seat type and attachment), and uses the **Lightest (steel weight)** objective.
4. Select **Create**.

## Inputs in the editor




The input panel has these sections, in this order: **Configuration**, **Loads (shear)**, **Members**, **Seat angle** (or **Seat plate and stiffener**), **Bolts**, **Erection bolts (seat to beam flange)**, **Welds**, **Stability (top or side angle)** and **Additional checks**. The **Welds** section shows only for a welded or stiffened seat. Clicking an element on the drawing opens its fields.

The key-geometry strip under the status shows **Seat** (for example L6X4X5/8 × 8 in (4 in OSL)), **Bearing length lb,req** with what governs it (yielding, crippling or kdes), **Bearing available**, **Setback (calc.)** and **Seat design force**.

### Seat configuration

| Button | Notes |
|---|---|
| **Bolted seat angle** (Manual Table 10-5) | Disabled on an HSS wall (*Seats are welded to HSS walls*) |
| **Welded seat angle** (Manual Table 10-6) | |
| **Stiffened seat** (Manual Table 10-8) | Always welded |

The section notes that seated connections carry vertical shear only, and that a framed connection should be used for axial force.

### Loads (shear only)

The **Loads (shear)** section works as described in Loads and load combinations, without the axial fields. All four **Input** modes are offered: **Service dead + live** (Dead RD, Live RL), **Service components → ASCE 7 combinations** (RD, RL, RLr, RS, RW, RE), **Required strength Ru** (Ra in ASD) and **Factored load cases (table)** (a name and V per case). With several combinations, each check is enveloped and the **Combo** column shows the governing one; shear is taken as |V|.

If a file contains axial input (PD, PL or P), a note says it is not used. The check **Axial force: not applicable to seated connections (shear only)** then fails, and no strength check uses the axial force.

### Members and support

| Field | Options and notes |
|---|---|
| **Supported beam**, **Beam material** | A W, M, S or HP shape |
| **Support** | **Column flange**, **Column web** or **HSS column wall** |
| **Support member**, **Support material** | A W shape, or an HSS for an HSS wall. HSS materials are limited to A500, A1085 and A53 Gr. B. |
| **Connected HSS face** | HSS wall only: **B (wide face)** or **H (narrow face)** |
| **Bolts into the HSS**, **Blind bolt available strength per fastener (manufacturer)** | Bolted seat on an HSS wall only: through-bolts (default) or blind bolts with the manufacturer's value (default 0); see Bolted to an HSS wall |
| **Seats on both sides of the web (same level)** | Column web only. The support thickness is shared by the two seats (bolt bearing, and base metal by Manual Eq. 9-7, 6.19 D/Fu). For a stiffened seat it replaces the web yield-line check. |
| **Setback (nominal)**, **Beam length underrun** | Default 1/2 in. and 1/4 in. The calculations use setback + underrun (3/4 in., Manual Part 10). |
| **Beam offset from support centerline** | Beam centerline off the column centerline (Design Example II.A-16) |

Switching **Support** to **HSS column wall** changes a W-shape member to an HSS12X12X1/2 in A500 Gr. C (rectangular); a bolted seat angle stays bolted (to a round HSS it becomes welded). Switching back from an HSS changes the member to a W14X90 in A992.

### Seat angle, or seat plate and stiffener

For a seat angle (**Seat angle** section):

| Field | Options and notes |
|---|---|
| **Seat angle** | An AISC L shape (default L6X4X5/8) |
| **Angle material** | A572 Gr. 50 (default), A36 and so on |
| **Outstanding leg (under the beam)** | **Short leg** or **Long leg**. The outstanding leg (OSL) sets the bearing length available. |
| **Angle length** | Along the support face (default 8 in.) |
| **Design bearing length (0 = lb,req)** | The lb used for the outstanding-leg flexure. The Manual tables are entered with lb rounded up (for example 1-1/8 in.); 0 uses the computed lb,req. |
| **Seat shift from beam centerline** | Moves the seat along the support face |

For a stiffened seat (**Seat plate and stiffener** section):

| Field | Options and notes |
|---|---|
| **Seat width W (projection)** | The projection from the support face |
| **Stiffener length l** | The vertical length of the stiffener and its welds |
| **Stiffener thickness**, **Plate material** | 3/8 in. to 1 in.; the material applies to the seat plate and the stiffener |
| **Seat plate thickness**, **Seat plate width (along the support)** | 3/8 in. to 3/4 in.; width B |
| **Stiffener corner clip** | Reduces the fitted bearing area under the seat plate |
| **Seat shift from beam centerline** | As for the seat angle |

**Beam bearing stiffeners (J10.8)** is a section of its own, below the seat:

| Field | Options and notes |
|---|---|
| **Beam bearing stiffeners over the seat** | **None** (default; saved connections are unchanged), **Design when required** or **Provided (as entered)** |
| **Stiffener thickness** | Provided only; otherwise designed (the next standard plate) |
| **Stiffener width (0 = max)** | 0 uses the largest 1/8 in. that fits (bf − tw)/2 |
| **Stiffener length** | **Full depth** (default) or **Half depth (from the bottom flange)** |
| **Corner clip (min.)**, **Stiffener fillets (min.)**, **Stiffener material** | Clips and fillets are increased as needed with **Design when required** |
| **Finished to bear on the bottom flange (J7)** | On by default: fitted bearing on the loaded flange, checked with Eq. J7-1 |

## Bolts and holes

**Bolt grade**, **Bolt diameter** and **Threads** apply to every bolt: the seat bolts, the erection bolts and the top angle bolts. For a bolted seat angle the section also shows:

| Field | Options and notes |
|---|---|
| **Joint** | **Bearing (snug / pretensioned)** or **Slip-critical**; **Surface Class A (μ = 0.30)** or **Class B (μ = 0.50)** |
| **Rows**, **Columns** | 1 to 4 each, under **Seat to support** |
| **Heel to first row, g1**, **Row pitch**, **Column gage** | Rows go down the vertical leg from the heel; columns go across |
| **Holes in angle**, **Holes in support** | **STD**, **OVS**, **SSL horizontal**, **SSL vertical**, **LSL horizontal**, **LSL vertical** |
| **Hole deformation at service is a design consideration (2.4 / 1.2)** | Ticked: bearing 2.4 d t Fu and tearout 1.2 lc t Fu. Cleared: 3.0 and 1.5. |

## Erection bolts

The **Erection bolts (seat to beam flange)** section places the bolts through the seat and the beam bottom flange. They hold the beam in place and carry no design load.

| Field | Options and notes |
|---|---|
| **Number of erection bolts** | 0 to 4 (2 required by Manual Part 10) |
| **Holes** | **STD**, **OVS**, **SSL across the beam** or **SSL along the beam** |
| **Gage across the flange**, **Support face to bolt line** | The bolt gage and the distance x from the support face |

## Seat welds

| Field | Shows for | Notes |
|---|---|---|
| **Fillet, each vertical edge** | Welded seat angle | One fillet along each vertical edge of the vertical leg |
| **2w returns at the top** | Welded seat angle | The returns of Manual Table 10-6 |
| **Stiffener + seat plate to support (both sides)** | Stiffened seat | Both faces of the stiffener, and the seat plate to the support |
| **Seat plate weld length each side (0 = 0.2 l)** | Stiffened seat | Seat plate to support, each side of the stiffener (Manual Fig. 10-11b) |
| **Seat plate to stiffener (both sides)** | Stiffened seat | Same length as the seat-plate-to-support weld |
| **Electrode** | Welded and stiffened seats | E70XX or E80XX |

## Top or side angle

| Field | Options and notes |
|---|---|
| **Stability** | **Top angle (top flange)**, **Side angle (top of web)** or **None**. **None** fails a code-compliance check. |
| **Angle**, **Material**, **Angle length** | Default L4X4X1/4 in A36, 8 in. long |
| **To the support** | Top angle only: **Bolted (2 bolts)** or **Welded along the toe only**, with the **Toe weld** size |
| **Gage, heel to bolt line**, **Bolt spacing** | The bolt line in each leg and the spacing of the two bolts. The beam leg is always bolted. |

## Additional checks

Tick **Structural integrity tension (AISC 360 B3.9)** to add the integrity checks as an independent case. See Structural integrity.

Tick **Cyclic loading (fatigue, AISC 360 Appendix 3)** to design the seat for fatigue, with the cycle count and the cyclic range of the reaction as for the single plate; see Fatigue (Appendix 3). The details: the seat outstanding leg in bending at the critical section (rolled base metal, Table A-3.1 Case 1.1, A), the vertical leg at the bolt holes (Section 2, or Case 1.5 for snug-tight joints), the seat-to-support welds (two lines of the vertical-leg length with the reaction at setback + lb/2, elastic, Case 8.2, F) and, for a stiffened seat, the stiffener-to-support welds (the reaction at 0.8 W, Case 8.2, F).

## The checks it runs

Each check cites the clause shown below. See Checks and compliance for reading the results.

### Beam web at the seat

- **Required bearing length** lb,req = max(lb,yielding, lb,crippling, kdes) (Manual Eqs. 10-1 to 10-3, Table 9-4 constants R1 to R6). For a seat angle, lb,req must fit on the OSL minus setback and underrun; for a stiffened seat, W ≥ lb,req + setback + underrun.
- **Beam web local yielding at the seat** (J10.2, Eq. J10-3) and **Beam web local crippling at the seat** (J10.3, Eq. J10-5a or J10-5b), at the bearing length available.
- **Beam web shear (end of beam)** (G2.1).
- With an offset beam (II.A-16), the seat elements are designed for twice the more highly loaded weld line or bolt column, shown as **Seat design force**.

### Beam bearing stiffeners over the seat

With **Beam bearing stiffeners** set to **Design when required**, ILX adds a stiffener pair over the seat when the beam web fails local yielding or crippling, or the required bearing length does not fit (with **Provided**, always). The pair stands mid-way along the bearing available on the seat and carries the **whole reaction** (the rule ILX uses for every bearing stiffener):

- **Bearing stiffeners, over the seat: full load (J10.8, J4.4)**: two stiffeners plus a 12tw web strip (the beam end) as a column, Lc = 0.75h; Lc/r ≤ 25 gives Pn = Fy Ag (Eq. J4-6), otherwise Chapter E.
- J10.8 detailing: width + tw/2 ≥ b/3 of the seat, thickness ≥ half the seat thickness and ≥ width/16, length ≥ ½ depth (¾ when crippling governs); corner clips that clear the beam fillets.
- **Fitted bearing stiffeners on the loaded flange (J7)**, 1.8 Fy Apb with Apb net of the clips (or fillet welds to the flange when not fitted); fillet welds to the web; beam web shear rupture along those welds (J4.2); minimum fillet size.
- **Bearing stiffeners stand over the seat bearing** and, as a recommendation, **clear the erection bolt nuts** on the bottom flange.

When the stiffeners are adequate, **Beam web local yielding / crippling at the seat** become recommendations titled *bearing stiffeners required*, the bearing-length fit (lb,req, or W ≥ lb,req + setback for a stiffened seat) becomes a recommendation as long as the bearing available is at least kdes, and the seat flexure takes the reaction at the stiffeners (e = setback + underrun + lb,available/2 − (t + 3/8 in.)). The drawings show the pair edge-on in the elevation and face-on with its corner clips in Section A-A.

The column under the seat has no J10 check: the seat reaction runs along the column, not normal to its flange (J10 covers forces normal to the flange). On a column web, the Ellifritt and Sputo yield line above is the support check.

### Outstanding leg of the seat angle

- **Seat angle outstanding leg: shear yielding**, 0.60 Fy L t (J4.2).
- **Seat angle outstanding leg: flexural yielding** (F11), Mn = Fy L t²/4 at the critical section t + 3/8 in. from the back of the vertical leg, with the reaction at mid-bearing.
- Recommendations: thickness 3/8 in. to 1 in. (the range of Tables 10-5 and 10-6), and seat length ≥ beam flange width.

### Seat-to-support bolts

- **Seat-to-support bolts: shear, bearing and tearout** (J3.11): the sum of the least of shear, bearing and tearout of each bolt. The angle tears out toward the bolt above, the support toward the bolt below. The eccentricity of the bolt group is neglected, as in Table 10-5. The calculation also shows the Table 10-5 tmin for bolt shear to govern.
- **Seat-to-support bolts: slip resistance** (J3.9), for slip-critical joints.
- **Seat angle vertical leg: shear rupture through the top bolt row** (J4.2). This is not a Table 10-5 limit state; ILX checks it for completeness.
- Detailing: row pitch and gage (J3.4), edge distances below the bottom row, to the angle ends and in the column flange (J3.5), maximum edge distance (J3.6), tightening clearances to the outstanding leg, the column web or the column flanges (Manual Part 7), hole types that need slip-critical bolts (J3.3), long slots in one ply only, pretensionable bolts and Group C sizes (J3.2).

### Seat-to-support welds

- **Seat angle to support welds** (J2.4, Manual Table 10-6 elastic method): two vertical welds over the vertical leg length, with the reaction at mid-bearing.
- Base metal: **Support thickness ≥ tmin to develop the seat welds** (Manual Eqs. 9-6 and 9-7). When the support is thinner, the weld strength is reduced by t / tmin.
- Weld size: minimum (Table J2.4) and maximum t − 1/16 in. along the angle edge, length ≥ 4w (J2.2b), and the 2w returns.

### Stiffened seat

- **Stiffener and seat plate to support welds** (J2.4, Manual Table 10-8, e = 0.8 W): two welds l plus 0.2 l under the seat plate each side. Base metal and weld reduction as for the welded seat.
- Seat-plate welds ≥ 0.2 l each side, and no longer than the seat-plate overhang (B − ts)/2. The seat-plate-to-stiffener weld must be at least the size of the seat-plate-to-support weld.
- Stiffener thickness: to develop the seat-plate welds (1.5w for Fy = 50 ksi, 2w for Fy = 36 ksi with E70), and ≥ (Fy,beam / Fy,stiffener) tw.
- **Stiffener width-to-thickness W/ts ≤ 0.56 sqrt(E/Fy)** (B4.1), local buckling.
- **Stiffener bearing under the seat plate (fitted)** (J7, Eq. J7-1), 1.8 Fy (W − clip) ts.
- Recommendations: seat plate ≥ 3/8 in.; stiffener finished to bear.

### Column web and HSS wall

- **Column web yield line under the stiffened seat (Ellifritt & Sputo)**: stiffened seats on a column web, Ellifritt and Sputo (1999) Eq. 2 with the 2000 errata, φ = 0.90 / Ω = 1.67. With seats on both sides of the web at the same level, it is replaced by a note.
- Stiffened seat on an HSS wall: **HSS face width ≥ max(0.4 l, 2 × seat weld) + ts + 2 (2.25 t)** (Design Example K.5), and the HSS wall strength factor **Ru W / t²** against Manual Table 10-15. The table covers square and rectangular HSS with Fy = 50 ksi and Fu = 62 ksi (ASTM A500 Grade C; its header), widths B = 5, 5.5, 6, 7, 8, 9, 10, 12, 14, 16, 18 and 20 in., and stiffener lengths l = 6 to 32 in. Inside it, ILX reads the table (LRFD or ASD column) and checks the required factor against it. A length between whole inches uses the next lower tabulated l: the Manual warns against interpolating. A "–" entry fails as a detailing check, because the Manual says the HSS flat is then too narrow for a 0.2l weld on each side of the stiffener. Outside the table (another width, length or material, e.g. A1085 with Fu = 65 ksi), ILX reports the required factor as a recommendation and says the table does not cover the case.
- Seats on an HSS wall must sit on the flat of the wall (B − 3t); on a round HSS, within half the diameter.

### Erection bolts and stability angle

- **Two erection bolts connect the beam flange to the seat**; gage ≥ 2⅔d (J3.4); edge distances to the seat toe, the beam end (with underrun), the seat ends and the beam flange tips (J3.5; the flange tip is advisory); tightening clearance to the seat vertical leg or the stiffener.
- Top or side angle: a stability angle is required (Manual Part 10); thickness ≥ 1/4 in.; thicker than 1/4 in. not recommended (Manual 16th Ed.) and legs ≥ 4 in. (advisory); toe weld size limits (J2.2b); bolt edge distances, gage and tightening clearance.

### Structural integrity (B3.9)

With **Structural integrity tension** ticked, ILX checks the nominal tension T = (2/3) Vu (LRFD) or Va (ASD), at least 10 kips (B3.9(b), Design Example II.A-12B), against:

- erection bolt shear, bearing 3.0 d t Fu and tearout 1.5 lc t Fu on the seat;
- block shear of the beam bottom flange;
- bolted seat: seat bolt tension (top row) and seat angle bending with prying (Manual Part 9);
- welded seats: the welds in tension (approximate).

It also reports the column bracing capacity, B3.9(c).

### Geometry feasibility

The connection is **NOT FEASIBLE** (see NOT FEASIBLE) when, for example, the seat gives no bearing beyond the beam end, the OSL is shorter than lb,req + setback + underrun, the seat does not fit the column flange, the web between the fillets (T) or the HSS flat, seat bolts are off the vertical leg or in a web fillet, erection bolts miss the beam flange or the seat, the top angle is longer than the web T, or a side angle is deeper than the beam web clear depth.

## Bolted to an HSS wall

With **Support** = **HSS column wall** and a **Bolted seat angle**, the seat-to-support bolts go into the wall of a rectangular HSS. This is the less common case: with HSS columns the connecting element is typically shop-welded to the HSS and bolted to the beam (Manual 16th Ed. p. 10-96), and Manual Table 10-5 does not cover HSS supports.

**Inputs** (Members section, shown only for a bolted seat on an HSS wall)

| Field | Default | Notes |
|---|---|---|
| **Bolts into the HSS** | **Through-bolts** | **Through-bolts (snug-tight, J3.11b)**: long bolts through the whole tube, both walls (Manual p. 7-12). **Blind bolts (manufacturer, J3.12)**: special fasteners installed from one side (Manual p. 7-13). |
| **Blind bolt available strength per fastener (manufacturer)** | 0 | Blind bolts only: the manufacturer's available shear strength per fastener in the design method you use (φRn for LRFD, Rn/Ω for ASD), in kips or kN. 0 fails the design until you enter it. |

### Checks

- Through-bolts: each seat bolt takes the least of the bolt shear (Table J3.2, single shear at the seat/HSS interface), the seat angle bearing and tearout (J3.11a), and the HSS wall bearing and tearout of Spec J3.11b: bearing Rn = 1.8 Fy d t (J7, Eq. J7-1, φ = 0.75, Ω = 2.00) and tearout 1.2 lc t Fu (Eq. J3-6g; 1.5 lc t Fu, Eq. J3-6h, when deformation is not a design consideration; 1.0 lc t Fu, Eq. J3-6i, for long slots perpendicular to the force). t

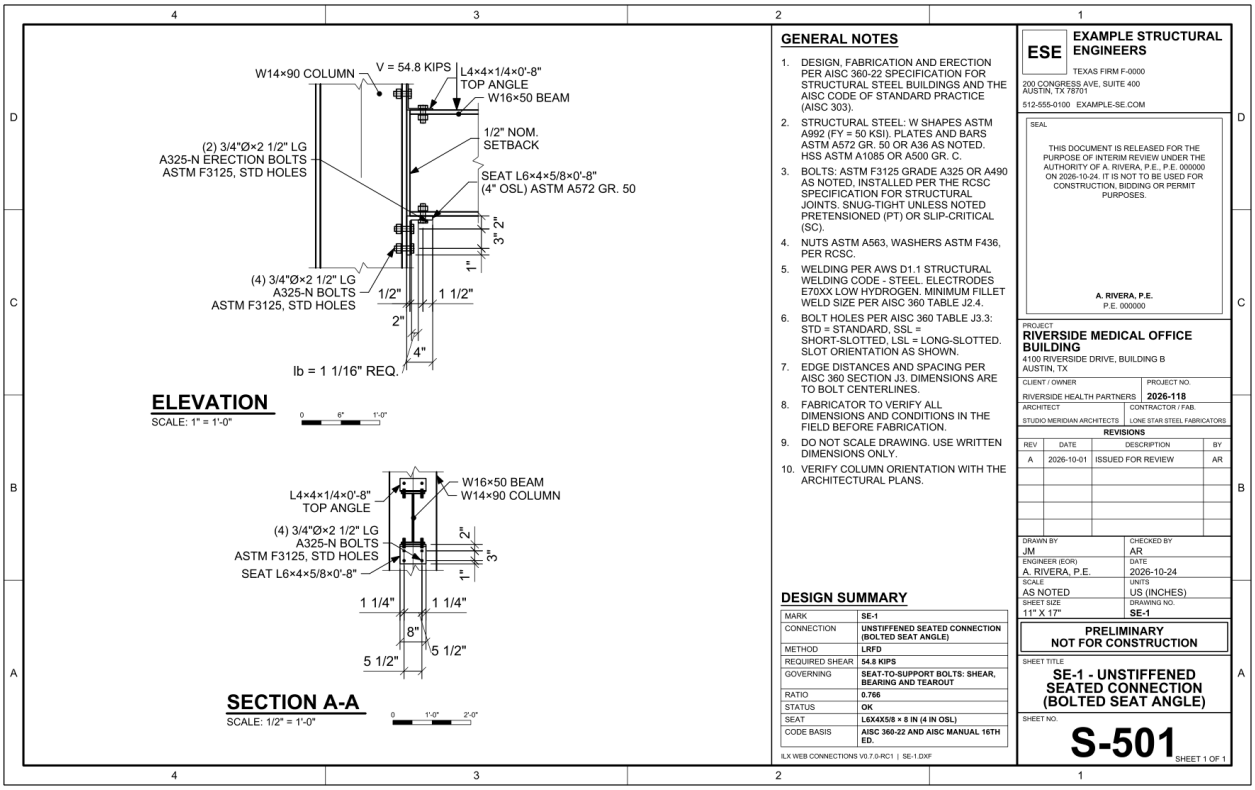
is the HSS design wall thickness, Fy and Fu those of the HSS material; the wall tears out only toward the bolt below (the column is continuous).

- Through-bolts are snug-tight only: the flexibility of the HSS walls precludes pretensioning (Manual p. 7-12). A **Slip-critical** joint fails.
- The bolt grip is the seat angle leg plus the whole HSS depth across the connected face. Long bolts may need ASTM A449 (Manual p. 7-12); the threads-excluded check uses this grip.
- Blind bolts: each bolt takes the least of your manufacturer value, the HSS wall bearing and tearout (J3.11a with the design wall thickness) and the seat angle bearing and tearout, with the bolt value labelled as manufacturer's data (J3.12).
- Both kinds: every hole must lie on the flat of the connected HSS face, or the hole check fails (NG). The whole seat angle must also sit on the flat (NOT FEASIBLE otherwise). Seated connections carry no axial force, so the J3.13 bolt-tension case does not arise.

**Gaps.** The HSS-wall limit states of a blind bolt (tear-out of the bolt in tension, wall distortion) are the manufacturer's: confirm them with the product literature. There is no HSS wall plastification check under the seat (as for the welded seat, Design Example K.4). Through-bolts pass through both walls: coordinate them with any connection on the opposite face. Round HSS is not available. The top angle bolts are detailing only, as on other supports.

The drawing shows through-bolts crossing the tube with the nut on the far wall, and the seat bolt callout adds *THROUGH-BOLTS, SNUG-TIGHT (AISC 360 J3.11b)* or *BLIND BOLTS PER MANUFACTURER (AISC 360 J3.12)*.

## Drawings, report and sheet



ILX draws two views:

- **ELEVATION**: the support profile, the beam end at the nominal setback, the seat angle (or seat plate and stiffener), the top or side angle, every bolt and weld, the setback, bearing-length and bolt dimensions, and the AWS weld symbols.
- **SECTION A-A**: looking at the support face, with the beam cross-section, the seat with its bolt pattern or edge welds (with returns), the stiffener, the top angle, the support edges and fillet lines, the bolt gages and the offset from the support centerline.

The weld symbols follow the calculation: one fillet on each vertical edge of a seat angle (with the return length in the tail, or *NO RET.*), both sides of the stiffener, the seat plate to the support *EA. SIDE*, and a field weld along the toe of a welded top angle. Bolt callouts give each group's bolt length.

The drawing sheet carries both views, the general notes and the design summary, with the seat description in the **SEAT** row. The report is titled **Seated Beam Connection**, and its input summary states that axial force is not applicable. See Drawings and Reports.

## Auto-design, compare and batch

**Auto-design** uses the shared dialog; see Auto-design, compare and batch. It never changes the seat type or attachment. It searches:

- **Seat angle (bolted or welded):** seat angles from L4X4X3/8 to L8X4X1, lightest first, with the short or long leg out and lengths of 6 or 8 in. A bolted seat also searches 1 to 3 bolt rows and, unless pinned, 3/4, 7/8 and 1 in. bolts, with g1 set for the tightening clearance. A welded seat searches fillets from 1/4 to 1/2 in., up to t − 1/16 in.
- **Stiffened seat:** W from 5 to 9 in., l from 9 to 24 in. and fillets from 1/4 to 7/16 in. (the same size seat plate to stiffener). The stiffener thickness is set from the weld, the W/ts limit and the beam web.

Pinned fields are kept. The objectives are **Lightest (steel weight)**, **Fewest bolts** (seat and erection bolts) and **Lowest cost (fabrication estimate)**.

**Compare** works as for the other types. **Batch** starts with the columns `beam`, `support`, `RD` and `RL`.

## Limitations and documented deviations

- **Shear only.** Axial input is reported as not applicable and never used.
- **No girder supports.** Bolted seats on a rectangular HSS wall use through-bolts or blind bolts; see Bolted to an HSS wall.
- **Bolted stiffened seats** (two angles as the stiffener, Manual Table 10-7) are not available.
- **Manual Table 10-15 (stiffened seats to HSS)** is looked up only where it applies (Fy = 50 ksi and Fu = 62 ksi, tabulated widths and lengths). K.5 (HSS14, l = 24 in.) reads 4,250 / 2,830 kip/in. against the required 3,880 / 2,590 kip/in. Outside the table, ILX reports only the required factor.
- **Unstiffened seats on HSS walls** are checked for base metal only, as in Design Example K.4. There is no wall plastification check. On a round HSS, the seat is kept within half the diameter as a flat-face proxy.
- **Column web yield line.** The φ = 0.90 / Ω = 1.67 applied to Ellifritt and Sputo's nominal strength is an engineering choice. Unstiffened seats on a column web have no yield-line check.
- **Seat bolt group.** Eccentricity is neglected (Table 10-5 practice). Unequal bolt-column forces are taken into account only for offset seats, as twice the larger force.

- **Integrity of welded seats** uses the weld throat in tension. Flexure of the seat leg and the support under the integrity tension is not evaluated.
- **Top angle and erection bolts** are detailing only; they are not designed for load, as in the Manual.
- **Design Example II.A-12A bolt shear.** The Example quotes 107 / 71.6 kips for four bolts. ILX gives 71.6 / 47.7 kips (4 × 17.9 and 4 × 11.9 kips); the quoted figures are six-bolt values. The conclusion is unchanged.
- **Verification.** ILX reproduces Design Examples II.A-12A, II.A-12B, II.A-13, II.A-14, II.A-15, II.A-16, K.4 and K.5 in LRFD and ASD, and Ellifritt and Sputo's Examples 1, 3 and 4. The checked values agree within 0.5 % (for example the 113-kip OSL strength as 112.5 kips, the 139-kip Table 10-8 weld as 139.4 kips), and every published conclusion is reproduced. See Verification.

CHAPTER 9

# Designing a shear tee connection

A shear tee connection is a simple shear connection made of one tee: a rolled WT, or a tee built up from two plates. The tee flange is bolted or welded to the support, and the tee stem is bolted or welded to the beam web. ILX designs it to AISC Manual 16th Ed. Part 10 and AISC 360-22, as in Design Examples II.A-30, II.A-31, K.1 and K.2.

## When to use it

- Use a shear tee when you want a one-sided connection that is bolted to the support (the single plate cannot be), or when a single plate or single angle does not have the capacity.
- Use a tee on an HSS column when the flange should be welded to the HSS face (narrow tee, K.2) or across the HSS corners (wide tee, K.1).
- Use a single plate when the support side is shop-welded and a plate is enough.
- Use a double angle for a symmetric connection with angles on both sides of the beam web.
- Use a single angle for light reactions with an angle bolted to the support.

The two parts of the tee have their own names in the app. The **flange** lies against the support. The **stem** lies against the beam web. The stem is offset from the support centerline by (tw + tsw)/2, the half thicknesses of the beam web and the stem.

## Creating one in the wizard

1. In the wizard's **Connection type** step, select the **Tee** card in the **Simple shear** group.
2. Under **Joint type**, select **Beam to column flange**, **Beam to column web**, **Beam to girder web** or **Beam to HSS column**. Selecting the HSS column changes the flange to welded.
3. Under **Configuration**, select a card. The first word is the flange (support side), the second the stem (beam side):

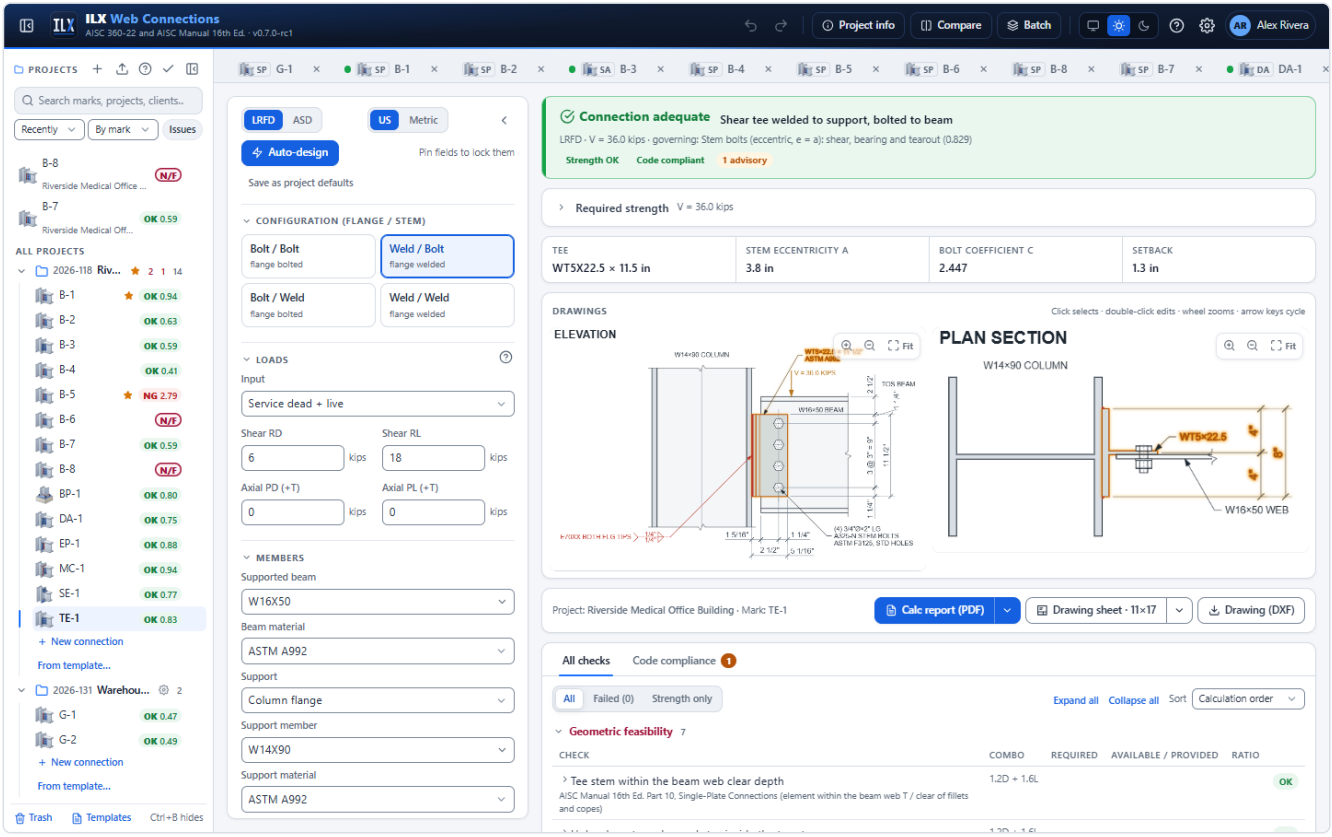
| Card | Sets | Basis |
|---|---|---|
| **All-bolted tee** | Flange bolted to the support, stem bolted to the beam | Example II.A-30 |
| **Welded / bolted tee** | Flange welded to the support, stem bolted to the beam | Example II.A-31 (and K.1, K.2 on an HSS) |
| **Bolted / welded tee** | Flange bolted to the support, stem welded to the beam | Manual Part 10 |
| **All-welded tee** | Flange welded to the support, stem welded to the beam | Manual Part 10 |

On an HSS column, **All-bolted tee** and **Bolted / welded tee** are greyed out. Their tooltip reads *A tee flange is welded to an HSS wall*.

1. **Members.** Enter the **Connection mark** (new marks are numbered TE-1, TE-2 and so on), the **Supported beam**, the **Column**, **Girder** or **HSS column**, and the **Tee (WT)**. A built-up tee is chosen later, in the editor.
2. **Loads.** Choose **Service D + L** (Dead RD, Live RL) or **Required strength** (Shear Ru or Ra, and **Axial** with + for tension). The other load modes are in the editor.

3. Tick **Auto-design the connection (tee, bolts) for these loads** to size the connection as it is created. The wizard keeps the beam, the support, the joint type and the configuration, and uses the lightest (steel weight) objective.
4. Select **Create**.

## Inputs in the editor




The input panel has these sections, in this order. Sections and fields that do not apply to the configuration are hidden: the **Bolts** section for the all-welded tee, the stem bolt fields for a welded stem, and the flange bolt fields for a welded flange. The key-geometry strip under the status shows **Tee** (for example WT5X22.5 × 11-1/2 in), **Stem eccentricity a**, **Bolt coefficient C** and **Setback**.

### Configuration (flange / stem)

| Field | Options and notes |
|---|---|
| Configuration buttons | **Bolt / Bolt** (all-bolted tee, II.A-30), **Weld / Bolt** (II.A-31), **Bolt / Weld**, **Weld / Weld**. The first word is the flange to the support, the second the stem to the beam. Each button says *flange bolted* or *flange welded*. |

Choosing a bolted flange while the support is an HSS wall changes **Support** to **Column flange**. Choose a W-shape **Support member** after the change.

## Loads and axial force

The **Loads** section is the shared one; see Loads and load combinations. All four input modes are supported: **Service dead + live**, **Service components → ASCE 7 combinations**, **Required strength (Ru, Pu)** and **Factored load cases (table)**. Axial tension and compression are both supported. With ASCE 7 combinations or load cases, each combination is checked and the envelope governs.

## Members and support

| Field | Options and notes |
|---|---|
| **Supported beam**, **Beam material** | AISC W shape; ASTM A992, A572 Gr. 50 and so on |
| **Support** | **Column flange**, **Column web**, **Girder web**, or **HSS column wall** (welded flange only). With a bolted flange the panel says *HSS walls need a welded tee flange.* |
| **Support member**, **Support material** | A W shape, or an HSS shape for the HSS wall with an HSS material (A500, A1085, A53 Gr. B). When you change **Support**, pick a matching member. |
| **Connected HSS face** | HSS wall: **B (wide face)** or **H (narrow face)** |
| **Girder TOS − beam TOS (+ = beam lower)** | Girder web only |
| **Connections on both sides of the web** | Column web or girder web. ILX then uses half the web thickness for this connection (bearing on the support, Manual Eq. 9-6 base metal, support shear rupture). |
| **Connection near the column end**, **Column end to the connection force** | Column web or column flange. Cleared (default): the connection is not near the column end. Ticked: enter the distance from the column end to the force. Column web: when it is less than $\sqrt{(8wab/(a + b))}$, ILX halves the Eq. 9-45 yield-line strength (Manual p. 9-18). Column flange with an axial force: web local yielding uses Eq. J10-3 when the distance is ≤ d, and web crippling Eqs. J10-5a/b when it is < d/2 (AISC 360-22 J10.2, J10.3). The report shows the limit, the distance and which equation was used. |

## The tee

| Field | Options and notes |
|---|---|
| **Tee** | **Rolled WT** or **Built-up (plates)** |
| **WT shape** | Rolled WT. The line under it gives d, bf, tf, tsw and the stem offset from the support centerline. |
| **Overall depth d**, **Flange width bf**, **Flange thickness tf**, **Stem thickness tsw** | Built-up tee only |
| **Flange-to-stem fillet** | Built-up tee only: the shop fillet on both faces of the stem |
| **Tee material** | A992, A36, A572 Gr. 50 and so on |
| **Stripped flange width (0 = full)** | Rolled WT only. A narrower flange for a narrow tee on an HSS flat (Example K.2). |
| **Top of beam to top of tee** | Where the tee sits on the beam web; it must clear the flange, fillet and any cope |
| **Tee length, l** | All-welded tee only. With bolts, the length follows from n, s and Lev. |

For a WT, k1 is taken from the parent W shape (WT5X22.5 from W10X45). For a built-up tee, k1 = tsw/2 + the flange-to-stem fillet.

### Beam copes

Tick **Top cope**, **Bottom cope** or both, and enter **Top cope length c**, **Top cope depth dc** (and the bottom equivalents) and the **Reentrant corner radius**. The cope checks are those of the single plate; see Beam copes.

### Bolts and holes

One bolt grade and diameter is used for the stem and the flange. The bolt rows are shared: the flange has two bolt lines with the same n and s as the stem.

| Field | Options and notes |
|---|---|
| **Bolt grade**, **Bolt diameter**, **Threads** | As for the single plate. **Threads**: **Included (N)** or **Excluded (X)**. |
| **Joint** | **Bearing (snug-tight)**, **Bearing, pretensioned** or **Slip-critical**; **Surface Class A (μ = 0.30)** or **Class B (μ = 0.50)** |
| **Rows, n** and **Spacing, s** | At least 2 rows whenever there are bolts |
| **Lev, tee (top & bottom)** | Vertical edge distance at the top and bottom bolts |
| **Bolt group strength** | **Σ per-bolt strengths (Design Examples; default)**, **n (or C) × weakest bolt (conservative)** or **Shear + Σ bearing/tearout (split)** |
| **Hole deformation at service is a design consideration (2.4 / 1.2)** | Ticked: bearing 2.4 d t Fu and tearout 1.2 lc t Fu. Cleared: 3.0 and 1.5 (J3.11). |

### Stem to beam

The section title reads **Stem to beam (bolted)** or **Stem to beam (welded)**.

| Field | Shows | Options and notes |
|---|---|---|
| **Stem bolt columns**, **Column gage** | Bolted stem | 1 to 3 columns; the gage shows with 2 or more |
| **Stem holes**, **Beam web holes** | Bolted stem | **STD**, **OVS**, **SSL** or **LSL**, horizontal or vertical |
| **Leh, stem toe** | Bolted stem | Last stem bolt column to the stem toe. a = tee depth − Leh, stem. |
| **Leh, beam web** | Bolted stem | First stem bolt column to the beam end |
| **Support face to beam end (setback)** | Welded stem | The beam end position |
| **Fillet, stem to beam web**, **Weld shape** | Welded stem | **C-shape**, **Vertical** or **Top & bottom** |
| **Cb, stem lateral-torsional buckling** | Always | Default 1.00, as in Example K.1 |
| **K (stem in compression)** | Net axial compression | Effective length factor for the stem in compression |

### Flange to support

The section title reads **Flange to support (bolted)** or **Flange to support (welded)**.

| Field | Shows | Options and notes |
|---|---|---|
| **Flange bolt gage g (centred on the support)** | Bolted flange | The two bolt lines are centred on the support, not on the stem |
| **Flange holes**, **Support holes** | Bolted flange | Hole types as above |
| **Fillet at each flange tip** | Welded flange | One fillet along each flange tip. On a wide tee on an HSS, ILX uses flare-bevel groove welds at the HSS corners instead. |
| **Flange weld eccentricity** | Welded flange | **Manual Table 10-2 (e = max(4 in., a))** (default, as in II.A-31), **e = a**, or **None (flexible support, K.1/K.2)** |
| **Electrode** | Always | E70XX or E80XX |

### Service conditions and additional checks

**Hot-dip galvanized**, **Unpainted weathering steel**, **Cyclic loading (fatigue, AISC 360 Appendix 3)**, **Structural integrity tension (AISC 360 B3.9)** and **End rotation / flange binding** (with **Required rotation** and the presets **0.03** and **0.025 (341)**) work as for the single plate; see Service conditions and additional checks. Weathering steel also lowers the flange bolt spacing limit to 14t ≤ 7 in.

**Fatigue (Appendix 3).** With **Cyclic loading** ticked the connection is designed for fatigue as for the single plate; see Fatigue (Appendix 3). The details: the tee stem, the tee flange and the beam web at the bolt holes (Table A-3.1 Section 2, or Case 1.5 for snug-tight joints), the stem-to-web fillets (elastic weld group, Case 8.2, F), the flange-tip welds to the support (two lines with the eccentricity of the strength check, Case 8.2, F) and the beam at a cope corner (Case 1.3).

## Checks it runs

The load path follows Manual Part 10: the flange bolts or welds carry the shear Rr; the stem bolts carry Rr at the eccentricity a, from the support face to the stem bolt group centroid. The stem and the beam are checked as a single plate welded to the tee flange, with the full eccentricity a. For reading the results, see Checks and compliance.

**Rotational ductility** (AISC Manual 16th Ed. Part 9, rotational ductility (Eqs. 9-51, 9-52, 9-53; Fig. 9-6))

- Bolted flange: d ≥ dmin, Eq. 9-52, with b from the stem centerline to the nearer bolt line minus k1, and p from Fig. 9-4.
- Welded flange: w ≥ wmin, Eq. 9-51, capped at (5/8) tsw. On a wide tee welded to an HSS, b uses B in place of bf (K.1).
- Bolted stem: tsw or tw ≤ d/2 + 1/16 in., Eq. 9-53. Eq. 9-53 is an alternate: when either rule is met, the other is shown as a recommendation.
- The extended single-plate tmax rule is advisory with one stem bolt column and required with two or more.

#### Stem bolts and stem

- Stem bolts (eccentric, e = a): shear, bearing and tearout by the instantaneous-center method (Manual Part 7), and slip resistance (J3.9).
- Tee stem shear yielding, shear rupture and block shear (J4.2, J4.3); tension, compression and axial block shear with axial load (J4.1, J4.4).
- Tee stem flexural yielding and lateral-torsional buckling as a rectangular bar, with Lb from the support face to the first stem bolt line (AISC 360-22 F11; Design Examples II.A-30, K.1).

- Tee stem flexural rupture, Mn = Fu Znet (J4.5; Manual Eq. 9-8).
- With axial load: stem axial, flexure and shear interaction, yielding and rupture (Manual Part 12, Eqs. 12-2 and 12-3).
- Welded stem: the stem-to-beam weld group, size limits, and the beam web base metal.

**Tee flange and flange bolts**

- Tee flange shear yielding on two planes beside the stem (J4.2(a)); shear rupture through the bolt lines (J4.2(b)); block shear of both flange halves (J4.3).
- Flange-to-support bolts: shear, bearing and tearout, concentric (J3.7, Table J3.2; J3.11; Manual Part 10). Slip resistance, with ksc under tension (J3.9).
- With axial tension: flange bolts in tension with shear and prying, each bolt line with its statical share (J3.7, J3.8, Manual Part 9 prying action); column flange prying.

**Flange welds and support**

- Flange-to-support welds by the instantaneous-center method (J2.4, Table J2.5; Manual Part 8), with the chosen eccentricity.
- Minimum fillet (J2.2b, Table J2.4), maximum tf − 1/16 in. (J2.2b(b)), length ≥ 4w (J2.2b(c),(d)). Flare-bevel welds: minimum effective throat (J2.1, Tables J2.2 and J2.3).
- Support thickness develops the welds, tmin = 3.09 D/Fu, doubled with connections on both sides (Manual Part 9). The tee flange thickness check is a recommendation.
- Support shear rupture along the two weld lines (J4.2(b)). With tension: tee flange bending between the stem and the tip welds.
- Support under axial load: column web yield line (Manual Eq. 9-45; halved near the column end, noncritical when the load spans more than 85 % of the web width) and punching; girder web (approximate); column web local yielding and crippling (J10.2, J10.3); column flange bending under the tip welds (approximate, J10.1); HSS wall plastification or sidewall yielding, and round HSS Eq. K2-2a.

**Beam**: web shear, coped section, block shear, beam tension (D2, D3), cope flexure (Dowswell), framing geometry (girder flange, column-web fit) and end rotation, as for the single plate.

**Structural integrity** (B3.9(b)): the stem-side group plus flange bolts in tension with prying, or flange welds in transverse tension and tee flange bending.

**Geometry and feasibility.** The design is **NOT FEASIBLE** when the beam end does not clear the tee flange, the stem holes fall in the tee fillet (x − dh/2 ≥ kdes), the flange bolts fall in the stem fillet or outside the flange, the tee flange and bolts or welds do not fit on the support, or a tee on an HSS flat is neither narrow nor wide (K.2). Further detailing checks: edge distances on the flange (J3.4, J3.5; a flange-tip edge between d and Le,min is a recommendation under Table J3.4 footnote [a], as in II.A-30), maximum edge and spacing (J3.6), slip-critical requirements for oversized and slotted holes, washers, bolt grip, tightening clearances (Manual Tables 7-15 and 7-16), and a setback of at least 1/2 in. beyond the flange.

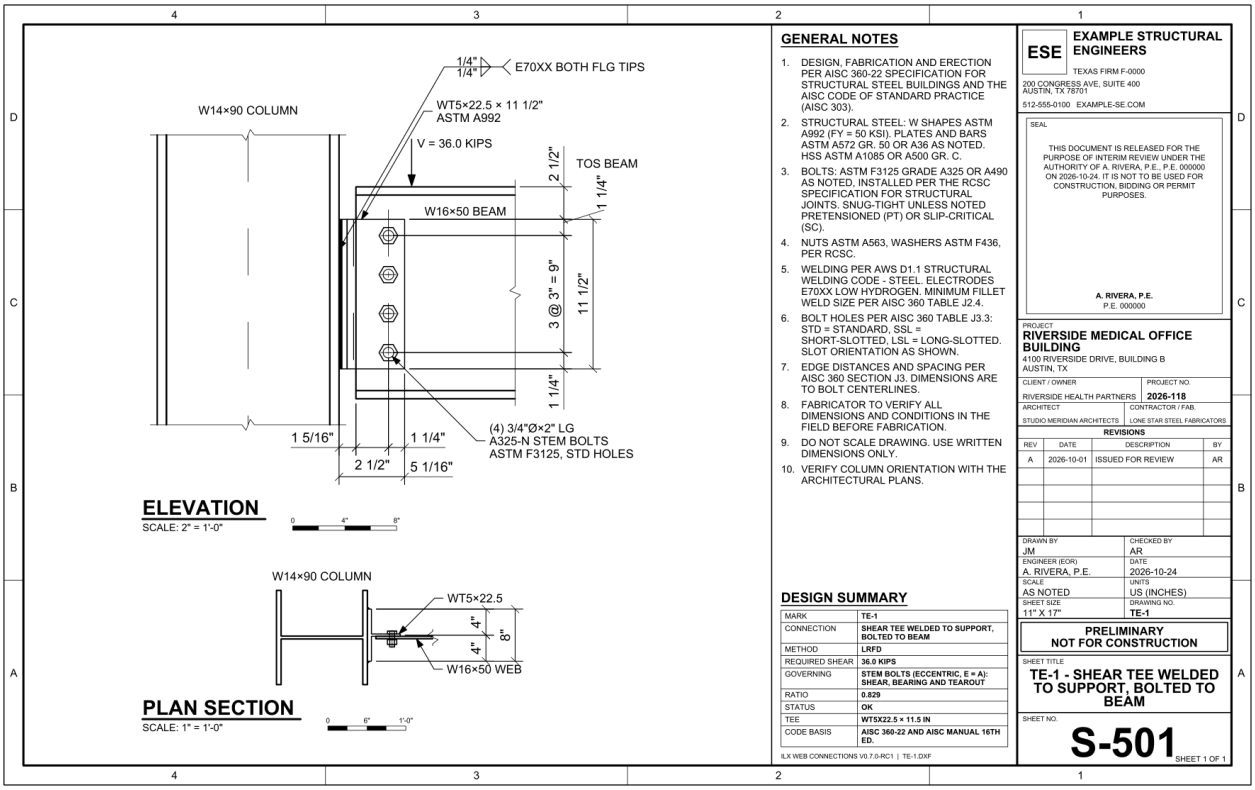
## Drawings, report and sheet

The editor and the sheet show two views:

- **Elevation**: the tee flange edge-on against the support with its fillet line, the stem on the beam web, the flange bolts in side view (head on the support, nut on the tee flange) and the stem bolts in end view.

- **Plan section**: the full T, with the flange across the support face, the flange bolts through the flange and the support, and the stem bolts through the stem and the beam web. On a wide tee on an HSS, the rounded HSS corners with the flare-bevel welds.

The tee callout gives the WT and length (and the stripped flange width), or the built-up flange and stem plates. Each bolt callout gives the bolt length of its group (**STEM BOLTS**, **FLANGE BOLTS**). The weld symbols read **BOTH FLG TIPS**, **BOTH HSS CORNERS** (flare-bevel), the stem weld shape (**3 SIDES**, **END**, **TOP & BOT**, field weld) and **FLG TO STEM** for a built-up tee.




The report is titled **Shear Tee Connection**, with the configuration (for example *All-bolted shear tee*). Its input summary lists the tee and its dimensions (d, bf, tf, tsw, k1), the stem bolts with a, the flange bolts or welds with the weld eccentricity, and the setback. See Drawings and Reports.

## Auto-design, compare and batch

Auto-design searches the bolt rows n (2 to 8), the bolt diameter (3/4, 7/8 and 1 in.) and the rolled tee, from WT4X9 to WT9X43, lightest first. Pinned fields are kept. In the editor, the button's tooltip reads *Search bolt rows, bolt size and the WT; pinned fields are kept*. The objectives are those of the shared dialog:

- **Lightest (steel weight)**: the tee weight, plus the weld weight;
- **Fewest bolts**: stem bolts plus 2n flange bolts;
- **Lowest cost (fabrication estimate)**: bolts, welds and steel.

Auto-design does not change the configuration, the support, the bolt grade, the weld sizes or the tee type. See Auto-design, compare and batch for the dialog.

Compare works for shear tees. Batch design reads the columns `beam`, `support`, `RD` and `RL` (plus any input field name). With **Auto-design each mark**, each mark is searched over the same rows, bolt sizes and tees.

## Limitations and documented deviations

**Verification.** The results match Design Examples II.A-30, II.A-31, K.1 and K.2 within 0.5 percent, in LRFD and ASD: stem bolts, stem shear yielding, rupture and block shear, flange bolts, stem flexure (yielding, rupture, and LTB in K.1), flange welds (Table 10-2 Welds B, 99.7 / 66.5 kips), flare-bevel and fillet welds on the HSS, dmin (0.481 in.), wmin, and the minimum support thickness. K.1 stem flexural yielding is 0.4 to 0.5 percent high because AISC rounds Z to 11.2 in.³ See How ILX was verified.

**Methods that differ from the Design Examples**

- **Weld eccentricity.** The default e = max(4 in., a) reproduces Manual Table 10-2 *Welds B*, which is for double angles; II.A-31 uses it for the tee. It is a user option.
- **Stem lateral-torsional buckling** is checked in every case, with Cb = 1.00 (K.1). II.A-30 and II.A-31 do not check it.
- **Extended tmax** is advisory with one stem bolt column (II.A-30 would fail it), and enforced with two or more.
- **Flange-tip edge distance** below Le,min is accepted down to one bolt diameter (an ILX choice).

**Approximate checks**, flagged in the results: girder-web out-of-plane strength, column-flange bending under the welded flange tips, HSS sidewall yielding for a wide tee (k = 1.5t), HSS face plastification with b1 = bf, and the HSS shelf of max(1/2 in., w + 1/4 in.) in the narrow-tee rule. The Design Examples assume flexible supports and check none of the support checks for shear alone.

**Scope**

- No Manual design table exists for shear tees; the tee is designed by calculation, as in the examples.
- One bolt grade and diameter for the stem and the flange; two flange bolt lines with the stem's rows. The flange bolts are concentric.
- A bolted flange is not allowed on an HSS wall.
- Not checked: flexural yielding of the tee flange under shear alone (the examples state it does not control), skewed or inclined tees, and the Astaneh and Nader test limits (bf/2tf ≥ 6.5, db/ts ≥ 2), which are not Manual requirements.

CHAPTER 10

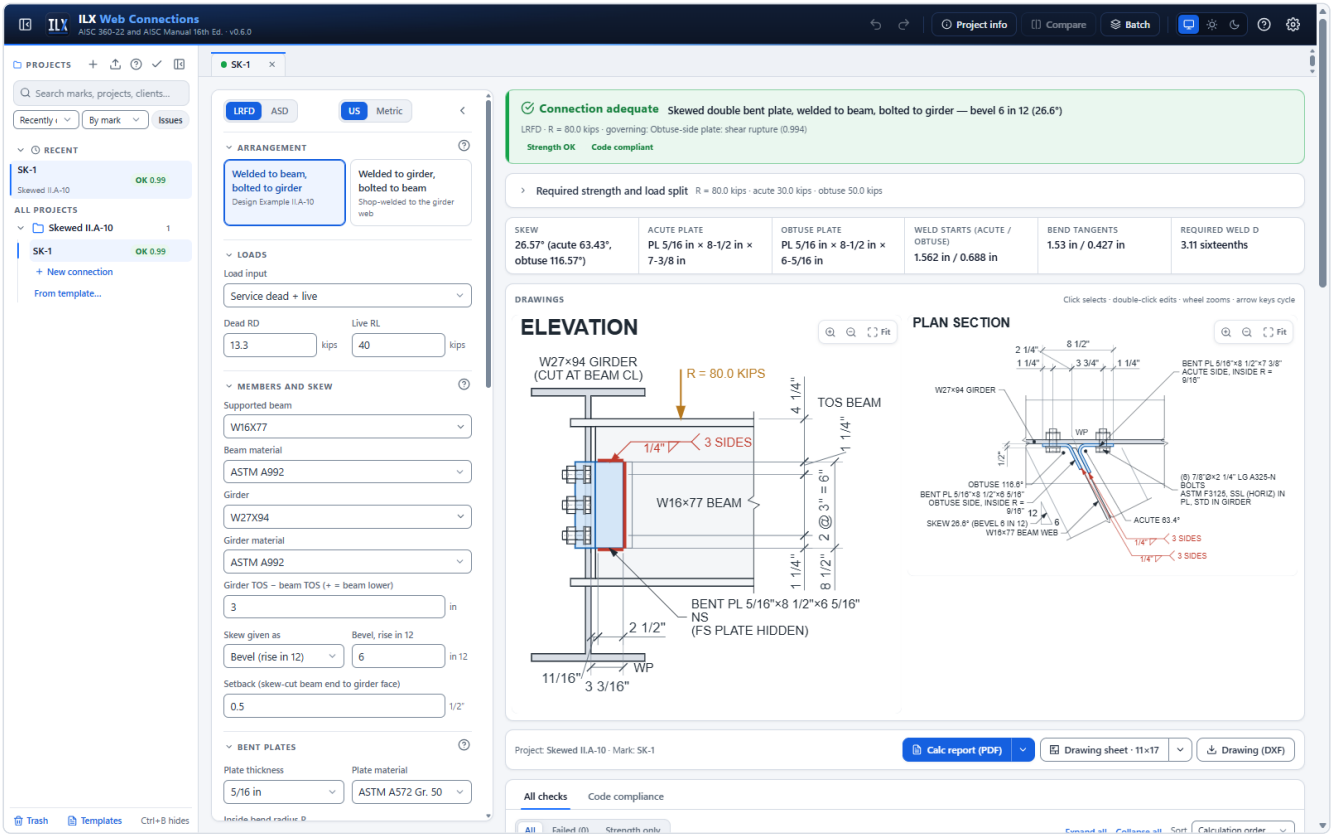
# Designing a skewed double bent-plate connection

A skewed beam frames into a girder web at a plan angle. Two bent plates, one on each face of the beam web, follow the skew. Each plate has two flat legs:

- a **support leg** lying flat on the girder web;
- a **beam leg** lying flat on the beam web.

The bend between the legs has an inside radius **R**.

ILX follows AISC Design Example II.A-10 (Design Examples v16.0, pp. IIA-115 to IIA-121). The defaults of a new skewed connection are that example. It verifies to within 1 % in LRFD and ASD; see Verification.




## The editor workflow

The editor works like the single-plate editor. Every change re-runs the design, and the status bar, drawings and checks update.

1. Choose **LRFD** or **ASD** and **US** or **Metric**.

2. Choose the arrangement.
3. Enter the loads.
4. Enter the members and the skew.
5. Set the plates, bolts and welds.
6. Fix whatever the status bar, the **GEOMETRY NOT FEASIBLE** banner or the checks report.
7. Create the calc report and the drawing sheet from the output bar.

To create one, choose **New connection** in a project. In the wizard, under **Connection type**, pick **Plate** in the **Simple shear** group, then **Skewed bent plate** under **Plate type**. The wizard asks for the beam, the girder, the bevel, the arrangement and the service loads. Everything else starts from II.A-10.

## Arrangement

| Arrangement | Shop | Field | Bolts | Welds |
|---|---|---|---|---|
| **Welded to beam, bolted to girder** (II.A-10) | plates welded to both faces of the beam web | support legs bolted to the girder web | one vertical line per plate, single shear | C-shaped fillets on the beam web |
| **Welded to girder, bolted to beam** | plates welded to the girder web | beam legs bolted through the beam web | one line through both plates and the web | C-shaped fillets on the girder web |

The second arrangement applies the II.A-10 procedure with the two members' roles exchanged. The split between the plates uses the weld-group centroids on the girder instead of the bolt lines. The welds are designed for the load at the work point. The beam web carries the full reaction in bearing.

**Bent plates** chooses **Two bent plates (II.A-10)**, the default, or **Single bent plate (Manual Fig. 10-37)**. For a single plate, **Face of the beam web** puts it on the obtuse face (the default) or the acute face. Both arrangements above work with one plate. See Single bent plate below.

## Members and skew

- **Supported beam** and **Girder**: W, M, S or HP shapes, with their materials.
- **Girder TOS − beam TOS (+ = beam lower)**. In II.A-10 the beam is 3 in. below the girder.
  - ILX checks that the plates sit in the girder's clear web, between the fillets.
  - It also checks that the beam flanges clear the girder flanges. If a flange enters the girder's flange zone, the connection is **NOT FEASIBLE**. ILX then suggests a cope and its required depth and length. The length is measured along the beam to clear the acute flange tip.
- **Skew given as**:
  - **Bevel (rise in 12)**, e.g. 6 in 12 = 26.57°;
  - **Angle (degrees)** from square framing, less than 75°.
- **Setback**: the clear gap from the skew-cut beam end to the girder face, measured perpendicular to the girder (the end is cut parallel to the girder web). It must be positive.

ILX names the two plates by the side of the beam they are on:

- **Acute side**: included angle 90° − φ. In II.A-10 this is bent plate A, 63.4°.
- **Obtuse side**: included angle 90° + φ. In II.A-10 this is bent plate B, 116.6°.

The **work point (WP)** is the beam centerline at the girder face. Bolt lines and weld starts are measured from it.

## Bent plates

- **Plate thickness** and **Plate material**. II.A-10 uses 5/16 in. A572 Gr. 50.
- **Inside bend radius R**. ILX checks R against the minimum inside radius for cold bending of ASTM A6 Appendix X4, Table X4.1, for the plate's steel group and thickness. That table is reproduced in the AISC Manual Part 10.
  - For example, 1.5t for A36 or A572 Gr. 50 up to 3/4 in. thick.
  - Tick **Bend line parallel to the final rolling direction** to apply the table's ×1.5.
  - A radius below the minimum makes the connection **NOT FEASIBLE**.
- **Top of beam to top of plates** and **Lev**. The plate depth is 2 Lev + (n − 1) s.
- **Cb, plate lateral-torsional buckling (F11)**: your input, default 1.84, for the flexure check at the bend (an ILX addition). Neither the Manual (skewed connections, pp. 10-88 and 10-89) nor Design Example II.A-10 gives a Cb. 1.84 is the single-plate value of Manual Eq. 10-8. The report shows "Cb = x (engineer input)".

The key-geometry strip shows:

- each plate's size: thickness × depth × developed length, measured at mid-thickness and rounded up to 1/16 in.;
- the weld starts and the bend tangents.

## Bent-plate bolts

- **Grade, diameter, threads, joint** (bearing or slip-critical) and **surface class**: all grades, thread conditions and joint types.
- **Bolts per line n**, **spacing s**, and **Leh**, the bolt line to the end of the bolted leg.
- The bolt line positions:
  - Welded to beam: **Acute bolt line from WP** and **Obtuse bolt line from WP**, both along the girder. II.A-10 uses 3 3/4 in. and 2 1/4 in.
  - Welded to girder: **Bolt line from WP, a**, along the beam.
- **Holes in plates** and **holes in the girder (or beam) web**. II.A-10 uses short slots, horizontal (SSL horizontal), in the plates and standard holes in the girder.
- **Bolt-line eccentricity**:
  - **None (load split by statics, II.A-10)**: each line takes its share of the reaction.
  - **From the work point**: a conservative option.
  - **Given**: a value you enter.
- **Bolt group strength**: (C/n) × Σ per-bolt, the "shear transfer at bolt holes" of the Design Examples, or C × the weakest bolt.

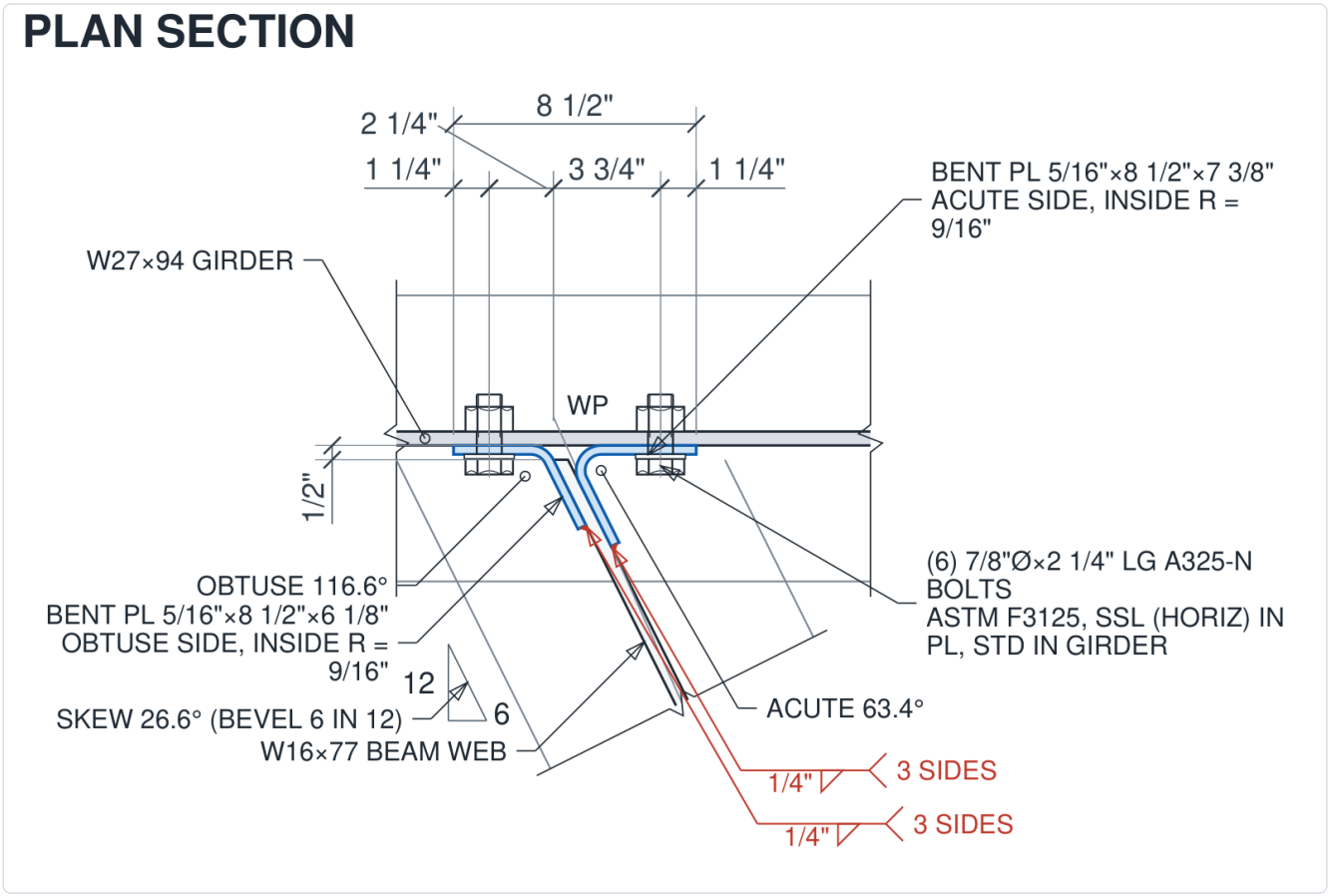
The plate is checked for the following, as II.A-10 does:

- the bolts, by the least of bolt shear, plate bearing or tearout, and girder-web bearing or tearout, at each bolt;
- plate shear yielding;
- shear rupture;
- block shear.

ILX adds two checks the example does not make, flagged in their titles:

- in-plane flexure of each plate at the bend;
- the Manual Eq. 10-8 shear and flexure interaction there.

## Bent-plate welds




- **Fillet size** and **electrode**.
- **Weld shape**: C-shape (3 sides), top and bottom only, or vertical at the toe only. **kl** is the horizontal weld length.
- **Weld group location**:
  - **Averaged, both plates (II.A-10)** places both weld groups at the average of the two weld starts. This follows the example: "the locations of the weld groups will be averaged and considered identical".
  - **Each plate its own** uses each plate's own location.
- **Acute / obtuse weld start from WP**. By default, **From the bend geometry** starts the welds where the beam leg leaves the bend (the tangent point) or at the beam end, whichever is farther, rounded up to 1/16 in. You can also enter a distance.
  - A weld that starts on the bend or past the beam end is **NOT FEASIBLE**.
  - For II.A-10's acute plate, the bend geometry gives 1.53 in.; the published value is 1 9/16 in.

Each weld group is solved by the instantaneous-center method (Manual Part 8) with the load at the work point. Table 8-8 is the example's equivalent. The required size D and the coefficient C are shown in the calc card. ILX also checks:

- the base metal: tmin = 6.19 D / Fu for plates on both faces of the beam web (Manual Eq. 9-7), or 3.09 D / Fu on one face of the girder web;
- minimum and maximum fillet size (Table J2.4; t − 1/16 in. along the plate edge);
- kl ≥ 4w;
- the lap length.

The bent-plate fillets lie on the flat legs, so they are lap fillets, not skewed T-joint fillets. The dihedral angles at the bends are labeled in the plan view for reference. A recommendation reports the acute opening angle, against the 30° welding-access guidance.

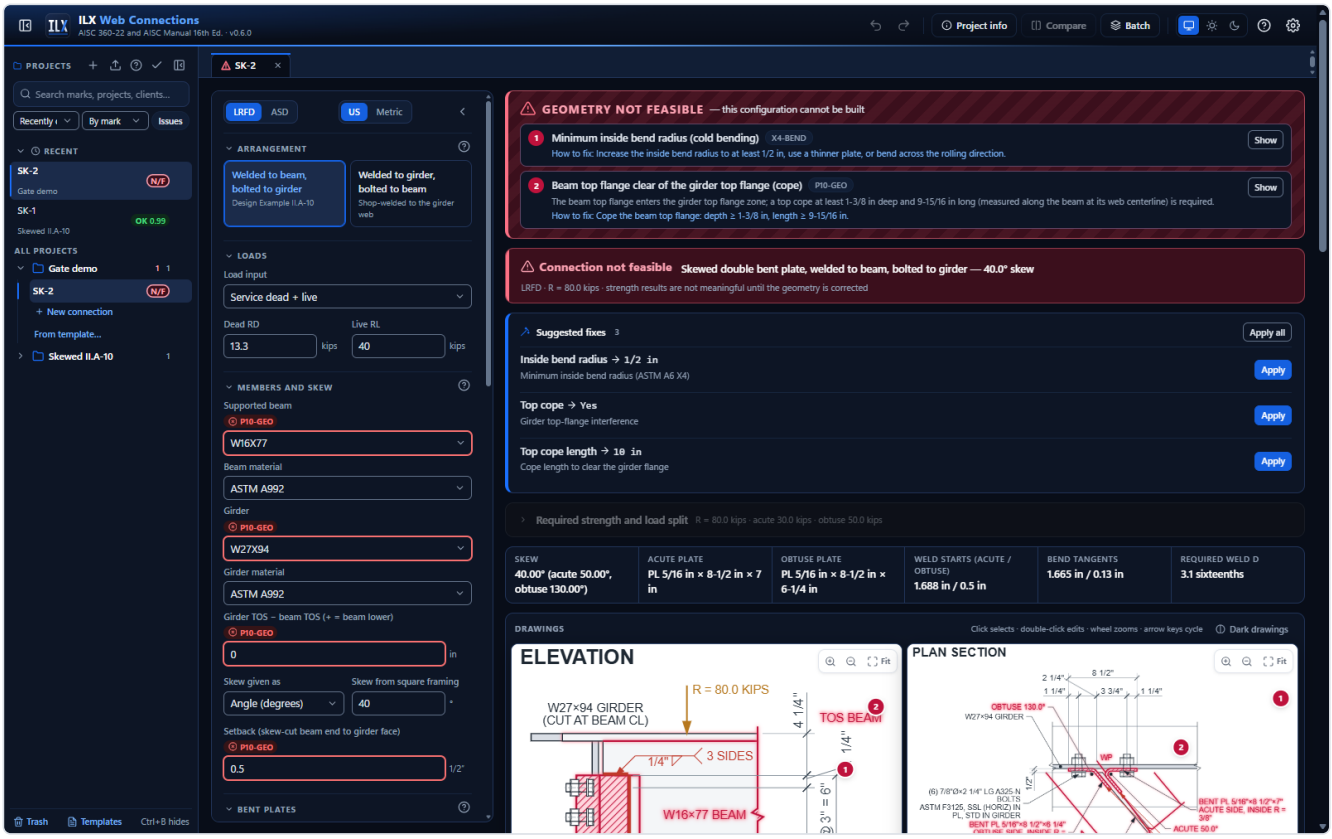
## Geometry gate

Any of the following marks the connection **NOT FEASIBLE**:

- the inside radius is below the ASTM A6 X4 minimum;
- the setback is zero or negative;
- a weld starts on the bend or past the beam end;
- a bolt head or washer would sit on the bend;
- the plates run into the beam or girder flanges;
- a beam flange clashes with a girder flange without a cope.

**Tightening clearance** checks the distance from each bolt to the other leg of the same plate against the impact-wrench clearance C1. On the acute side it also checks the beam top flange. A shortfall is NG (detailing), not NOT FEASIBLE: the bolt can still be tightened from the other side.

A skew over 45° raises a recommendation, because the Manual's skewed details stop at 45°.




## Single bent plate

As the skew grows, the bolts on the acute side need a larger gage on the support to leave room to enter and tighten them. The AISC Manual (16th Ed., Part 10, pp. 10-88 and 10-89) says that if that gage becomes objectionable, "a single bent plate, illustrated in Figure 10-37, may provide a better solution". Choose **Single bent plate** under **Bent plates**, then the face of the beam web it sits on.

The Manual's rules, and what ILX does with each:

- **Load at the work point.** "The load is assumed to be applied at the point where the skewed beam centerline intersects the face of the supporting member." The one plate carries the full reaction R; there is no load split.
- **Bolts.** "Both the shop and field bolts should be designed for shear and the eccentric moment." The bolt group is always solved by the instantaneous-center method (Manual Part 7) with e = WP to the bolt line: along the girder when bolted to the girder, along the beam (a) when bolted to the beam. The **Bolt-line eccentricity** option does not apply and is hidden. Each bolt is the least of bolt shear (single shear), plate bearing or tearout, and girder-web or beam-web bearing or tearout.
- **Welds.** Each weld group is solved by the instantaneous-center method (Manual Part 8) at its own location from the work point. **Weld group location** does not apply and is hidden. "A C-shaped weld is preferable to avoid turning the beam during shop fabrication": another weld shape raises a recommendation.
- **Base metal.** With the plate welded to one face of the beam web, tmin = 3.09 D / Fu (Manual Eq. 9-6). Welded to the girder, the girder-web rule is unchanged.
- **Plate.** Shear yielding, shear rupture and block shear are checked for the full R.

- **Flexure.** "Single bent plates should be checked for flexural strength." The Manual does not say where or how. Because each bolt and weld group is designed for a load through the work point, ILX takes the in-plane moment at a section as R × the distance from the work point. It checks:
  - flexural yielding (Spec F11) with e = the larger of WP to the bolt line and WP to the weld start;
  - flexural rupture through the bolt holes, Fu Znet, with e = WP to the bolt line;
  - the Manual Eq. 10-8 shear and flexure interaction.

  This is how the Manual checks a conventional single-plate connection at its bolt line. The calc steps say that the section and load path are ILX's choice.
- **Geometry.** The bend radius, weld-on-the-flat, bolt-head-on-the-flat, tightening clearance, beam clearance and flange checks apply to the one plate. Only that side's bolt-line and weld-start inputs are shown.
- **Range of skews.** Table 10-14B recommends single bent plates for skews from 4 in 12 to 12 in 8 (18.4° to 56.3°). Outside that range ILX raises a recommendation, not a failure.

The plan view draws the one plate. The elevation draws it as the near plate, seen from its own face of the beam web. The report and the status bar title the connection "Skewed single bent plate".

What the single-plate option does not cover:

- the **all-bolted** single bent plate (Fig. 10-37(a), Table 10-14A, with its D and H dimensions by bevel and 1 in. maximum bolts);
- the **wrapped** configuration of Fig. 10-37(c). Only the conventional configuration is designed;
- the small out-of-plane offset between the work point and the plate on the beam web (half the web plus the plate). The Manual gives no rule for it, and a recommendation says it is not included.

## Copes

Tick **Top cope** or **Bottom cope** and enter c and dc; the girder-flange check proposes the values. ILX then checks:

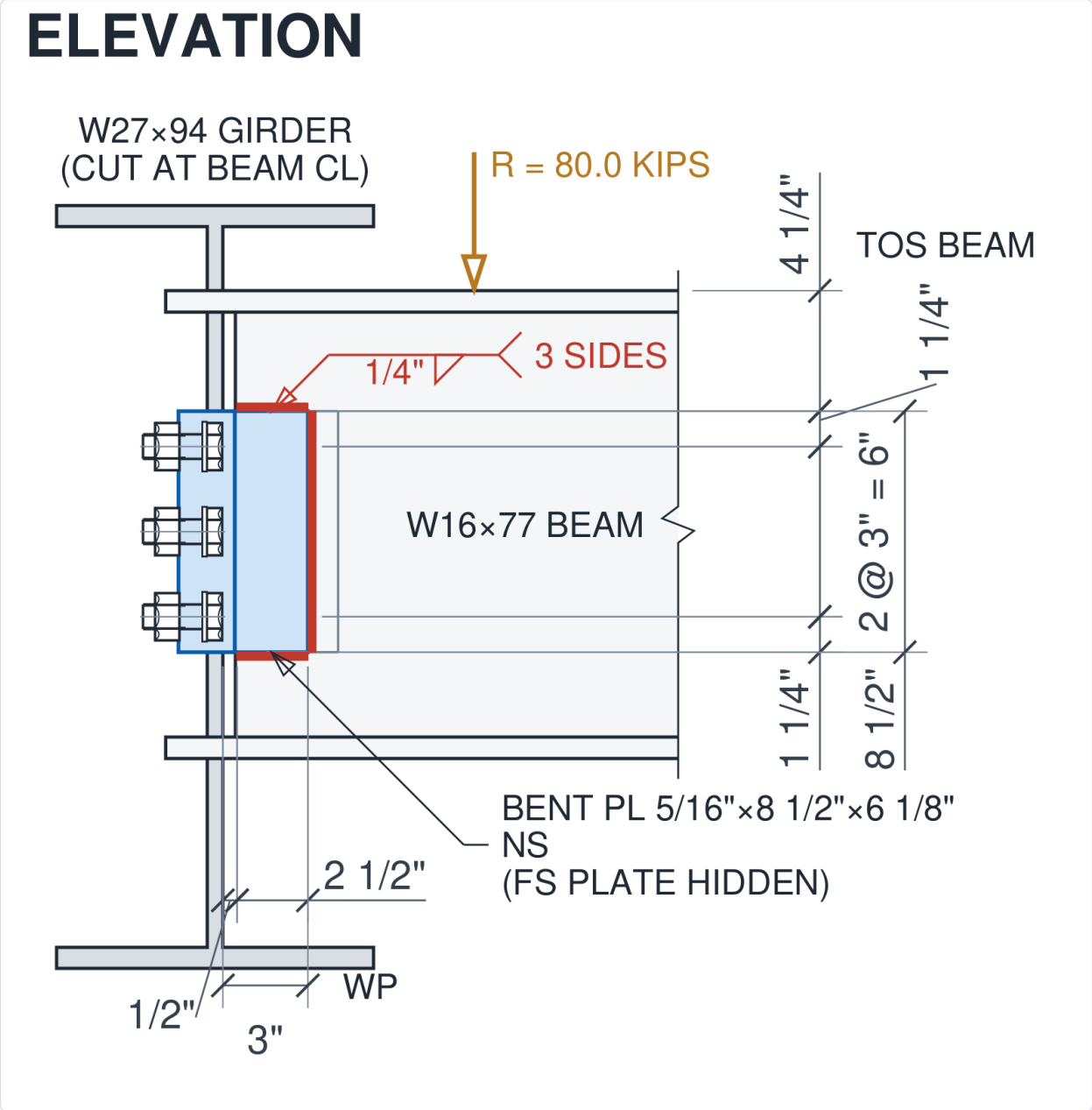
- the coped web in shear;
- the single-cope local flexure (Manual Part 9, Dowswell 2018), with e measured from the work point to the cope face;
- the reentrant radius (AISC 360-22 M2.2).

A double cope is accepted, but its local flexure is not checked here; a recommendation says so.

## Drawings and outputs

- **Plan section**: the true skew, with:
  - the girder web, with its bottom flange hidden;
  - the skew-cut beam web;
  - both bent plates, with their bends;
  - the bolts, drawn realistically through the support legs;
  - the toe fillets in section, with their AWS A2.4 symbols;
  - the acute and obtuse included angles, the bevel triangle, and the dimensions along the girder.
- **Elevation**, looking normal to the beam web from the obtuse side:
  - the girder cut at the beam centerline;
  - the near plate, with its weld strips and side-view bolts;

- the far plate, hidden;
- the dimensions from the work point and from the top of beam.




Click an element to select it and double-click to edit it in place, as in the single-plate editor. The calc report and the drawing sheet (PDF and DXF, at every sheet size) come from the output bar.

## Limits

- One vertical bolt line per plate.
- Shear only: no axial load and no load combinations beyond D + L or a given required strength.
- No auto-design.
- Skewed single plates and skewed end plates are not part of this type. Their Manual Part 10 text (gap, bevel and the Table 10-14C weld details) is documented in research/24 for a later version.

CHAPTER 11

# Designing an FR moment connection

A fully restrained (FR) moment connection transfers the beam end moment to the column through the beam flanges, and the shear through the web. ILX designs the three FR connections of AISC Manual Part 11 to AISC 360-22, with the column checks of J10 and the stiffeners and doublers they require.

## When to use an FR connection

Use it for a beam framing into a column flange where the joint must carry moment: wind or gravity frames, R = 3 systems designed to AISC 360 only, and seismic moment frames (OMF, IMF, SMF) designed to AISC 341-22 with the AISC 358-22 prequalified connections (see Seismic moment frames).

- **Bolted flange plates**: plates shop-welded to the column, field-bolted to the beam flanges (Design Example II.B-1).
- **Welded flange plates**: plates shop-welded to the column, field fillet-welded to the beam flanges (II.B-2).
- **Directly welded**: beam flanges CJP groove-welded to the column flange (II.B-3).

For an IMF or SMF, the directly welded connection is designed as the reduced beam section (RBS, 358-22 Chapter 5) or the welded unreinforced flange-welded web (WUF-W, Chapter 8), and the bolted flange plates as the bolted flange plate (BFP, Chapter 7). Welded flange plates have no prequalified connection and stay flagged as not adequate. For a simple shear connection, use Designing a single-plate connection.

## Creating an FR connection in the wizard

The wizard has three steps: **Connection**, **Members** and **Loads**.

1. **Connection.** Under **Connection type**, in the **Moment** group, select **FR moment**. There is no joint-type choice: the beam frames into the column flange.
2. Under **FR moment connection (Manual Part 11)**, select one of the three cards. Each sets the **Connection** field of the editor:

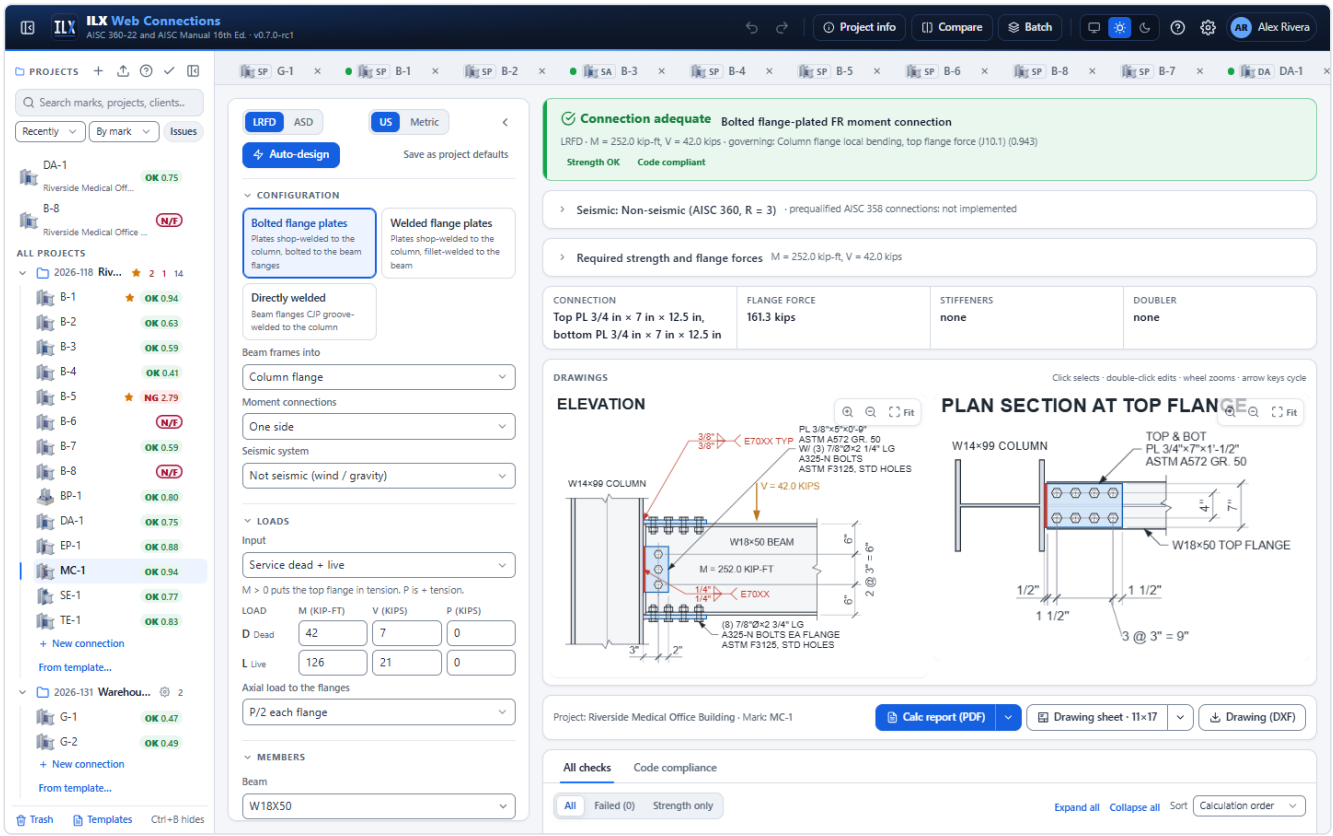
| Card | Sets | Description |
|---|---|---|
| **Bolted flange plates** | Bolted flange plates | Plates welded to the column, bolted to the beam flanges (II.B-1) |
| **Welded flange plates** | Welded flange plates | Plates welded to the column and fillet-welded to the beam (II.B-2) |
| **Directly welded flanges** | Directly welded | CJP groove-welded beam flanges, bolted shear tab (II.B-3) |

1. **Members.** Enter the **Connection mark** (MC-1, MC-2 and so on by default), the **Beam** and the **Column**. Both are W shapes.
2. **Loads.** Enter the service moments **Dead MD** and **Live ML** (kip-ft) and the service shears **Dead VD** and **Live VL** (kips). The wizard notes that *Column stiffeners and doublers are designed automatically when required (AISC 360 J10).* Use the editor for the other load modes, axial load, the opposite beam and the column forces.

3. Optional: tick **Auto-design the connection (flange plates, bolts, welds) for these loads**. ILX sizes the hardware before it creates the connection, keeping the beam, column and connection kind (see Auto-design, batch and compare). A notification gives the result: adequate or best effort, with the ratio.
4. Select **Create**.

The wizard always creates the connection in US units. Switch to **Metric** in the editor toolbar afterwards if needed.

## Inputs by section




The toolbar has **Design method** (**LRFD** or **ASD**), **Units** (**US** or **Metric**), **Auto-design** and **Save as project defaults**. The last one saves this type's materials, hardware and methods as project defaults; members and loads are not saved.

Above the drawings, the key data strip shows the **Connection** (the plates or CJP flanges), the **Flange force**, the **Stiffeners** and the **Doubler**. Open **Required strength and flange forces** to see the combination and the flange forces worked out.

### FR configuration and scope

| Field | Options and notes |
|---|---|
| Configuration buttons | **Bolted flange plates**, **Welded flange plates**, **Directly welded** |
| **Beam frames into** | **Column flange**, or **Column web (advisory only)**. The column-web case is not designed: it adds a failing scope check and the beam-side checks assume a column flange. |

| Field | Options and notes |
|---|---|
| **Moment connections** | **One side**, or **Both column flanges** (adds the opposite beam and its moment M2) |
| **Seismic system** | **Not seismic (wind / gravity)**, **R = 3 (AISC 360 only)**, **OMF (AISC 341-22 E1)**, **IMF (AISC 341-22 E2, AISC 358-22)**, **SMF (AISC 341-22 E3, AISC 358-22)** |

For OMF, IMF and SMF a **Seismic (AISC 341-22 / 358-22)** section opens below the configuration (see Seismic moment frames), and a **Seismic** card above the drawings shows the design basis (Mpr, Sh, Vh, Mf, Vu, the protected zone) and the status of each prequalified connection. With welded flange plates in an IMF or SMF the editor warns that the configuration is not a prequalified connection.

## Moment, shear and axial loads

Choose the load mode with **Input**. M > 0 puts the top flange in tension. P is positive in tension.

| Input | Enter | Combinations used |
|---|---|---|
| **Service dead + live** (default) | M, V and P for D and L | LRFD: 1.2D + 1.6L, or 1.4D when it governs. ASD: D + L. |
| **Service components → ASCE 7 combinations** | M, V and P for D, L, Lr, S, W and E | Every ASCE 7-22 LRFD (2.3) or ASD (2.4) combination, W and E with both signs; the envelope governs |
| **Required strength (Mu, Vu, Pu)** | **Mu**, **Vu**, **Pu (+T)** (Ma, Va, Pa in ASD) | None (used as entered) |
| **Load cases (table)** | Named cases with M, V and P; **Add case** adds a row | Each case is checked; the envelope governs |

- Moments are in kip-ft (kN-m), forces in kips (kN). V cannot be negative, except for W and E.
- With **Both column flanges**, an **M2** column appears: the opposite beam's moment, + when its top flange is in tension.
- **Axial load to the flanges**: **P/2 each flange** or **In proportion to area**. With a welded web, P/2 is always used.
- Because W and E act with both signs, a reversing moment designs both flanges for tension and compression. The **Load combinations** table lists every combination (see Load combinations in the results).

## Beam and column

| Field | Notes |
|---|---|
| **Beam**, **Beam material** | W, M, S or HP shape; ASTM A992 and so on |
| **Opposite beam** | Both column flanges only. It uses the same connection type and plate sizes. |
| **Column**, **Column material** | W, M, S or HP shape |
| **Beam setback** | Column face to the beam end. For directly welded flanges it is also the CJP root opening on the drawing. |

## Flange plates or flange welds

The section is titled **Flange plates**, or **Flange welds** for directly welded flanges.

| Field | Shows for | Notes |
|---|---|---|
| **Top plate t**, **Top plate width**, **Bottom plate t**, **Bottom plate width** | Flange plates | 1/4 in. to 2 in. |
| **Top plate length**, **Bottom plate length** | Welded flange plates | Bolted plates take their length from the bolts: af + (n − 1) s + end distance |
| **Flange plate material** | Flange plates | |
| **Plate to column** | Flange plates | **Fillet, both faces** or **CJP groove weld** |
| **Top plate fillet**, **Bottom plate fillet** | Fillet to column | Fillet on each face of the plate |
| **Bolts per row**, **Pitch** | Bolted | Two rows of 1 to 12 bolts each |
| **Gage (2 rows)**, **Column face to 1st bolt** | Bolted | af is measured from the column face |
| **End distance (plate end)**, **Beam underrun tolerance** | Bolted (underrun also welded) | The underrun reduces the beam flange end distance and lengthens the unbraced plate |
| **Holes in plates**, **Holes in beam flanges** | Bolted | **STD**, **OVS**, SSL or LSL **along the beam (‖ force)** or **across the flange (⊥ force)** |
| **Shim / filler** | Bolted | Fillers over 1/4 in. reduce bolt shear (J5.2) |
| **Top plate to beam fillet**, **Bottom plate to beam fillet** | Welded flange plates | |
| **Top weld length / side (0 = max)**, **Bottom weld length / side (0 = max)** | Welded flange plates | 0 uses all the length available on the plate |
| **Beam end to start of weld** | Welded flange plates | Part of the unbraced plate length in compression |
| **Transverse weld at the end of a narrower plate** | Welded flange plates | Adds the end weld when the plate is narrower than the beam flange |
| **K, plate in compression** | Flange plates | Default 0.65 |
| **CJP backing** | Directly welded | **Left in place** or **Removed + reinforcing fillet** |
| **Reinforcing fillet** | Backing removed | |
| **Access hole length**, **Access hole height**, **Access hole radius** | Directly welded | The J1.6 weld access hole in the beam web |

## Flange and web bolts

The **Bolts** section appears when there are bolts: bolted flange plates, or a web plate. The same bolts are used in the flange plates and the web plate.

| Field | Options |
|---|---|
| **Bolt grade** | The bolt grades of the bolt library |
| **Bolt diameter** | The sizes allowed for the grade |
| **Threads** | **Included (N)** or **Excluded (X)** |
| **Joint** | **Bearing (snug / pretensioned)** or **Slip-critical** |
| **Surface** (slip-critical) | **Class A (μ = 0.30)** or **Class B (μ = 0.50)** |
| **Hole deformation is a design consideration (2.4 / 1.2)** | Cleared: 3.0 and 1.5 (J3.11) |

### Web shear connection

| Field | Shows for | Notes |
|---|---|---|
| **Web connection** | Always | **Single plate bolted to the web** or **Beam web welded to the column** |
| **Web plate design** | Web plate | **Direct shear (flanges absorb eccentricity, II.B-1)** (default) or **Single-plate procedure (Part 10)** |
| **Web plate t**, **Web plate material** | Web plate | |
| **Bolts, n**, **Spacing, s** | Web plate | 2 to 20 bolts in one vertical row |
| **Lev, plate**, **Top of beam to plate (0 = centred)** | Web plate | |
| **Weld line to bolts, a**, **Leh, plate** | Web plate | |
| **Plate holes**, **Beam web holes** | Web plate | STD, OVS, SSL, LSL |

**Direct shear** follows Design Example II.B-1: the flange connections absorb the eccentricity, so the web bolts and weld are checked concentrically and the Part 10 simple-connection rules are dropped. **Single-plate procedure (Part 10)** runs the full single-plate design instead.

### Welds to the column flange

| Field | Shows for | Notes |
|---|---|---|
| **Web plate fillet (both sides)** | Web plate | Plate to column flange |
| **Web weld** | Welded web | **Fillet, both sides** or **CJP** |
| **Web fillet** | Welded web, fillet | |
| **Electrode** | Always | E70XX or E80XX |

### Column stiffening (J10)

The **Column (J10)** section holds the column forces and the stiffener and doubler options.

| Field | Notes |
|---|---|
| **Column axial Pu (compression +)** | Required column axial strength (Pa in ASD), for α Pr / Py in J10.6 |
| **Column story shear Vu** | Reduces the panel-zone shear |
| **Column ends at this connection (roof)** | Applies the end-of-member reductions of J10.1 to J10.5 |
| **Column top to top of beam** | Shows when the column ends here. Each check states the distance to the flange force and the equation it selects: J10.1 halved within 10tf, Eq. J10-3 within d, Eqs. J10-5a/b within d/2 |
| **Analysis includes inelastic panel-zone deformation (J10.6(b))** | Uses J10.6(b) in place of J10.6(a) |

| Field | Notes |
|---|---|
| **Transverse stiffeners** | **Design when required** (default), **Provided (as entered)** or **None** |
| **Stiffener t** | Provided only; otherwise designed |
| **Stiffener width (0 = max)**, **Stiffener length** | **Full depth** or **Half depth** |
| **Corner clip (min.)**, **Stiffener fillets (min.)**, **Stiffener material** | With **Design when required**, the clips and fillets are increased as needed |
| **Compression stiffeners finished to bear** | Fitted bearing, minimum welds to the flange |
| **Web doubler** | **Design when required** (default), **Provided (as entered)** or **None** |
| **Doubler t** | Provided only; otherwise designed |
| **Doublers** | **One side of the web** or **Both sides** |
| **Doubler to flanges**, **Doubler fillet (min.)** | **Fillet** or **CJP** |
| **Doubler extension past the flange forces**, **Doubler material** | |

With **Design when required**, ILX adds stiffeners when any of J10.1 to J10.5 fails and a doubler when J10.6 fails, and proposes the designed sizes as suggested fixes.

## Seismic moment frames (AISC 341-22 / 358-22)

ILX follows AISC 341-22 Chapter E and AISC 358-22 as written. Where they leave a value or a choice to you, it is an input with a stated default, and the calculation report states it. You are the engineer of record.

### Which connection is designed

| Seismic system | Connection | Design |
|---|---|---|
| IMF, SMF | **Directly welded**, **Welded flange connection (AISC 358-22)** = **Reduced beam section, RBS (Chapter 5)** | 358-22 Chapter 5 |
| IMF, SMF | **Directly welded**, **WUF-W (Chapter 8)** | 358-22 Chapter 8 |
| IMF, SMF | **Bolted flange plates** | Bolted flange plate (BFP), 358-22 Chapter 7 |
| IMF, SMF | **Welded flange plates** | Not prequalified: a failing scope check |
| OMF | Any, with **OMF connection (AISC 341-22 E1.6b)** | (a) required flexure 1.1 Ry Mp / αs and shear from Ecl = 2(1.1 Ry Mp)/Lcf; (b) the maximum moment and shear the system can deliver, entered; (c) the prescriptive welded details (directly welded flanges only); or an AISC 358 connection as for an IMF |

The AISC 358-22 procedures are LRFD whatever the design method (358-22 1.3); the column checks at the probable flange force use Pf / αs in ASD. OMF (a) and (b) work at the design-method level (αs = 1.5 in ASD).

**Inputs** (section **Seismic (AISC 341-22 / 358-22)**)

| Field | Default | Notes |
|---|---|---|
| **OMF connection (AISC 341-22 E1.6b)** | (a) | OMF only |
| **Welded flange connection (AISC 358-22)** | RBS | Directly welded flanges in an IMF / SMF |
| **Clear span between column faces, Lcf** | 360 in. | Span-to-depth limit, Lh = Lcf − 2 Sh, and Ecl for an OMF |
| **Gravity line load wu (1.2D + f1L + 0.15S)** | 1.25 kip/ft | Factored, on the beam between the hinges (358-22 2.4.4); f1 not less than 0.5 |
| **Further gravity shear at the hinge** | 0 | Concentrated loads, same combination |
| **Cpr (0 = AISC 358-22 value)** | 0 | (Fy + Fu)/(2Fy) ≤ 1.2 (Eq. 2.4-2); 1.4 for WUF-W (8.6). A value entered overrides the standard |
| **Moment connection at this end only (far end pinned)** | off | Vh = Mpr / (Lcf − Sh) + Vgravity (2.4.4 User Note) |
| **Maximum moment by the system**, **Corresponding shear** | 0 | OMF (b) |
| **Beam Ry**, **Beam Rt** | 0 | 0 = AISC 341-22 Table A3.2 (A992 and A572 Gr. 50 shapes 1.1 / 1.1, A36 1.5 / 1.2) |
| **RBS a**, **RBS b**, **RBS c** | 0 (auto) | Auto: a = 0.625 bf and b = 0.75 d to 1/4 in., c the smallest 1/8 in. from 0.1 bf with Mf ≤ φd Mpe; the values are offered as suggested fixes so you can keep them |
| **Concrete structural slab, shear studs at 12 in. max.** | on | The chapter's exception to supplemental bracing at the hinge; BFP column depth W36 (W14 without a slab) |
| **Slab isolated from the column (358-22 2.3.4)** | off | BFP in an SMF with a slab |
| **A moment-ratio exception of AISC 341-22 E3.4a applies** | off | Eq. E3-1 becomes information |

With the WUF-W, and the RBS in an SMF, the web connection is the one 358-22 prescribes and the **Web connection** section shows its single plate: **Single plate t** (at least tw for WUF-W, 3/8 in. for RBS), **Single plate width (0 = auto)** (setback + access hole + 2 in.), **Plate overlap with each access hole** (1/4 to 1/2 in., WUF-W) and **Single plate to column** (fillets or CJP). An IMF RBS may keep the bolted single plate (5.5(2) Exception). The CJP backing has a third option, **Bottom removed, top left (358-22 3.3)**.

### What is checked

- Capacity design (358-22 2.4, 2.5): Mpr = Cpr Ry Fy Ze, Sh, Lh, Vh = 2 Mpr / Lh + Vgravity, Mf = Mpr + Vh Sh, Vu = Vh + wu Sh, shown under **Required strength and flange forces** and in the report.
- Prequalification limits of the chapter (beam depth, weight, flange thickness, clear span-to-depth ratio, column depth, rolled W shapes) and the width-to-thickness ratios of Table D1.1b (highly ductile for an SMF, moderately ductile for an IMF; RBS flange width at the ends of the centre two-thirds).
- RBS: a, b, c within Eqs. 5.7-1 to 5.7-3, ZRBS (Eq. 5.7-4), Mf ≤ φd Ry Fy Zb (Eq. 5.7-7), the reduced section for the load combinations, the web (CJP or the IMF bolted plate with its slip-critical, J3-6a, holes and 5/8 tp fillet rules).
- WUF-W: Cpr = 1.4, Sh = 0, the single plate (thickness, overlap, width, welds for 0.6 Ry Fyp hp tp, fillet tp − 1/16), the AWS D1.8 access hole (length 3 tbf ± 1/2 in., height 3/4 tbf to tbf, radius 3/8 in.).

- BFP: plates, bolts and holes of 7.4, Eq. 7.5-1 (net section), the bolts (Eq. 7.5-7), plate yielding, rupture, block shear (plate and beam flange) and buckling (Eqs. 7.5-8 to 7.5-11, KL = 0.65 S1 by default through **K, plate in compression**), lc ≥ 2 db, and the web plate for Vu.
- Beam shear at Vu (G2.1), the CJP web at Vu, 358 Chapter 3 backing and reinforcing fillet, demand-critical welds.
- Continuity plates (AISC 341-22 E3.6f): required when the column J10 limit states fail at Pf = Mf / (αs d*) (0.85 Mf with a welded web) or tcf < bbf/6 (Eq. E3-7); then their thickness, b/t (Eq. E3-9), width, welds and the 358-22 3.6 corner clips. With **Design when required** the plates are sized to meet them.
- SMF panel zone (E3.6e): Σ Mf / db − Vc with φv = 1.00, the doubler, Eq. E3-6 for the web and the doubler, the doubler welds (groove welds in practice) and extension. IMF and OMF use the J10.6 check with the load combinations (E2.6e, E1 User Notes).
- SMF column-beam moment ratio (Eq. E3-1), column bracing (E3.4c), beam bracing and the protected zone, as notes with their values.

Not built: the extended end plates of 358-22 Chapter 6 (BUEEP / BSEEP, the next step with the end-plate moment connection), the double tee (Chapter 13) and the slotted web (Chapter 14).

## Checks the FR connection runs

Every check is run for every load combination and the governing one is reported. See Checks and compliance for reading the results.

**Flange force.** Puf = M / dm + Pf (Manual Eq. 11-1). dm is taken per element, as in the Design Examples: the beam depth d for the bolts and the plate-to-beam welds, d + tp,top/2 + tp,bot/2 for the plates, the plate-to-column welds and the column, and d − tf for directly welded flanges.

**Beam.**

- Flexural yielding at the connection, Mp (AISC 360-22 F2.1).
- Tension flange rupture at the bolt holes (AISC 360-22 F13.1), bolted flange plates.

**Bolted flange plates.**

- Flange bolts: shear, bearing and tearout in the plate and the beam flange, summed per bolt (AISC 360-22 J3.7, Table J3.2; AISC 360-22 J3.11), with the long-joint and J5.2 filler reductions. Slip resistance (AISC 360-22 J3.9) when slip-critical.
- Flange plate: tensile yielding (J4.1(a)), tensile rupture (J4.1(b)), block shear rupture (J4.3, three failure paths), compression (J4.4) with Lc = K max(af, s).
- Beam flange block shear rupture (J4.3).
- Detailing: pitch and gage ≥ 2⅔d (J3.4, 3d preferred), edge and end distances (J3.5, Table J3.4), maximum spacing and edge distance (J3.6), tightening clearance and bolt seat clear of the web fillet (Manual Part 7), slip-critical bolts required for OVS holes or slots parallel to the force (J3.3), a pretension recommendation for bearing bolts and a shim note (Manual Part 11).

**Welded flange plates.**

- Flange plate tensile yielding (J4.1(a)) and compression (J4.4), with L = setback + underrun + start of weld.
- Plate-to-beam fillets (AISC 360-22 J2.4(c), Eq. J2-6), with the size limits (J2.2b, Table J2.4), l ≥ 4w, l ≥ the distance between longitudinal welds, and the end-loaded β note.
- Base metal ≥ 3.09D/Fu (Manual Part 9) and beam flange shear rupture at the welds (J4.2(b)).

**Plate to column.** Fillets on both faces (AISC 360-22 J2.4, Table J2.5) with minimum and practical maximum sizes, or a CJP groove weld that develops the plate.

**Directly welded flanges.**

- CJP groove weld, base metal governs: Fy bf tf (AISC 360-22 J2.4, Table J2.5).
- Matching filler metal (recommendation), beam flange within the column flange width, backing note.
- Weld access holes (AISC 360-22 J1.6): length ≥ 1.5 tw and ≥ 1½ in., height ≥ tw and ≥ ¾ in. (need not exceed 2 in.), radius ≥ ⅜ in., finish note.

**Web connection.**

- Web plate, direct shear: bolts (J3.7, J3.11) and the fillet welds to the column (J2.4), concentric, plus the single-plate engine's plate, beam web and column flange checks, shown as **Web plate: …** and **Web connection: …**.
- Web plate, Part 10: the full single-plate design.
- Welded web: fillet welds both sides (J2.4), web base metal ≥ 6.19D/Fu (Manual Part 9), minimum size, or a CJP note; beam web shear yielding along the weld (J4.2(a)).

**Column (J10).** For each flange force, and for the opposite beam's:

- Flange local bending (AISC 360-22 J10.1), tension forces; not required when the loaded width is < 0.15 bf.
- Web local yielding (J10.2).
- Web crippling (J10.3), compression forces.
- Web compression buckling (J10.5), only with beams on both flanges in compression at the same level.
- Web panel-zone shear (J10.6), Vr = ΣPuf − Vc, by J10.6(a) or (b).

**Stiffeners (AISC 360-22 J10.8).** Width + tw/2 ≥ b/3, thickness ≥ t/2 of the loading plate or flange and ≥ width/16, length ≥ ½ depth (¾ for crippling, full depth for J10.5), tension or compression strength (compression as a column with a 25tw web strip, Lc = 0.75h), flange and web welds, column web shear rupture along the welds, and corner clips that clear the column fillets (Manual Part 4 / AISC Design Guide 13).

**Doublers (AISC 360-22 J10.9).** Panel-zone shear with the doubler (t = tw + Σtd, Fy = the lower of column and doubler), doubler-to-flange welds for the doubler's share of the shear, fillet ≤ td − 1/16 or CJP, and recommendations from Manual Part 4 / Design Guide 13: thickness ≥ (dz + wz)/90, extension ≥ 6 in. past the flange forces without stiffeners, k-region clearance.

**NOT FEASIBLE** is reported when the geometry cannot be built: the first flange bolt off the beam, bolt rows that do not fit the plate or flange, a plate or beam flange wider than the column flange, no shelf for a fillet, a weld longer than the plate, an access-hole radius larger than its height, or a stiffener wider than the column flange outstand.

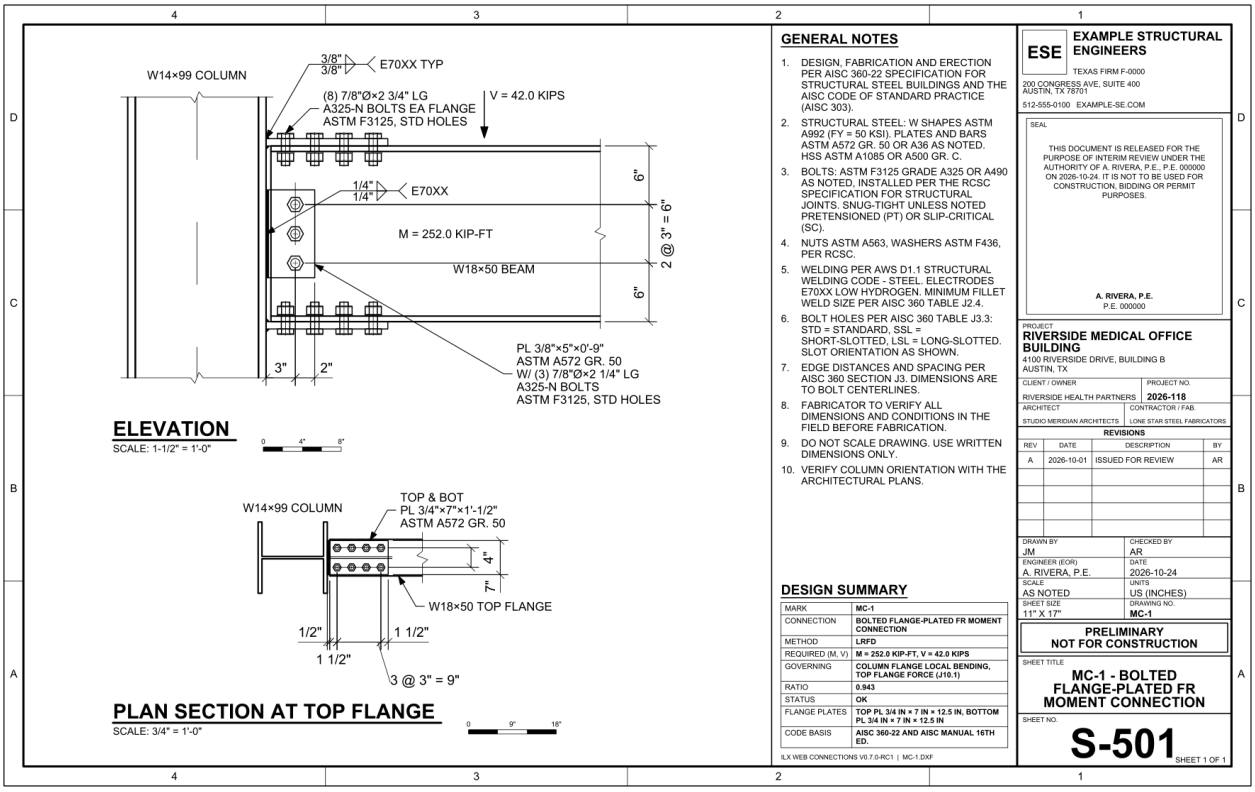
**Fatigue (Appendix 3).** The **Cyclic loading (fatigue)** section designs the connection for fatigue: the cycle count nSR and the cyclic range as a fraction of the live moment ML or as the moment range entered (see Fatigue (Appendix 3)). The flange force range is the flange force at the governing moment times the range ratio. The details: directly welded flanges, the CJP weld to the column (Table A-3.1 Case 5.4, C; Eq. A-3-1 applies only to CJP welds tested per Section 3.6, an engineer setting, see Settings) and the beam web at the weld access holes (Case 1.4: C for R ≥ 1 in. ground, else E′); flange plates, the plate at its fillets to the column (Case 5.7, C and C″ with RFIL) and those fillets (Case 8.2, F), or the CJP weld (Case 5.4); bolted plates, the plate and the beam flange at the bolt holes (Section 2, or Case 1.5 for snug-tight joints); welded plates, the plate and the beam flange at the ends of the longitudinal welds (Case 4.1, E for t ≤ 1/2 in., E′ above) and the plate-to-beam fillets (Case 8.2); the web shear tab as a single plate.

## FR drawings, report and sheet

The editor draws two views:

- **ELEVATION**: the column in side view with the stiffeners at the flange force lines and the doubler face-on; the beam with its flange plates (flange bolts in side view, or plate-to-beam fillets) or CJP flange welds with backing bars or reinforcing fillets and the access holes; the shear tab with its bolts or the welded web; dimension chains, V and M.
- **PLAN SECTION AT TOP FLANGE**: the column section cut at the top flange, with the stiffeners, corner clips and doubler, the beam top flange, the flange plate and its bolt pattern.

In a seismic design the elevation hatches the protected zone on both beam flanges and notes its length from the column face, the plan shows the RBS radius cut with a and b in the dimension chain and a callout of c and R, and the elevation draws the WUF-W single plate (1 in. straight at the column, 30° slopes) or the RBS backing plate over the CJP web weld. The access-hole note cites AWS D1.8 where the connection requires its geometry, and a bottom-only backing removal gives one weld symbol per flange (**R** at the bottom, **B** at the top).




The sheet carries the same two views and the shared sheet content (see Drawing sheets). Its weld symbols follow AWS A2.4:

- CJP beam flanges: single-bevel groove symbol with the joint designation **TC-U4a** in the tail, backing **B** (left in place) or **R** (removed, with the reinforcing fillet), and the field-weld flag.
- Flange plates to the column: fillets on both sides, or a single-bevel CJP with backing.
- Flange plates to the beam: field fillets marked **3 SIDES** with the transverse end weld, or **BOTH SIDES**.
- The shear tab, the welded web, the stiffeners and the doublers.

Each bolt group has one callout with its own bolt length: the flange bolts in the plan, the web bolts in the elevation. See Weld symbols and Drawings. The calculation report lists the inputs (configuration, members, seismic basis, loads, plates, bolts, welds, stiffeners and doubler) and every check; see Reports.

## FR auto-design, batch and compare

**Auto-design** uses the shared dialog described in Auto-design, compare and batch. For FR connections it never changes the members, the loads, the connection kind, the joint, the stiffener and doubler options or the geometry rules (pitch, gage, edge distances, setback). It sizes the hardware only, from the lightest up:

| Connection | What it searches, in order |
|---|---|
| Bolted flange plates | For each bolt diameter (3/4, 7/8 and 1 in., shared with the web bolts): the fewest bolts per row (2 to 10), the lightest plate width × thickness (both plates alike; widths from the current width, the beam flange width and the flange width + 1 in.), then the smallest plate-to-column fillet |
| Welded flange plates | For each flange: the thinnest plate, then the smallest plate-to-beam fillet (the plate lengthened in 2 in. steps when the weld is too short), then the smallest plate-to-column fillet |
| All, with a web plate | The fewest web bolts (2 to 12), the thinnest web plate, the smallest fillet to the column |

For directly welded flanges only the bolt diameter and the web plate are searched; the CJP welds need no sizing. Stiffeners and doublers are designed by the calculation itself.

A design is listed only when every non-advisory check passes. When no hardware can pass (for example the beam fails F13.1), the closest design is shown. The objectives are **Lightest (steel weight)**, **Fewest bolts** and **Lowest cost (fabrication estimate)**. The wizard's auto-design keeps **beam**, **column** and **conn**, and uses the lightest objective.

**Batch design** pastes a schedule of marks with the columns `mark`, `beam`, `column`, `MD`, `ML`, `VD` and `VL`. **Auto-design each mark** is available. **Compare** works as for the other types.

## PR moment connections: column stiffeners and doubler

The PR moment connection (top and seat angles, Manual Part 11) checks the same column limit states at the angle force lines: **web local yielding** at both angles (J10.2), **web crippling** at the compression angle (J10.3), the column flange in bending through the tension-angle bolts (prying, Manual Part 9) and the **panel zone** (J10.6). Its **Column reinforcement (J10.8, J10.9)** section offers the FR options:

- **Column stiffeners at the angles**: **None** (default; saved connections are unchanged), **Design when required** or **Provided (as entered)**, with the same thickness, width, length, clip, fillet, material and *finished to bear* fields as above. A pair is placed at each angle force line (the faying surfaces, or the angle-leg centrelines with that lever arm) where J10.2 / J10.3 fail. It carries Ru − φRn of the governing limit state (the FR rule) and its strength is added to those checks (*with stiffeners*). The angle length and thickness are the delivering plate for J10.8(a) and (b). A recommendation checks that the column-leg bolts inside the column flange clear the stiffener (tightening clearance).
- **Panel-zone web doubler**: **None** (default), **Design when required** or **Provided**, with doublers on one or both sides, fillet or CJP edges and an extension past the angle forces. Panel-zone shear with the doubler (J10.6, J10.9) and its welds, and the Manual Part 4 / Design Guide 13 recommendations, as for the FR connection.

Two PR inputs are engineering choices that the Manual leaves to you, so set them:

- **Flange-force lever arm** (Configuration): the beam depth d or the distance between the angle-leg centrelines. Manual Eq. 11-1 does not fix dm for flange angles. The Kishi-Chen moment-rotation model always rotates about the seat-angle leg, with a lever arm of d + tt/2 + ts/2.
- **Column ends near the joint (Spec. J10.2, J10.3)** and **Column top to top of beam** (Members): cleared (default), the column is not near its end and the checks use Eqs. J10-2 and J10-4. Ticked, the top-angle force is taken at the top of the beam and the seat force d lower; web local yielding uses Eq. J10-3 at an angle within d of the column end, and web crippling Eqs. J10-5a/b within d/2. The report states the distance and the equation.
- **Column J10.2/J10.3 bearing length lb** (Members): this is the bearing length at the column face that web local yielding and crippling use at each angle. The default 0 uses each angle's thickness. The calculation shows which value was used.

The moment-rotation curve and the B3.4b classification use the Kishi-Chen power model. The initial stiffness Rki and the capacity Mu come from Kishi and Chen (1990), *ASCE Journal of Structural Engineering* 116(7). Those equations use the seat angle's own thickness and length and the web angles' position on the beam web. The shape parameter n comes from Kishi, Chen, Goto and Matsuoka (1993), *AISC Engineering Journal* 30(3), Table 1. The 1993 paper restates the 1990 equations for equal top and seat angles with the web angles centred on the beam depth, and gives the same values there. The model is for angles bolted to the beam. If the angles are welded to the beam, the **Kishi-Chen model scope** check fails and the curve carries a note. The classification is then not valid for that connection. To get a valid classification, use bolted angles or establish the moment-rotation curve another way.

The stiffeners do not change the column-flange prying check at the tension-angle bolts (a conservative omission). The PR drawings show the stiffeners edge-on and the doubler face-on in the elevation, and the top stiffener pair with its clips in the plan section.

The B3.4 strength recommendation compares the moment at a rotation of 0.02 rad with 0.2 Mp. The Commentary takes the strength at 0.02 rad when the curve has no peak, and the power model has none. Without web angles, a **Shear connection** recommendation is shown. The Manual's flange-angle PR connection includes a simple shear connection (p. 11-27). Without web angles, the seat carries the reaction, checked with a simplified Part 10 unstiffened-seat model.

## PR moment connections: flange plates

The PR type also designs **flange-plated PR connections** (Manual pp. 11-28 to 11-30, Figs. 11-23 and 11-24). Select **Flange connection → Flange plates, bolted to the beam** or **Flange plates, welded to the beam**. In the wizard, use the two flange-plate cards. In this connection, top and bottom flange plates are welded to the column flange and bolted or welded to the beam flanges, and a single shear plate carries the shear.

**Strength.** The Manual takes the strength from the bolt, weld and connecting-element limit states of Parts 7, 8 and 9, and the FR flange-plate shop and field practices apply. ILX therefore runs the FR flange-plate checks described above for each load combination: flange bolts, plate tension, rupture, block shear and compression, beam flange block shear and F13.1, welds to the beam and to the column, and the column J10 checks with stiffeners and doubler. The flange-plate inputs (**Flange plates (Manual Fig. 11-23)** section) are the FR inputs. With the PR inputs set to the example geometry, the results reproduce Design Examples II.B-1 and II.B-2.

**Unwelded length.** The Manual says an unwelded length of 1½ times the plate width is "normally assumed to permit the elongation of the plate necessary for PR moment connection behavior". For welded plates, **Unwelded length from the column (0 = 1½ bp)** sets where the plate-to-beam welds start, measured from the column face. The default 0 uses 1½ bp for each plate. The welds run from there to the plate end, so the default plates are 18 in. long.

This length is also the plate's unbraced length in compression. A shorter entered length gives an advisory recommendation. If no weld fits past it, the geometry is not feasible. The plan drawing carries the note "DO NOT WELD FIRST ... FROM COL". For bolted plates, the Manual gives no length. The calculation reports the free length from the column face to the first bolt, af, against 1½ bp.

**Shear plate.** Manual p. 11-29 calls the shear connection "a simple shear connection". **Shear plate design** defaults to **Simple shear connection (Part 10)**, the full single-plate procedure with its eccentricity. **Direct shear** uses the FR assumption of p. 11-4 instead.

**Moment-rotation curve (engineer input).** Part 11 gives no moment-rotation model for flange plates. It only requires "a reliable moment-rotation, M-θ, curve" from tests, compilations or normalized curves (p. 11-26). B3.4b requires the curve to be "based on the technical literature or established by analytical or experimental means". Enter the curve under **Stiffness and classification (B3.4b)**:

- **M-θ curve**: **Linear, M = Ki θ** (default; the linear model after shakedown, p. 11-26) or **Power model (Ki, Mu, n)**, the same three-parameter form as the angle model.
- **Stiffness Ki**: kip-ft/rad (kN-m/rad). The default 0 means not entered.
- **Curve moment Mu**: required for the power model. For the linear curve it is an optional cap (0 = none).
- **Shape parameter n**: power model only (default 1.0).
- **Source of the M-θ curve**: the tests, literature or analysis the curve comes from. It is printed in the report.

Until Ki is entered, the **Moment-rotation curve** recommendation stays open and the connection is not classified. Once it is entered, the classification uses the same Commentary B3.4 rules as the angles: Ks = Ms/θs at the service moment, compared with 2EI/L and 20EI/L, and the moment at 0.02 rad compared with 0.2 Mp. The editor plots the curve. A recommendation notes that the curve must include the nonlinear and shakedown behavior (p. 11-26).

**Drawings.** The flange-plated configurations use the FR flange-plate elevation and plan. Welded plates show their welds starting past the unwelded length.

## FR limitations and deviations

### Scope

- Seismic: RBS, WUF-W and BFP (AISC 358-22) and OMF E1.6b. Welded flange plates in an IMF / SMF and column-web joints are flagged not adequate; extended end plates (Chapter 6), double tees (Chapter 13) and built-up or box members are not designed. The AWS D1.8 access-hole geometry is taken from AISC Design Guide 21, Fig. 11-5 (D1.8 itself is not in the ILX library). The panel-zone story shear Vc is the **Column story shear** entered; plug-welded doublers are not designed.
- Column-web joints are advisory only: a failing check describes the Manual Part 11 detail, and the column is not designed.
- Beams and columns are W, M, S or HP shapes. The opposite beam uses the same connection type and plate sizes.
- Beam flexure is checked as Mp at the connection only. Lateral-torsional and flange local buckling belong to the member design.

### Simplifications

- Diagonal panel-zone stiffeners are not designed; doublers only.
- The doubler is not credited in web local yielding (J10.2) or crippling (J10.3); the stiffeners carry those limit states.
- With beams on both sides, the stiffener-to-web weld force is the full stiffener force, not the difference of the two sides (conservative).

- Flange-plate bolts are in single shear without prying. In **Direct shear**, the web plate carries no moment (II.B-1).
- Bearing bolt values are used even when the pretension recommendation applies, as in Design Example II.B-1.
- The corner-clip, doubler-extent and (dz + wz)/90 rules are recommendations taken from the Design Guide 13 practice.
- The views are sized for the 11 × 17 sheet; on 8.5 × 11 portrait they may not fit, and an SMF design with continuity plates and a doubler needs 11 × 17 or larger.

**Verification**

ILX reproduces Design Examples II.B-1 (LRFD and ASD), II.B-2 (LRFD and ASD), II.B-3 and J.7:

- II.B-1: flange bolts, plate tension, rupture, block shear and compression, beam flange block shear, web plate, column J10.1 to J10.3 and F13.1 all within 0.4%; the flange forces (168 and 161 kips) are exact.
- II.B-2: within 0.6%. The beam flange shear rupture is 0.5% low because the Example rounds Anv.
- II.B-3: the Example refers the column checks to II.B-1. With dm = d − tf the flange force is 173.5 kips, just above the W14X99 flange local bending strength of 171 kips, so ILX designs stiffeners. This follows from the larger flange force of a directly welded connection; it is not a deviation.
- J.7: web local yielding and crippling within 0.1%.

See How ILX was verified.

CHAPTER 12

# Designing an end-plate moment connection

An end-plate moment connection is a plate shop-welded to the beam end and field-bolted to the column flange, or to a second end plate in a beam splice. The bolts at the tension flange and the end plate in bending carry the moment. The bolts at the compression flange carry the shear. ILX designs the 15 flush and extended configurations of AISC Design Guide 39, *End-Plate Moment Connections* (2023), Chapter 5, to AISC 360-22. It also checks the column, and designs the continuity plates and doublers the column needs. For seismic moment frames it designs the AISC 358-22 Chapter 6 extended end plates (4E, 4ES and 8ES) for SMF and IMF, and OMF connections by AISC 341-22 E1.6b (see Seismic moment frames).

## When to use an end-plate moment connection

Use it for a beam framing into a column flange, or a beam-to-beam splice, where the joint must carry moment: gravity and wind frames, R = 3 systems designed to AISC 360 only, and seismic moment frames (SMF and IMF with the 4E, 4ES or 8ES, and OMF).

- **Flush end plates** stay within the beam depth. They suit lighter moments, splices and joints where a projecting plate is in the way.
- **Extended end plates** project beyond the tension flange, with bolts outside the flange. They carry larger moments, and the stiffened versions add a triangular stiffener on the extension.

For SMF and IMF, AISC 358-22 prequalifies only the three extended end plates 4E, 4ES and 8ES; the other twelve configurations are flagged as not covered for those systems. For a simple shear end plate, use Designing a shear end-plate connection. For welded flanges or flange plates, use Designing an FR moment connection.

## Creating an end-plate moment connection

1. In the wizard's **Connection type** step, in the **Moment** group, select **End-plate moment**.
2. Choose the configuration. Each configuration starts from the plate, bolt and weld geometry of its DG39 Chapter 5 example. The members and loads are not copied.
3. Enter the **Connection mark** (ME-1, ME-2 and so on by default), the **Beam** and the **Column**, then the loads, and select **Create**.

Change the configuration, procedure and every dimension in the editor afterwards. The toolbar has **Design method** (**LRFD** or **ASD**) and **Units** (**US** or **Metric**), as for the other types. Auto-design is not available for this type yet; the checks propose the end-plate thickness, bolt diameter, continuity plates and doubler they need as suggested fixes.

# End-plate configuration and procedure

## The 15 configurations

The **End-plate configuration** sets the bolt pattern at the tension flange, the yield-line table and the stiffeners. The bolt rows are counted at one flange.

| Configuration | Bolts at the tension flange | Stiffener | Yield lines (DG39) | Column flange (Appendix A) |
|---|---|---|---|---|
| **Two-bolt flush unstiffened** | One row of 2 inside the flange | None | Table 5-2 | Table A-1 |
| **Four-bolt flush unstiffened** | Two rows of 2 inside | None | Table 5-3 | Table A-2 |
| **Four-bolt flush, web stiffeners between the tension rows** | Two rows of 2 inside, a web stiffener each side of the web between them | Web stiffeners | Table 5-4 | Table A-2 |
| **Four-bolt flush, web stiffeners below the tension rows** | Two rows of 2 inside, web stiffeners below the inner row | Web stiffeners | Table 5-5 | Table A-2 |
| **Six-bolt flush unstiffened (three rows)** | Three rows of 2 inside | None | Table 5-6 | Table A-4 |
| **Six-bolt flush four-wide / two-wide unstiffened** | A row of 4 and a row of 2 inside | None | Table 5-7 | Table A-3 |
| **Four-bolt extended unstiffened (4E)** | 2 outside, 2 inside | None | Table 5-10 | Table A-2 |
| **Four-bolt extended stiffened (4ES)** | 2 outside, 2 inside | End-plate stiffener | Table 5-11 | Table A-2 |
| **Multiple-row extended 1/2 unstiffened** (MRE 1/2) | 2 outside, two rows of 2 inside | None | Table 5-12 | Table A-4 |
| **Multiple-row extended 1/3 unstiffened** (MRE 1/3) | 2 outside, three rows of 2 inside | None | Table 5-13 | Table A-5 |
| **Multiple-row extended 1/3 stiffened** | 2 outside, three rows of 2 inside | End-plate stiffener | Table 5-14 | Table A-5 |
| **Eight-bolt extended four-wide unstiffened (8E/4W)** | 4 outside, 4 inside | None | Table 5-15 | Table A-6 |
| **Eight-bolt extended stiffened (8ES)** | Two rows of 2 outside, two rows of 2 inside | End-plate stiffener | Table 5-16 | Table A-5 |
| **12-bolt multiple-row extended 1/3 four-wide / two-wide unstiffened** | 4 outside; 4, 2 and 2 inside | None | Table 5-17 | Table A-7 |
| **12-bolt extended stiffened (MRE 2/2)** | 2 and 4 outside; 4 and 2 inside | End-plate stiffener | Table 5-18 | Table A-8 |

Four-wide rows have bolts at ±g/2 and ±(g/2 + go). Inside rows sit at pfi, then pb apart, below the inside face of the tension flange. Outside rows sit at pfo (then pb) beyond its outside face.

## Configuration fields

| Field | Options and notes |
|---|---|
| **End-plate configuration** | The 15 configurations above. Default: **Four-bolt extended unstiffened (4E)**. |

| Field | Options and notes |
|---|---|
| **Design procedure** | **Thick plate (no prying)** (default) or **Thin plate (prying)**. See below. |
| **Beam frames into** | **Column flange** (default), or a beam-to-beam **splice**. A splice has no column checks; the mating end plate is identical (DG39 Fig. 1-1(d)). |
| **Moment connections at the column** | **One side** (default), or both column flanges. Two-sided adds the **opposite beam** and its moment M2 for the panel zone and web compression buckling. |
| **Seismic system** | **Not seismic** (default), **R = 3**, **OMF**, **IMF** or **SMF**. OMF, IMF and SMF open the **Seismic (AISC 341-22 / 358-22)** section (see Seismic moment frames). |
| **Bolts at the other flange** | **Same pattern (moment reversal)** (default), or **Two compression bolts** at pfi from the other flange. |

### Thick or thin plate: your choice

DG39 §4.1.1 accepts both procedures. Pick the one you want to design to:

- **Thick plate (no prying)** (§5.1.1). The plate is stiff enough that the bolts see no prying. The bolts are checked for φ Pt Σ ni hi ≥ Mr,eq (Eq. 5-3). The plate is checked for γr φb Fyp tp² Yp ≥ 1.10 Mr,eq (Eq. 5-4). This gives a thicker plate and smaller bolts.
- **Thin plate (prying)** (§5.1.2). The plate yields first and the bolts carry prying forces. The plate is checked for γr φb Fyp tp² Yp ≥ Mr,eq (Eq. 5-5). The bolts are checked for φ Σ αi Pqi ni hi ≥ Mr,eq (Eq. 5-6), with the tee-stub prying force at every bolt location. This gives a thinner plate and larger bolts.

γr is 0.80 for flush plates and 1.0 for extended plates (Eq. 5-1). The procedure is shown in the drawing key data and the report title, for example *4E, thick plate*.

**Bolts at the other flange.** DG39 tabulates the bolt pattern at the tension flange only. **Same pattern** mirrors it at the other flange, as needed for moment reversal. **Two compression bolts** puts two bolts at pfi from the compression flange, which is enough for shear when the moment never reverses. If any combination puts that flange in tension, a failing check says so.

## End-plate loads

The load modes are those of the FR moment connection (see Moment, shear and axial loads):

| Input | Enter |
|---|---|
| **Service D + L** (default) | M, V and P for dead and live load. Defaults: MD = 40 kip-ft, ML = 80 kip-ft, VD = 10 kips, VL = 20 kips. |
| **ASCE 7 combinations** | M, V and P for D, L, Lr, S, W and E. Every ASCE 7-22 combination is run, with W and E in both directions. |
| **Required strength** | Mu, Vu and Pu (Ma, Va and Pa in ASD), used as entered |
| **Load cases** | A table of named cases; each is checked and the envelope governs |

- M > 0 puts the top flange in tension. P is positive in tension. Moments are in kip-ft (kN-m), forces in kips (kN).
- The design moment is Mr,eq = Mr + (Tr/2)(d − tbf) (DG39 Eq. 3-30). Axial compression is neglected, which is conservative (§3.5). The flange force is Tr,calc = Mr/(d − tbf) + Tr/2 (Eq. 3-39).
- If the axial term exceeds half the moment, a recommendation warns that the yield-line pattern may change (§3.7.2(11)).

- A moment that changes sign between combinations is a load reversal. The bolts must then be pretensioned (RCSC 4.2(2)).
- **Column axial load** (compression +) enters the panel-zone strength (J10.6). **Column story shear** Vrc reduces the panel-zone shear (Eq. 3-34).
- **Beam axial force in the panel-zone shear**. **DG39 Eq. 3-34** (default) uses the equation as written, so beam tension reduces the panel shear. **Adds to the flange force** adds P/2 to the flange force, as in Example 5.2-1, a knee joint where the axial force acts with the flange force. DG39 does not settle which applies to your frame, so choose it.
- **Analysis includes inelastic panel-zone deformation** uses J10.6(b) in place of J10.6(a).

## End-plate beam and column

| Field | Notes |
|---|---|
| **Beam section** | **Rolled W shape** (default) or **Built-up (welded plates)** |
| **Beam**, **Beam material** | W, M, S or HP shape. Default W18X50, ASTM A992. |
| **Built-up beam depth d**, **flange width bf**, **flange thickness tf**, **web thickness tw**, **flange-to-web fillet** | Built-up beams only. Defaults 24, 8, 1/2, 3/8 and 1/4 in. |
| **Column**, **Column material** | W, M, S or HP shape. Default W14X99, ASTM A992. |
| **Opposite beam** | Two-sided connections only. It uses the same end plate. |
| **Column ends at this connection (roof)** | The column stops at the beam. Web local yielding uses Ct = 0.5, and the column flange uses the top-of-column yield lines of Appendix A. |
| **Column end to the connection force** | The column top above the top of the beam |
| **Column cap plate** | A cap plate at the column top, with its **thickness**, **width (0 = column flange)**, **fillet to the column flange** (single) and **fillet to the column web** (both faces). Defaults 1/2 in., the flange width, 1/2 in. and 1/4 in. |

With a cap plate and a flush end plate, the column flange uses the cap-plate yield lines of Appendix A. Appendix A has no cap-plate yield lines for extended plates, so that combination gives a failing check and the flange is checked as a top-of-column flange.

## End plate and stiffeners

| Field | Default | Notes |
|---|---|---|
| **End-plate width bp** | 8 in. | Only bf + max(tp, 1 in.) is effective (Eq. 4-3) |
| **Plate thickness tp** | 1 in. | |
| **Plate material** | A572 Gr. 50 | |
| **Bolt gage** g | 5½ in. | g ≤ bf (Eq. 4-1); g + 2go ≤ bf for four-wide rows (Eq. 4-2) |
| **Four-wide bolt spacing go** | 3 in. | Four-wide configurations |
| **Bolt pitch inside the flange pfi** | 1¾ in. | Inside face of the flange to the first inside row |

| Field | Default | Notes |
|---|---|---|
| **Bolt pitch outside the flange pfo** | 1¾ in. | Extended plates |
| **Bolt row pitch pb** | 3 in. | Between rows |
| **End distance de** | 1½ in. | Extended plates: outer row to the plate edge |
| **Row 1 to the web stiffener pso**, **Web stiffener to the next row psi** | 1½ in. each | Flush plates with web stiffeners. Between the rows, pb = pso + tws + psi. |
| **Plate projection beyond the flange (flush)** | 1/2 in. | Must clear the flange fillet weld |
| **Web stiffener thickness**, **length**, **corner clip**, **fillet**, **material** | 1/4 in., 5 in., 1/2 in., 3/16 in., A572 Gr. 50 | Flush plates with web stiffeners: a plate each side of the beam web |
| **End-plate stiffener thickness**, **material** | 1/2 in., A572 Gr. 50 | 4ES, MRE 1/3 stiffened, 8ES, 12-bolt stiffened |
| **End-plate stiffener length (0 = automatic)** | 0 | Automatic: hst / tan 30°, rounded up to 1/4 in. (Eq. 4-4) |
| **End-plate stiffener welds**, **fillet** | Fillet, both sides; 3/8 in. | Or CJP groove welds, or **CJP to the end plate, fillets to the flange** (AISC 358-22 6.6.6(6) for stiffeners over 3/8 in., as DG39 Examples 6.3-2 and 6.3-3) |
| **Stiffener-to-flange fillet (0 = same)** | 0 | A different fillet for the stiffener-to-flange weld (Example 5.3-7 uses 7/16 in. there and 1/2 in. to the plate) |

The **effective plate width** bp,eff = min(bp, bf + max(tp, 1 in.)) is used in every yield line and tee stub (Eq. 4-3, §3.7.2(5)). A wider plate is allowed and a recommendation shows the width used.

## End-plate bolts

| Field | Options and notes |
|---|---|
| **Bolt grade** | The bolt library (default A325, Group 120) |
| **Bolt diameter** | Default 7/8 in. |
| **Threads** | **Included (N)** (default) or **Excluded (X)**, for the shear strength |
| **Joint type** | **Pretensioned** (default) or snug-tight (bearing) |
| **Hole deformation at service** | Bearing 2.4 d t Fu (default) or 3.0 d t Fu (J3.11) |

**Snug-tight or pretensioned.** DG39 §4.1.3 allows snug-tight Group 120 (A325) bolts for gravity, wind and R = 3 loads that are not fatigue loads and do not reverse significantly. With snug-tight bolts a recommendation says so. Group 150 and Group 200 bolts in tension, and any joint with a reversing moment, must be pretensioned (RCSC 4.2), or a failing check says so.

The pretension Tb matters only in the thin-plate procedure, where Pq = max(Pt − Qmax, Tb) (Eq. 3-27):

- Pretensioned: Tb from AISC 360-22 Table J3.1.

- Snug-tight: Tb from DG39 Table 3-3. It is a fraction of the full pretension for Group 120 and a tabulated value for Group 150. Table 3-3 has no values above 1 in. or for Group 200, so Tb = 0 is used there and a recommendation says so.

**Cyclic loading (fatigue).** In the Bolts section, *Cyclic loading* turns on the AISC 360-22 Appendix 3 checks: the tension bolts (Case 8.5, with the thin-plate prying force; 20 % of the load for pretensioned bolts with only steel in the grip) and the beam flange to end-plate weld (CJP, Case 5.4, or fillets, Cases 5.7 and 8.2), for the tension-flange force Tr,calc times the cyclic range of the moment.

## End-plate welds

| Field | Options and notes |
|---|---|
| **Electrode** | E70XX (default) or E80XX |
| **Flange to end plate** | **CJP groove weld** (default) or **Fillet, both faces** |
| **Flange fillet weld** | Default 3/8 in., both faces of the flange |
| **Flange weld demand** | **Required strength, ≥ 60 % / 40 % of the flange (DG39 Eq. 3-38)** (default) or **Flange yield strength** |
| **Effective flange fillet length** | **Automatic** (default), **2bf − 2k1**, **2bf − tw**, or **2bf − tw − 2w** |
| **Web fillet near the tension bolts** | Default 5/16 in., both faces |
| **Web fillet (shear)** | Default 1/4 in., both faces, over the rest of the web |

Three weld rules are left to the engineer by DG39, so ILX asks for them:

- **Flange weld demand.** DG39 Eq. 3-38 designs the flange weld for the required flange force, but not less than 60 % (LRFD) or 40 % (ASD) of the flange yield strength Fy bf tf. §3.7.5 says the weld is *normally* designed for the flange yield strength. **Flange yield strength** does that. The same choice applies to the web weld near the tension bolts (Eq. 3-41).
- **Effective flange fillet length.** The DG39 examples use different lengths. **2bf − 2k1** is used for rolled beams (Examples 5.2-1 and 5.2-2). **2bf − tw** is used in Example 5.3-1. **2bf − tw − 2w** deducts the web welds for built-up beams (Examples 5.2-3 to 5.2-6). **Automatic** uses k1 for rolled beams and the web welds for built-up beams.
- **CJP flange welds** develop the flange. DG39 §4.2.3 calls for no weld access holes, welding from the outside face of the flange, and a full-depth PJP weld over the web between the flange-to-web fillets. A recommendation repeats this.

## Column, continuity plates and doublers

| Field | Options and notes |
|---|---|
| **Transverse stiffeners** (continuity plates) | **Design when required** (default), **Provided (as entered)** or **None** |
| Stiffener **thickness**, **width (0 = max)**, **length**, **corner clip**, **fillet**, **material**, **finished to bear** | As for the FR connection. Defaults 1/2 in., maximum width, full depth, 3/4 in. clip, 1/4 in. fillet, A572 Gr. 50. |
| **Web doubler** | **Design when required** (default), **Provided (as entered)** or **None** |
| Doubler **thickness**, **sides**, **edge weld**, **fillet**, **extension**, **material** | Defaults 3/8 in., one side, fillet 1/4 in., no extension, A572 Gr. 50 |

With **Design when required**, ILX designs continuity plates when any column limit state at a flange falls short. They are designed for Tr,calc − min φRn of the column limit states at that flange (DG39 Eq. 3-48), using the same J10.8 designer as the FR connection. A doubler is added when the panel zone fails. The designed sizes are offered as suggested fixes. With **None**, a failing column check stays failed.

## Checks the end-plate connection runs

Every check is run for every load combination and the governing one is reported. See Checks and compliance for reading the results.

**End plate.**

- Flexural yielding by yield lines: Mpl = Fyp $t_p^2$ Yp (Eq. 3-15), with Yp from the configuration's table (Tables 5-2 to 5-7, 5-10 to 5-18) and bp,eff (Eq. 4-3). The calculation lists every bolt row's h, s = √(bp g)/2 and the pf, ps values used. Thick plates are checked against 1.10 Mr,eq (Eq. 5-4), thin plates against Mr,eq (Eq. 5-5), both with γr. The required thickness tp,reqd is shown and suggested.
- Extended part, unstiffened extended plates (4E, MRE 1/2, MRE 1/3, 8E/4W, 12-bolt four-wide): shear yielding and shear rupture under the share of the flange force carried by the outside bolts (§5.1.3 A.3; AISC 360-22 J4.2).
- Effective width bp ≤ bf + max(tp, 1 in.) (Eq. 4-3, recommendation). For 4ES, MRE 1/3 stiffened and 8ES, de < s, so that the tabulated yield-line pattern applies (§5.3.1).

**Bolts.**

- Thick plate: tension rupture without prying, φMnp = φ Pt Σ ni hi ≥ Mr,eq, with Pt = Fnt Ab and φ = 0.75 (Ω = 2.00) (Eq. 5-3). The required diameter db,reqd is shown and suggested.
- Thin plate: tension rupture with prying, φMq = φ Σ αi Pqi ni hi ≥ Mr,eq (Eqs. 5-6, 3-28). Each bolt location is a vertical tee stub (next to a flange) or a horizontal one (next to the web or a stiffener), as in DG39 Fig. 5-1. Each has its own w, b and a (Eq. 3-24), F' (Eq. 3-26), Qmax (Eq. 3-23) and Pq = max(Pt − Qmax, Tb) (Eq. 3-27), all listed. Second interior rows take Pq = Tb. Four-wide rows use the α factors of the tables. If a tee stub has no flexural strength left for prying (a negative radicand in Eq. 3-23), a failing check names the bolt location.
- Shear by the compression-side bolts: bolt shear, bearing on the end plate and on the column flange (AISC 360-22 J3.7, J3.11; DG39 §3.7.2(4)). Tearout does not control (Example 5.2-1).
- Pretension required (RCSC 4.2) and the snug-tight notes above.
- Detailing: g ≤ bf or g + 2go ≤ bf (Eqs. 4-1, 4-2), spacing ≥ 2⅔d (J3.4), side and end edge distances (J3.5, J3.6), and the tightening clearance from the flange (§3.7.2(3), recommendation).

**Welds and beam.**

- Flange-to-plate fillets for the flange weld demand above (Eq. 3-38; AISC 360-22 J2.4, θ = 90°), and the minimum size (Table J2.4). CJP welds are a note.
- Beam web tension yielding near the tension bolts (Eq. 3-40; J4.1(a)). The web is checked from the flange to 6 in. past the innermost tension row, carrying the share of the flange force of the bolts next to the web.
- Web-to-plate welds near the tension bolts (Eq. 3-41).
- Web-to-plate welds for shear, and web shear rupture along them (J2.4, J4.2(b)), over the lesser of d − 2tf − the tension zone, and d/2 − tf (§3.7.2(10)).
- Minimum web fillet (Table J2.4).

**Web stiffeners** (flush plates with web stiffeners, Examples 5.2-3 and 5.2-4).

- The web and the stiffeners share the flange force over the tension zone.
- The stiffener-to-web fillets and the base metal shear rupture along them (J2.4, J4.2(b)).

- Length ≥ hws / tan 30° (Eq. 4-4, recommendation).

**End-plate stiffener** (4ES, MRE 1/3 stiffened, 8ES, 12-bolt stiffened).

- Thickness ts ≥ tbw Fyb / Fys (Eq. 3-36).
- Slenderness hst / ts ≤ 0.56 √(E/Fys) (Eq. 3-37). It is required only for high-seismic-ductility design, so here it is a recommendation.
- Length Lst ≥ hst, a slope of 45° at most (§4.2.5), and Lst ≥ hst / tan 30° (Eq. 4-4, recommendation).
- Fillets: to the beam flange for the stiffener's shear yield strength, and to the end plate for its tensile yield strength, per inch. CJP welds are a note.

**Column** (beam to the column flange).

- Column flange flexural yielding at the tension bolts, by the yield lines of DG39 Appendix A: Mcf = Fyc tcf² Yc (Eq. 3-43), Rn = Mcf/(d − tbf) (Eq. 3-42). The demand is 1.10 Tr,calc, because the column flange must act as a thick plate (Eq. 3-19). The unstiffened, top-of-column and cap-plate cases each have their own table.
- With continuity plates, the stiffened yield lines of Appendix A, with bcf,eff = 2bs + tcw + max(tcf, 1 in.) (Example 5.3-2). When the stiffened flange is adequate, the unstiffened check becomes *continuity plates required*.
- Web local yielding at both flanges: Rn = (6 Ct kc + lb) Fyc tcw (Eq. 3-46), with lb = tbf + 2w + 2tp (Eq. 3-47), limited by the plate projection as in Example 5.2-1. Ct = 0.5 when the column top is within dc of the flange force, otherwise 1.0. With a cap plate at the tension flange, the cap plate delivers the force (Example 5.2-1).
- Web crippling at the compression flange (J10.3).
- Web compression buckling (J10.5), two-sided connections only.
- Continuity plates (Eq. 3-48; AISC 360-22 J10.8): width, thickness, length, strength, welds and clips, as for the FR connection (see Column stiffening (J10)).
- Cap plate (Example 5.2-1): the weld to the column flange and the welds to the column web for the flange weld demand, column web shear rupture at the weld (J4.2(b)), and a cap plate at least as thick as the beam flange.
- Panel-zone shear (DG39 Eq. 3-34; J10.6) by J10.6(a) or (b), and with the doubler (J10.9) with its welds to the column flanges.

**NOT FEASIBLE** is reported when the geometry cannot be built: a bolt hole in the beam flange or web, outer bolts off the plate, tension rows in the compression half of the beam, a web stiffener that does not fit between the rows, a flush plate edge inside the flange weld, or the first bolt row above the column top.

**Tested parameter range.** The DG39 equations were validated within the tested ranges of Table 5-1 (flush) and Table 5-9 (extended), ± 10 % (§4.1.2). A recommendation lists every parameter outside: pf, pb, g, go, d, bp, tf and, for extended plates, the extension pext. Outside the range the yield-line pattern may differ, so the design rests on your judgment.

| Parameter | Flush (Table 5-1), in. | Extended (Table 5-9), in. |
|---|---|---|
| pf | 1⅛ to 2¼ | 1 to 2½ (MRE 1/2: to 5) |
| pb | 1⅞ to 3½ | — |
| g | 2¼ to 4¼ | 2¾ to 7 |
| go | 3 | 2¼ to 3¾ |
| d | 16 to 24 (two-bolt from 8; six-bolt to 36; four-wide to 60) | 15¾ to 72 |
| bp | 5 to 14 | 5 to 10¼ |
| tf | 3/16 to 3/4 | 3/8 to 1 |

| Parameter | Flush (Table 5-1), in. | Extended (Table 5-9), in. |
|---|---|---|
| pext | — | 2½ to 7½ |

## Seismic moment frames (AISC 358-22 Chapter 6, AISC 341-22)

Set **Seismic system** to **SMF**, **IMF** or **OMF** and a **Seismic (AISC 341-22 / 358-22)** section opens below the configuration. A **Seismic** card above the drawings shows the design basis: Ry and Cpr, Mpr, Sh, Lh, Vh, Mf, Ffu, Vu and the protected zone.

### Which design applies

| System | Design |
|---|---|
| **SMF**, **IMF** | AISC 358-22 Chapter 6: the bolted unstiffened extended end plate **4E** (BUEEP) or the bolted stiffened extended end plates **4ES** and **8ES** (BSEEP), designed by LRFD for the probable moment at the column face. The beam is bolted to the column flange with the same pattern at both flanges. Any other configuration, a beam splice or two compression bolts at the other flange is flagged as not covered. |
| **OMF**, **E1.6b(a)** (default) | The DG39 Chapter 5 design (DG39 applies Chapter 5 to OMF) with two more load combinations, ±Mr = 1.1 Ry Mp / αs with Vr from Ecl = 2(1.1 Ry Mp)/Lcf (Eq. E1-1) plus the gravity shear, divided by αs. Continuity plates follow Spec. J10 at those forces; the panel zone uses the code combinations only (E1.6b User Note). |
| **OMF**, **E1.6b(b)** | As (a), with the maximum moment and shear the system can deliver, entered by you. |
| **OMF**, **AISC 358-22 Chapter 6** | The Chapter 6 design with the IMF limits (E1.6b(c) accepts a connection meeting E2.6 or E3.6). |

AISC 341-22 E1.6b(c) is the prescriptive detail for beam flanges welded to the column, so it is not an option for an end plate.

The code load combinations always run the DG39 Chapter 5 design as well, so the connection is also checked for the loads you entered. The AISC 358-22 checks run once, by LRFD even when the design method is ASD (358-22 1.3), and their keys start with sm_.

### Seismic fields

| Field | Default | Notes |
|---|---|---|
| **OMF connection** | E1.6b(a) | OMF only: (a), (b) or the 358-22 Chapter 6 connection |
| **Clear span between column faces, Lcf** | 360 in. | Lh = Lcf − 2 Sh, and the span-to-depth limit (≥ 7 SMF, ≥ 5 IMF, 6.3.1(5)) |
| **Gravity line load wu** | 1.25 kip/ft | 1.2D + f1L + 0.15S on the beam between the hinges (358-22 2.4.4), with f1 from your building code |
| **Further gravity shear at the hinge** | 0 | Concentrated loads in the same combination, added to wu Lh / 2 |
| **Moment connection at this end only** | Off | Vh = Mpr / (Lcf − Sh) + Vgravity (2.4.4 User Note) |
| **Cpr** | 0 = Eq. 2.4-2 | (Fy + Fu)/(2 Fy) ≤ 1.2; an entry overrides it, and the standard value is printed beside it |

| Field | Default | Notes |
|---|---|---|
| **Beam Ry**, **Rt** | 0 = Table A3.2 | AISC 341-22 Table A3.2 permits other values by testing |
| **Maximum moment by the system**, **Corresponding shear** | 0 | OMF E1.6b(b) |
| **Concrete structural slab with shear connectors** | On | 6.2 Exception: nominal beam depth ≥ 24 in.; also the bracing exception of 6.3.1(7) |
| **Slab isolated, no shear connectors within 1.5d** | Off | 6.2 Exception (2) and (3), 2.3.4: you confirm it, or a failing check says so |
| **A moment-ratio exception of E3.4a applies** | Off | SMF: the ratio then shows as information |
| **Fillet w in lb = tbf + 2w + 2tp** | w = 0 | Eq. 6.7-17 takes w from a double fillet flange weld. The 358 flange weld is a CJP, so as written w = 0. DG39 Chapter 6 counts the 5/16 in. inside fillet once (tbf + w + 2tp); choose that to follow DG39. |
| **Minimum pitch of 6.6.2 as a recommendation** | Off | 358-22 6.6.2 states pfi, pfo ≥ db + 1/2 in. (db + 3/4 in. above 1 in.). DG39 Table 6-3 lists it as a recommendation, and Table 6.1 caps the 8ES pitch at 2 in., which rules out 1 3/8 in. bolts. Off, it is a requirement. |

The stiffener weld choice of the End-plate stiffener section applies: 358-22 needs a CJP groove weld to the end plate when the stiffener is thicker than 3/8 in. Select **CJP to the end plate, fillets to the flange**. The continuity plate and doubler fields of the Column section apply as well.

## Checks (AISC 358-22 Chapter 6 and AISC 341-22 Chapter E)

**Capacity design** (6.7.1 Steps 1 to 4; 2.4): Mpr = Cpr Ry Fy Zb (Eq. 2.4-1). Sh is the lesser of d/2 and 3bf for the 4E (Eq. 6.7-1), or Lst + tp for the 4ES and 8ES (Eq. 6.7-2). Then Vh = 2 Mpr / Lh + Vgravity (Eq. 2.4-3), Mf = Mpr + Vh Sh (Eq. 2.4-4), Vu = Vh + wu Sh (2.5) and Ffu = Mf / (d − tbf) (Eq. 6.7-6).

**Prequalification limits** (6.2, 6.3):

- every Table 6.1 parameter within its range for the configuration: tbf, bbf, d, tp, bp, g, pfi, pfo and, for the 8ES, pb. 358-22 says "all connection elements shall be within the ranges shown", so there is no tolerance;
- a rolled W or built-up I beam (built-up: web-to-flange fillets ≥ 0.75 tw and ≥ 1/4 in. near the connection, 6.3.1(1));
- a rolled W column no deeper than W36;
- the clear span-to-depth ratio;
- the slab exception;
- width-to-thickness of the beam and column as highly ductile (SMF) or moderately ductile (IMF) members (AISC 341-22 Table D1.1b).

**Bolts and detailing** (6.5, 6.6): pretensioned A325, A490, F1852 or F2280 bolts (Chapter 4); g ≤ bbf; the minimum pitch; pb ≥ 2⅔ db (8ES); bp ≥ bbf, with the effective width bbf + 1 in. at most; stiffener length Lst ≥ hst / tan 30° (Eq. 6.6-1), with 1 in. landings.

**End plate and bolts** (6.7.1):

- Bolt rupture without prying: Mf ≤ φn 2 Pt Σ hi, with db,req (Eqs. 6.7-3, 6.7-4) shown and suggested.
- Plate yielding: φd Fyp tp² Yp ≥ 1.11 Mf, with tp,req (Eq. 6.7-5) shown and suggested. Yp comes from Table 6.2 (4E), Table 6.3 (4ES) or Table 6.4 (8ES), Case 1 (de ≤ s) or Case 2 (de > s), with pfi ≤ s.

- 4E only: shear yielding and rupture of the extension, Ffu/2 ≤ φd 0.6 Fyp bp tp and ≤ φn 0.6 Fup An (Eqs. 6.7-7, 6.7-8).
- 4ES and 8ES: stiffener thickness ts ≥ tbw Fyb / Fys (Eq. 6.7-9) and hst / ts ≤ 0.56 √(E/Fys) (Eq. 6.7-10). The stiffener-to-flange weld develops the stiffener in shear. The stiffener-to-plate weld is a CJP, or fillets that develop it in tension when ts ≤ 3/8 in.
- Beam shear at Vu (Spec. G2.1).
- Bolt shear Vu ≤ φn nb Fnv Ab, with the 4 or 8 bolts of one flange (Eq. 6.7-11).
- Bearing and tearout on the end plate and the column flange, Vu ≤ φn Σ rni (Eq. 6.7-12). Every bolt of the compression flange is checked, at either flange because the seismic moment reverses. Tearout goes toward the next hole or the plate edge.

The resistance factors are φd = 1.00 for ductile and φn = 0.90 for nonductile limit states (358-22 2.4.1).

**Welds** (6.6.6):

- The flange weld must be the CJP groove weld without backing, with the 5/16 in. fillet on the inside face. It is demand critical. There are no weld access holes.
- The web fillets near the tension bolts develop the web in tension (Fy tw per inch) from the flange to 6 in. past the farthest row.
- The web fillets of the compression-side zone carry Vu, as DG39 Example 6.3-3 does.

**Column** (6.7.2), at both flanges:

- Flange yielding: tcf ≥ √(1.11 Mf / (φd Fyc Yc)) (Eq. 6.7-13), with Yc from Table 6.5 (four-bolt) or Table 6.6 (eight-bolt). When the top bolt row is less than s below the column top, the top-of-column yield lines of DG39 Appendix A are used, because the 358 tables assume more than s.
- Web local yielding (Eqs. 6.7-16, 6.7-17), with Ct = 0.5 when the column top is less than dc above the beam.
- Web crippling (Eqs. 6.7-18 to 6.7-21).

If any of these fails, continuity plates are designed for Fsu = Ffu − min(φRn) (Eq. 6.7-22) with the J10.8 designer. They are then checked to AISC 341-22 E3.6f.2: thickness ≥ 0.5 tbf (0.75 tbf two-sided), b/t ≤ 0.56 √(E/(Ry Fy)), width to the beam flange tips, flange welds CJP or fillets ≥ 0.75 t, web welds, and the 358-22 3.6 clips. The 6.4(3) exception for plates ≤ 3/8 in. is also checked. The column flange is then rechecked with the stiffened Yc, with psi and pso measured from the plate faces.

**Panel zone and moment ratio**: for an SMF, the panel zone by AISC 341-22 E3.6e (Ru = Σ Mf / (d − tbf) − Vc, φv = 1.00, doubler designed, Eq. E3-6) and the column-beam moment ratio by Eq. E3-1. For an IMF, the J10.6 panel-zone check of the code combinations applies (E2.6e).

**Bracing and protected zone**: supplemental bracing between the end of the protected zone and d/2 beyond it, unless the slab exception applies (6.3.1(7)). Beam bracing spacing per AISC 341-22 D1.2. The protected zone (6.3.1(8)) runs from the column face to the lesser of d and 3bf beyond the end plate (4E), or to the end of the stiffener plus the lesser of d/2 and 3bf (4ES, 8ES). It is hatched on the drawing.

## Seismic verification

ILX reproduces the three worked examples of DG39 Chapter 6, which follow the 358-22 procedure: 6.3-1 (4E), 6.3-2 (4ES) and 6.3-3 (8ES). The comparison covers the capacity design, the bolt and plate design, the stiffener, bolt shear and bearing, the welds, the column side, the panel zone and the moment ratio. Where DG39 departs from the standard as written, ILX follows the standard and the test says why:

- the panel-zone demand (DG39 also subtracts half the beam axial force);

- the tearout of the 8ES end plate (DG39 judges by inspection that it does not control);
- the pitch rule above;
- rounding in Example 6.3-1, where unrounded Mf is 0.3 % over the bolt strength.

The record is in research/67.

## End-plate drawings, report and sheet

The editor draws the connection in elevation and the end plate face-on. The key data strip shows the plate, the configuration and procedure, the flange force, Yp, the continuity plates and the doubler. Open **Required strength and flange forces** to see Mr,eq and Tr,calc worked out. The calculation report lists the inputs (configuration and procedure, beam, support, seismic basis, loads, end plate, bolts, welds, stiffeners, continuity plates and doubler) and every check; see Reports and Drawings.

## End-plate limitations and verification

### Scope

- Seismic: AISC 358-22 Chapter 6 covers the 4E, 4ES and 8ES only. Built-up, boxed and cruciform columns are not designed. The top-of-column (roof) joint uses the DG39 Appendix A yield lines; 358-22 itself prequalifies intermediate-story joints (its Commentary 2.6 calls a top-of-column detail "believed to be capable of providing reliable performance").
- Built-up columns are not modeled: the column is a rolled W, M, S or HP shape. Built-up beams are supported.
- Sloped beams and knee-joint geometry are not modeled. The beam is horizontal and frames square to the column flange.
- Panel-zone tension field action is not used. The panel zone is checked by J10.6 and strengthened with doublers only.
- Column-flange prying is not modeled. The column flange must act as a thick plate (φRn ≥ 1.10 Tr,calc, Eq. 3-19). If it does not, add continuity plates or use a heavier column.

### Engineer's choices

DG39 leaves these to you. The defaults are listed above:

- the procedure (thick or thin);
- the bolt pattern at the other flange;
- the flange weld demand rule (60 % / 40 % or the flange yield strength);
- the effective flange fillet length;
- the sign of the beam axial force in the panel zone (Eq. 3-34 or Example 5.2-1);
- snug-tight or pretensioned bolts, and Table 3-3;
- the column top and cap plate;
- the continuity plates and doubler (design when required, provided or none).

### Verification

ILX reproduces all 30 non-seismic end-plate examples of DG39, in LRFD and ASD, and the three seismic examples of Chapter 6 (see Seismic moment frames):

- the 15 thick-plate examples of Chapter 5 (5.2-1 to 5.2-6 flush, 5.3-1 to 5.3-9 extended), one per configuration;
- the 15 thin-plate examples of Appendix B (B.2-1 to B.2-6, B.3-1 to B.3-9).

The comparisons cover Yp, the plate and bolt strengths, the prying forces, the welds and the stiffeners, and the column side in the two examples with a column (5.2-1 and 5.3-2). The method and the example-by-example record are in research/64. See How ILX was verified.

CHAPTER 13

# Designing a double-tee moment connection

The **Double-tee moment** type (Wizard group *Moment*, mark prefix `DT`) designs the prequalified double-tee connection of AISC 358-22 Chapter 13 with the AISC 341-22 Chapter E requirements. Two identical T-stubs, cut from a rolled W shape, are bolted to the column flange with **tension bolts** and to the beam flanges with **shear bolts**; the beam web has a bolted single plate welded to the column flange. Yielding is intended in the beam just past the last shear bolt.

| System | Basis |
|---|---|
| Special moment frame (SMF) | 358-22 13.2; highly ductile members, panel zone and column-beam moment ratio (341-22 E3) |
| Intermediate moment frame (IMF) | 358-22 13.2; moderately ductile members (341-22 E2) |
| Ordinary moment frame (OMF) | 341-22 E1.6b(c): an OMF may use a connection that satisfies E2.6, so ILX designs the IMF connection |

There is **no non-seismic option**. Neither AISC 358 nor the Steel Construction Manual covers a double-tee moment connection outside a seismic system, so ILX does not offer one.

The design is a **capacity design**: the forces follow from the beam's probable maximum moment Mpr at the plastic hinge, not from entered loads, so the editor has no Loads section. The connection is designed by LRFD whatever the design method (358-22 1.3, φd = 1.00, φn = 0.90); choosing ASD changes only the AISC 341 terms that carry αs = 1.5 (Ca of Table D1.1b, Eq. E3-2 and the brace forces).

## Double-tee inputs

Every value AISC 358 leaves to the engineer is an input; its default is listed here.

- **Frame and capacity design**: the seismic system (SMF); one beam or two identical beams on both column flanges; the clear span **Lcf** (360 in.; Lcf / d ≥ 9, 13.3.1(5)); the beam length **Lo** between column work points for the stiffness check (0 = Lcf + dc); a moment connection at this end only (off; 2.4.4 User Note); the gravity line load **wu** = 1.2D + f1L + 0.15S (1.25 kip/ft) and a further concentrated gravity shear Vg (0) for Vgravity (2.4.4); Ry, Rt (0 = AISC 341-22 Table A3.2) and Cpr (0 = Eq. 2.4-2); a concrete structural slab (off) and, in an SMF with a slab, its isolation from the column (13.2 Exception); the E3.4a moment-ratio exception (off).
- **Beam and column**: rolled W shapes (beam W21×50, column W14×159 by default), their materials, the column axial load Pr and story shear Vc (both 0), whether the column ends at the joint, and the J10.6 panel-zone option.
- **T-stubs**: the W shape both T-stubs are cut from (W21×93), its material (A992; 13.4.1 also permits A913 Gr. 50), the T-stub width **WT** along the column flange (14 1/2 in.; the stem is not tapered), a shim between the stem and the beam flange (0, at most 1/4 in., 13.4.5(1)) and a finger shim between the T-flange and the column (0).
- **Shear bolts**: grade (A490), diameter (7/8 in.), threads (excluded), the number per flange **nvb** (0 = the least even number meeting Eq. 13.5-4 and the Step 15 back-check), spacing svb (3 in.), gage gvb (4 in.), the beam flange

end distance Lehb (1 3/4 in.), the stem end distance (1 3/4 in.), the beam setback **SB** (0 = clear of the T-flange fillet and the tension bolt heads plus 1/2 in.), the stem holes (standard or oversized) and the faying surface class (A; μ enters Eq. 13.5-31).

- **Tension bolts**: grade (A490), diameter (1 in.), **four or eight** per T-stub (eight: two rows of four), the gage **gtb** across the stem (5 in.; gtb / tft ≤ 7.0), the inner gage **gic** across the column flange (5 1/2 in.), the spacing of the outer bolts with eight bolts (3 in.), the T-flange holes (standard, oversized or short slots parallel to the column), and which Step 22 equation checks the column flange (Eq. 13.5-53; Eq. 13.5-59 is the alternative the standard allows).
- **Continuity plates and doubler**: continuity plates are always required (13.4.2). *Designed* sizes them at least tbf thick and to E3.6f.2; *provided* uses your thickness. Width (0 = to the column flange edge less 1/4 in.), fillet size, corner clip along the web (0 = AISC 358-22 3.6). The panel-zone doubler: design when required, none or provided, with its material, thickness, one or two plates, welds and extension.
- **Web single plate**: the design procedure (Manual Part 10 with the eccentricity at the bolts, or direct shear with the T-stubs restraining the rotation), the configuration (auto = conventional when a ≤ 3 1/2 in., else extended; the Step 21 User Note expects an extended plate because of the setback), bolts, spacing, thickness, material, edge distances, the plate-to-column weld (two-sided fillets, two-sided PJP or CJP, 13.4.3), the clearance to the T-flanges, the web bolts and the hole types (standard or short slots parallel to the beam).

## Double-tee checks

The 358-22 13.5 procedure, step by step:

1. **Mpr** = Cpr Ry Fy Zb at the hinge, **Sh** = S1 + Lvb (Eqs. 13.5-5, 13.5-6), Vh (2.4.4), **Mf** = Mpr + Vh Sh (2.4.5) and the beam shear Vu (2.5).
2. Beam flange net section Zb,net Rt Fub ≥ Zb Ry Fyb with the holes in both flanges (Eq. 13.5-1), and the largest hole of Eq. 13.5-2.
3. Shear bolts: φrnv (Eq. 13.5-3), the Step 4 estimate (Eq. 13.5-4) and the back-check nvb φrnv ≥ Ff (Eq. 13.5-37) with Ff = Mf / (db + tst) (Eq. 13.5-36).
4. T-stem: the Step 9 sizing at Fpr = Mf / 1.05db (Eqs. 13.5-7 to 13.5-11), then gross yielding, net fracture and flexural buckling at Ff on the lesser of WT and the Whitmore width (Eqs. 13.5-38 to 13.5-41; AISC 360 E3 above KL/r = 25).
5. Tension bolts and T-flange: the Step 10 and 11 sizing (Eqs. 13.5-12 to 13.5-23) and the Step 17 back-check ntb φT ≥ Ff, φT the least of the plastic mechanism, mixed mode and bolt fracture (Eqs. 13.5-42 to 13.5-52).
6. **Stiffness**: the connection must be fully restrained, Ki ≥ 18 E Ibeam / Lo (Eqs. 13.5-24 to 13.5-35).
7. Bearing and tearout of the shear bolts on the beam flange and the stem, and block shear of both (Steps 18 and 19, φd: ductile modes). The alternate mechanism of Fig. 13.7 is not checked, as the standard says.
8. Beam shear at Vu (Step 20) and the web single plate at Vu through the single-plate design (Step 21), with the beam web block shear of Commentary Fig. C-13.2.
9. Column flange yield lines (Step 22, Eqs. 13.5-53 to 13.5-59), continuity plates (13.4.2 and 341-22 E3.6f.2), the SMF panel zone (E3.6e) and column-beam moment ratio (E3.4a).

Prequalification limits (13.3: beam ≤ W24, ≤ 55 lb/ft, tbf ≤ 5/8 in., column ≤ W36 with a slab or W14 without, width-to-thickness by AISC 341), detailing (13.4: materials, two shear bolts per row, the hole types, gtb / tft ≤ 7, pretensioned bolts, Class A surfaces, shims), spacing, edge distances and tightening clearances, the lateral bracing at the hinge (13.3.1(7)) and the **protected zone** (the T-stubs and the beam to one beam depth past the last shear bolt, 13.3.1(8)) complete the report. Steps 9 to 11 and the Step 4 estimate are shown as recommendations: the back-checks of Steps 15 to 17 at the actual flange force decide adequacy.

## Double-tee drawings

The **elevation** shows the column with its continuity plates and doubler, both T-stubs with the tension bolts through the column flange and the shear bolts through the beam flanges, the beam set back past the T-flanges, the web single plate with its weld symbol, the dimension chain from the column face (setback SB, the beam flange end distance, the shear-bolt pitch, the stem end distance), the gage gtb and the hatched protected zone. The **T-STUB (PLAN)** view gives the stem from above with the shear-bolt pattern (gvb, WT, the bolt rows) over the beam flange; the **T-FLANGE (FROM BEAM)** view, on the drawing sheets, gives the tension-bolt pattern (gtb, gic, the outer spacing, the edge distances). On screen the elevation and the plan are shown. The drawing set (PDF and DXF) has the elevation on the connection's sheet and the plan and T-flange on a second sheet, T-STUB DETAILS.

## Double-tee limits

- Rolled W shapes only: built-up beams and columns and cruciform columns (358-22 2.3) are not designed; the T-stubs are cut from rolled sections as 13.4.1 requires. Tapered stems are not modelled (the stem width is WT).
- Interpretations, stated in the report: Eq. 13.5-19 is printed $b' = b + dtb/2$ while Eq. 13.5-48 gives $b' = b - dtb/2$; ILX uses $b - dtb/2$ throughout. Eq. 13.5-21 is printed with tst for tft. Eq. 13.5-53 is printed with tfc to the first power and is read as $tfc^2$ ($Y_c$ is dimensionless; Eq. 13.5-59 is its square-root form). The "b" of Eq. 13.5-54 is read as bc. Lstem in Eq. 13.5-29 is the stem length from the T-flange to the stem end.
- Fatigue (AISC 360 Appendix 3) is not evaluated for this type.
- No worked example exists in AISC 358-22 or its Commentary; the tests are hand calculations of each equation.

CHAPTER 14

# Designing a splice

ILX designs two kinds of splice:

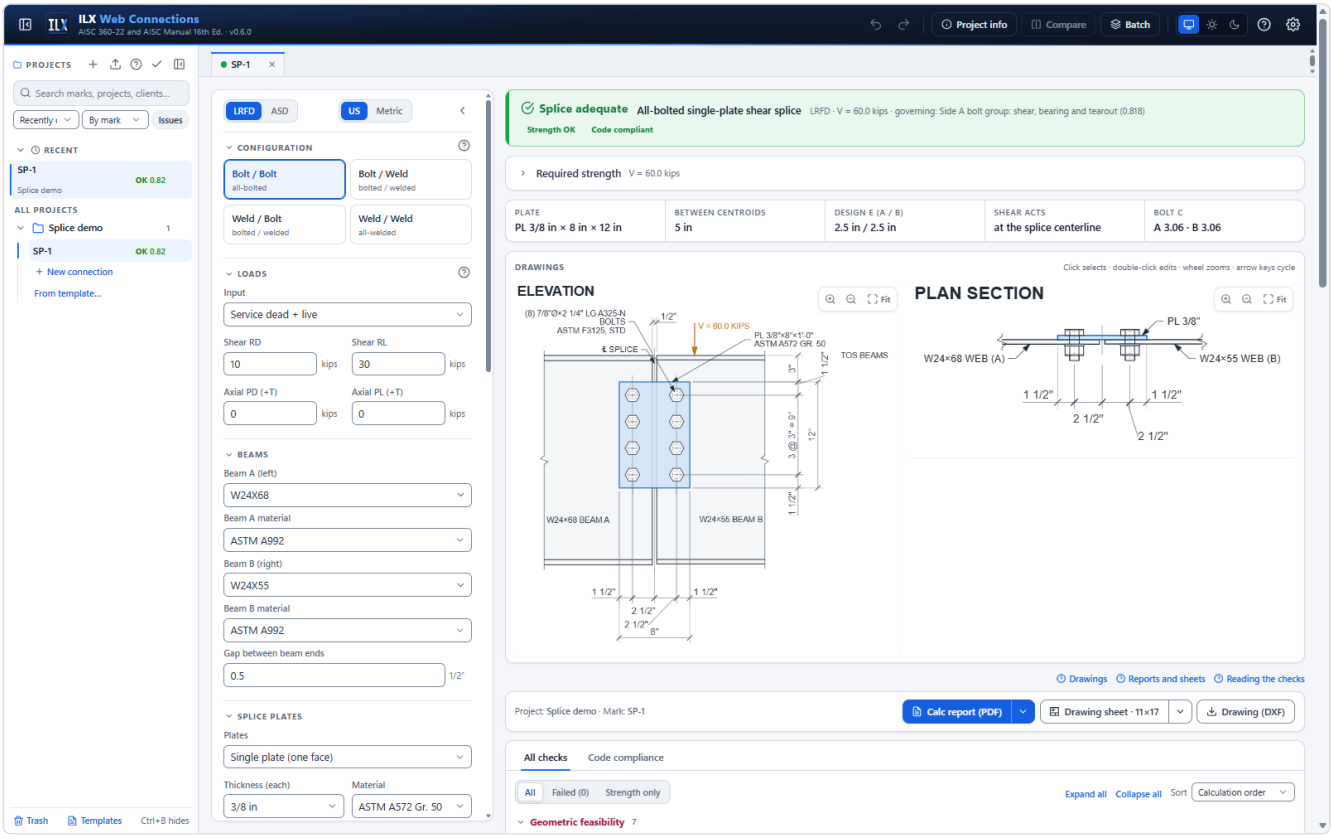
- **Beam shear splice.** One or two plates join the webs of two beams at a point of contraflexure and carry the shear across the gap. Each side is bolted or welded.
- **Compression splice.** Flange plates on the outside of both flanges join two wide-flange members. Use it for a truss chord or strut, or for a column. The ends can be finished to bear or not.

Both editors work like the single-plate editor (see Designing a single-plate connection):

- the design reruns live;
- the status bar shows **OK**, **NG** or **GEOMETRY NOT FEASIBLE**;
- clicking a part in the drawing opens its fields;
- the Checks and Compliance tabs explain every result.

To start one, choose **New connection** and pick **Beam shear splice** or **Compression splice (chord / column)** in the first wizard step.

## Beam shear splice



### Configuration

| Field | Meaning |
|---|---|
| **Configuration** | Bolt / Bolt, Bolt / Weld, Weld / Bolt or Weld / Weld. The first word is side A (beam A, on the left), the second side B. |
| **Splice plates** | **Single plate** on one face of the webs, or **Two plates (both faces)**. With two plates the bolts are in double shear, and each plate is welded on its own face. |
| **Gap between beam ends** | The clear distance between the beam ends. The splice centerline (℄ SPLICE in the drawing) is the middle of the gap. |
| **Cb, plate lateral-torsional buckling (F11)** (Splice plates) | Your input, default 1.84. The plate flexure check uses AISC 360-22 F11 with Lb = the gap. Neither the Manual (shear splices, pp. 10-68 and 10-69) nor Design Examples II.A-20 and II.A-21 (which use Mn = Fy Zx) give a Cb for a splice plate. 1.84 is the single-plate value of Manual Eq. 10-8. The report shows "Cb = x (engineer input)". |
| **Beam A / Beam B** | The two beams and their materials. Their tops are level. The beams may differ, as in AISC Design Example II.A-20 (W24X68 to W24X55). |

### Loads on the shear splice

Enter the shear across the splice and, if there is one, the axial force (+ tension). You can enter it in any of these forms:

- service dead and live loads
- a required strength
- ASCE 7 combinations
- your own load cases

The splice is assumed to sit at a point of contraflexure, so the moment at the splice centerline is zero.

### Each side

A **bolted side** has:

- **Bolt rows** (shared by both sides), spacing **s** and the plate's vertical edge distance **Lev**;
- the number of **bolt columns** and their **gage**;
- **Centerline to first bolt column**. The beam-end edge distance is this value minus half the gap.
- **Plate edge distance** beyond the last column.

A **welded side** has:

- the **lap** of the plate on the web, measured from the beam end;
- the **weld** size;
- the **weld shape**: C-shape, vertical only, or top and bottom.

The welds start at the beam end.

### Eccentricity

The shear does not pass through either connection, so each side is designed for a moment V × e. **Eccentricity carried by** sets how that moment is shared:

| Setting | Each side is designed for | Basis |
|---|---|---|
| **Shear at the splice centerline** | V at e = its centroid's distance from the centerline. On a symmetric splice this is half the distance between the groups. | Kulak and Green (1990); Design Example II.A-20 |
| **Side A full / Side B full** | That side takes the full distance between the two centroids; the other side takes shear only. | Design Example II.A-21: the more rigid weld group takes the full moment |
| **Both sides full (conservative)** | The full distance on both sides. | Conservative bound |
| **Automatic** (default) | Bolted / bolted: centerline. Bolted / welded: the welded side full. Welded / welded: both sides full. | |

The plate is checked for flexure with the largest of these moments, as both AISC examples do.

### What the shear splice checks

| Element | Checks |
|---|---|
| Bolt groups | Instantaneous-center coefficient C at the design eccentricity and load angle. For each bolt, the least of shear, bearing and tearout in the plate(s) and the web. The shear is checked in both directions. Slip resistance applies when the joint is slip-critical. |
| Weld groups | Instantaneous-center strength of the weld shape at its eccentricity. Web and plate thickness needed to develop the weld (Manual Part 9). |
| Splice plate | Shear yielding and rupture, block shear, flexural yielding and rupture. Under axial load: tension yielding and rupture, axial block shear, compression buckling, and the Manual Part 12 interaction. |
| Beams | Web shear. If coped: net shear rupture, block shear, local flexural strength at the cope and rupture through the holes. Under axial tension: axial block shear, tension yielding and rupture. |
| Detailing | Spacing, edge distances (plate, beam ends, copes), maximum edge distance and spacing, wrench clearance to the flanges, hole and slip-critical rules, washers, weld sizes and lap, cope radius. |

### Copes

Turn on a top and/or bottom cope and choose which beam ends are coped (**Coped beams**: both, A or B). The cope length is measured from the beam end.

The coped-beam moment is taken from the cope face to the line of action of the shear. That line of action is never taken closer to the beam than the splice centerline.

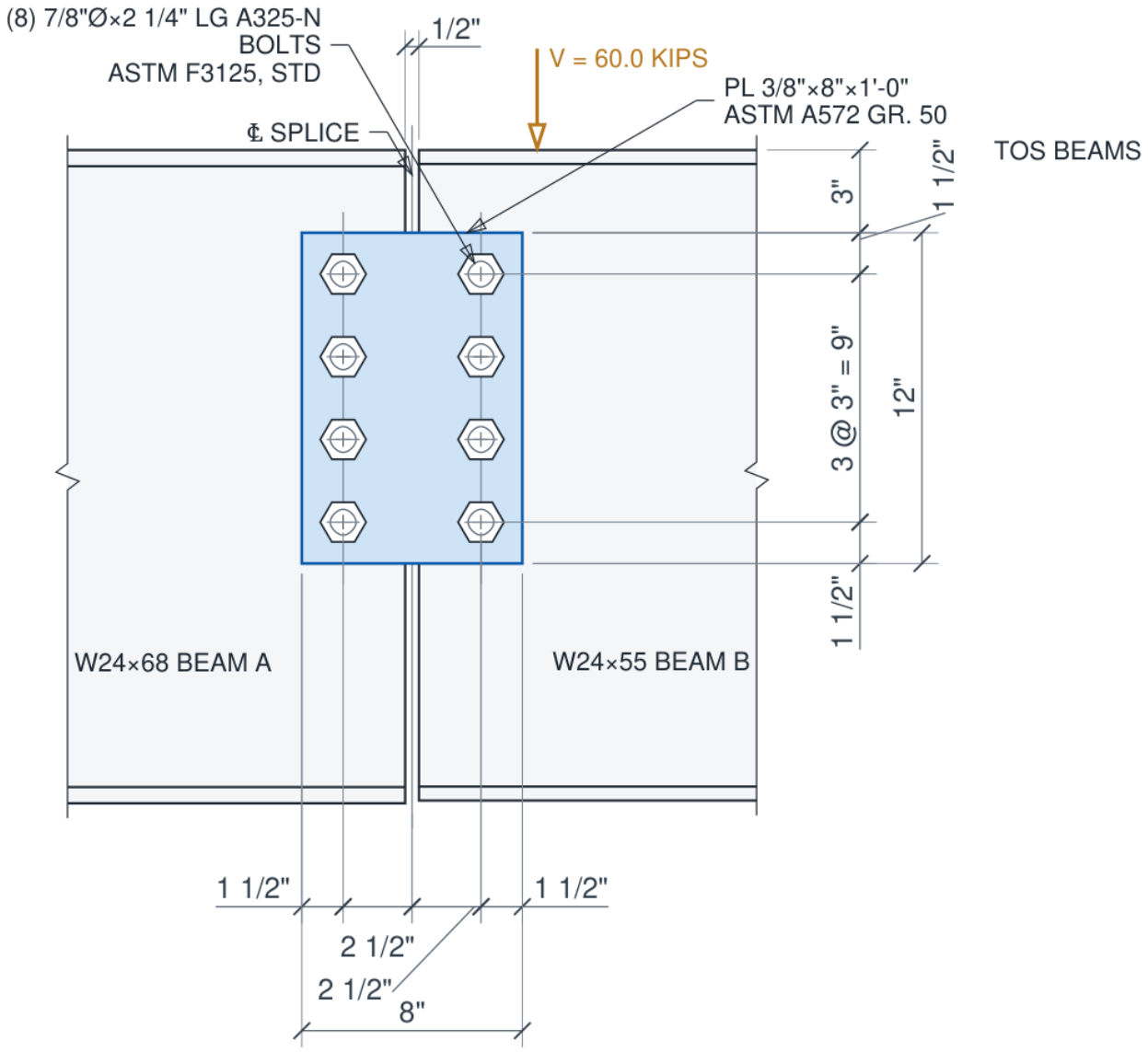
### Geometry not feasible

The design is marked NOT FEASIBLE when any of these hold:

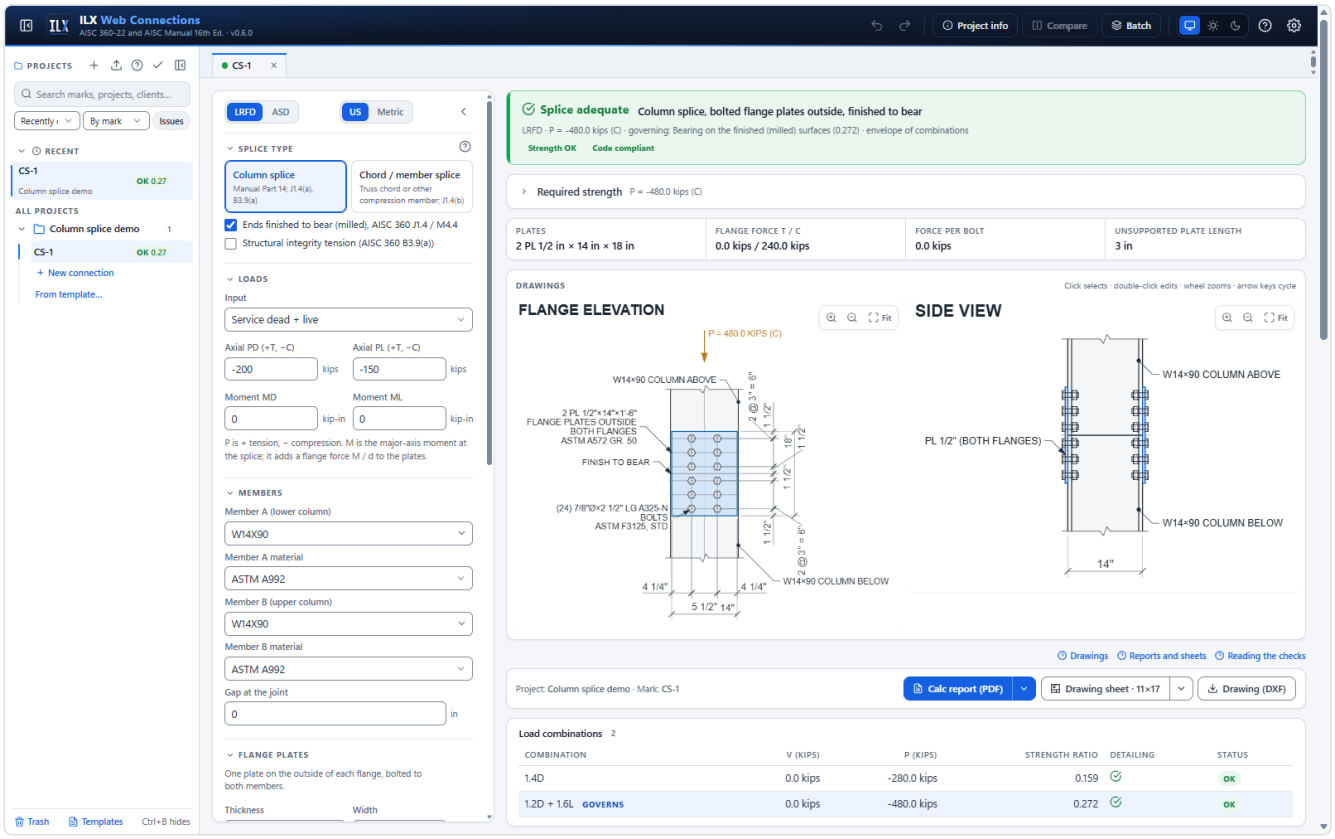
- the beam ends touch;
- the plate runs into the flange fillets or a cope;
- a bolt lies off the plate or the web;
- holes overlap;
- a cope removes the whole web;
- a weld has no web to sit on.

The message names the fix.

# ELEVATION


(8) 7/8"Ø×2 1/4" LG A325-N
BOLTS
ASTM F3125, STD
1/2"
V = 60.0 KIPS
PL 3/8"×8"×1'-0"
ASTM A572 GR. 50
℄ SPLICE
TOS BEAMS
3"
1 1/2"
3 @ 3" = 9"
12"
1 1/2"
W24×68 BEAM A
W24×55 BEAM B
1 1/2"
1 1/2"
2 1/2"
2 1/2"
8"

# Compression splice (column or chord)




## Splice type

| Field | Meaning |
|---|---|
| **Splice type** | **Column**, a Manual Part 14 column splice, or **Chord / strut**, a compression member other than a column. |
| **Finished to bear** | The member ends are milled and in contact. Compression then passes in bearing, and the plates carry tension and hold the parts in line. When this is off, the plates and bolts carry the whole force, as in Design Example II.C-3. |
| **Member A / Member B** | The two members. When their depths differ, ILX places a filler of half the depth difference between the plate and the shallower member's flange. |
| **Gap at the joint** | 0 for a bearing joint. Up to 1/16 in. is permitted and gaps up to 1/4 in. are shimmed (AISC 360 M4.4). A larger gap is not feasible for a bearing splice. |

## Loads on the compression splice

Enter the axial force (+ tension) and the moment about the major axis at the splice, in one of these forms:

- service dead and live loads
- a required strength
- load cases

Each flange carries F = P/2 ± M/d, where d is the shallower member's depth.

For a **column** finished to bear:

- the plates are designed for the net flange tension from the load cases. A dead load that offsets the moment tension is taken into account.
- turn on **Structural integrity** and enter the nominal D + L tributary to the column below this splice to check AISC 360 B3.9(a). That check uses nominal strengths only.
- choose a **Seismic column splice** for a column of a seismic force-resisting system (AISC 341-22):
  - **SFRS column: D2.5**. Enter the loads of the overstrength seismic load where they govern (D2.5b(a)). With the **distance to the beam flange connection** ILX checks the 4 ft rule of D2.5a (a clear height under 8 ft puts the splice at half of it). With the **story height H** the report gives the D2.5c shear Mpc / (αs H) about both axes. The flange plates carry no shear: splice the web with plates on both sides (D2.5d).
  - **Braced frame** (SCBF, EBF, BRBF; F2.6d, F3.6d, F4.6d) adds a load case M = 0.5 Mp / αs of the smaller member with P = 0, and the shear Σ Mp / (αs Hc) with the **clear height Hc**.
  - **Special moment frame** (E3.6g.5, bolted splices) adds a load case M = Ry Fy Zx / αs of the smaller column (Ry from Table A3.2) and the web splice shear Σ Mpc / (αs Hc).
  - Every seismic splice checks the D2.2 bolted-joint rules: standard holes or short slots across the load, and pretensioned bolts with Class A (or better) faying surfaces.

For a **chord** finished to bear:

- enter the member length and the splice location;
- the flange plates are designed for the lesser of 50% of the compression as tension and the moment from a transverse load of 2% of the compression at the splice, with the member pinned (AISC 360 J1.4(b));
- the transverse shear is reported. Web splice plates are not designed.

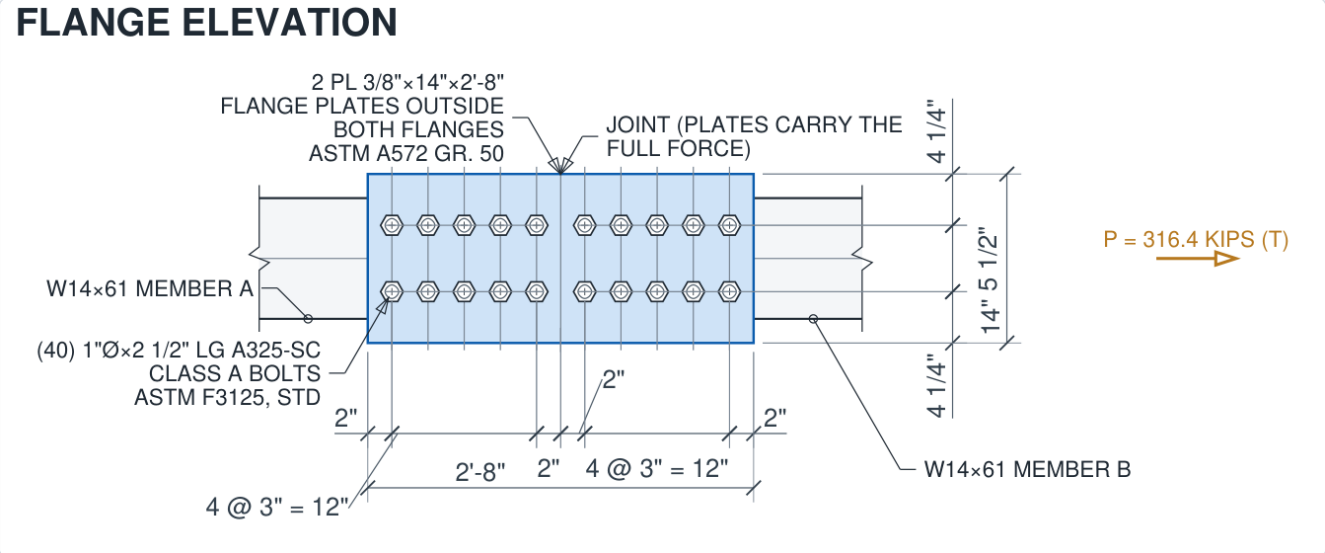
## Flange plates and bolts

| Field | Meaning |
|---|---|
| **Plate thickness, width, material** | One plate on the outside of each flange. |
| **Bolt lines per flange** | 2, or 4 (with **Outer line spacing**). **Inner gage** is the distance between the two lines next to the web. |
| **Side A / B bolt rows**, **spacing** | Rows along the member on each side of the joint. |
| **End distances** | Plate end to the last row on each side, and each member end to its first row. |
| **Effective length factor K** | For the plate in compression between the rows nearest the joint. The default is 0.65, as in AISC Design Example II.B-1. |
| **Fillers developed** | Turn this on when the fillers are extended and developed. Otherwise fillers thicker than 1/4 in. reduce the bolt shear (AISC 360 J5.2). |
| **Gusset section along the member, l** and **Holes on that section** | Chord / strut only. Use them when the flange plates are the gusset plates of a truss panel point, as in Design Example II.C-3. **l** is the length of each plate's section along the member edge, through the bolt line; **holes** is the number of bolt holes on that line. ILX then checks the shear yielding and shear rupture of each plate on that section. Leave l at 0 for an ordinary splice. |

### What the compression splice checks

| Element | Checks |
|---|---|
| Bolts (per flange, each side) | The sum of each bolt's least of shear, bearing and tearout. Tearout runs toward the plate and member ends in tension and between the holes in compression. Slip resistance applies for slip-critical joints. |
| Flange plates | Yielding on the Whitmore section, gross yielding, net rupture, block shear (inner and outer blocks), and compression buckling across the joint. |
| Gusset section (chord, when l > 0) | Shear yielding, 0.60 Fy l t, and shear rupture, 0.60 Fu [l − n (dh + 1/16 in.)] t, of each plate along the member (AISC 360 J4.2), for the member force each plate carries. |
| Members | Flange block shear and tensile rupture at the holes (shear lag). |
| Bearing | Bearing on the milled surfaces (1.8 Fy Apb), a check that the flanges line up (otherwise a butt plate is needed), and the fit of the joint. |
| Detailing | Spacing and gage, plate and flange edge distances, maximum edge distance and spacing, clearance of the inner bolt line from the web fillet, hole and slip-critical rules, washers, and the erection load note for columns. |

AISC Manual Table 14-3 gives standard column splice details. Use them as the minimum for ordinary gravity columns.




## Verification of the splice examples

Both splice types reproduce the AISC Design Examples end to end, in LRFD and ASD:

- II.A-20, the all-bolted shear splice;
- II.A-21, the bolted and welded shear splice;
- II.C-3, each member-to-plate joint of the heavy wide-flange compression connection.

The comparison tables and the documented differences are on the Verification page (see Verification).

- **II.A-21 weld length:** AISC measures the short welds from the splice centerline; ILX uses the weld that can actually be placed, measured from the beam end. This is conservative, and with AISC's idealization ILX reproduces every published value.

- **II.C-3 gusset shear:** the shear yielding and rupture of the gusset plates along the chord (l = 32 in., 5 holes) and along the vertical (l = 31¾ in., 7 holes) are reproduced with the gusset section inputs. For the vertical, ILX gives 514 kips where the example prints 515, because the example rounds the net area.

CHAPTER 15

# Designing a truss connection

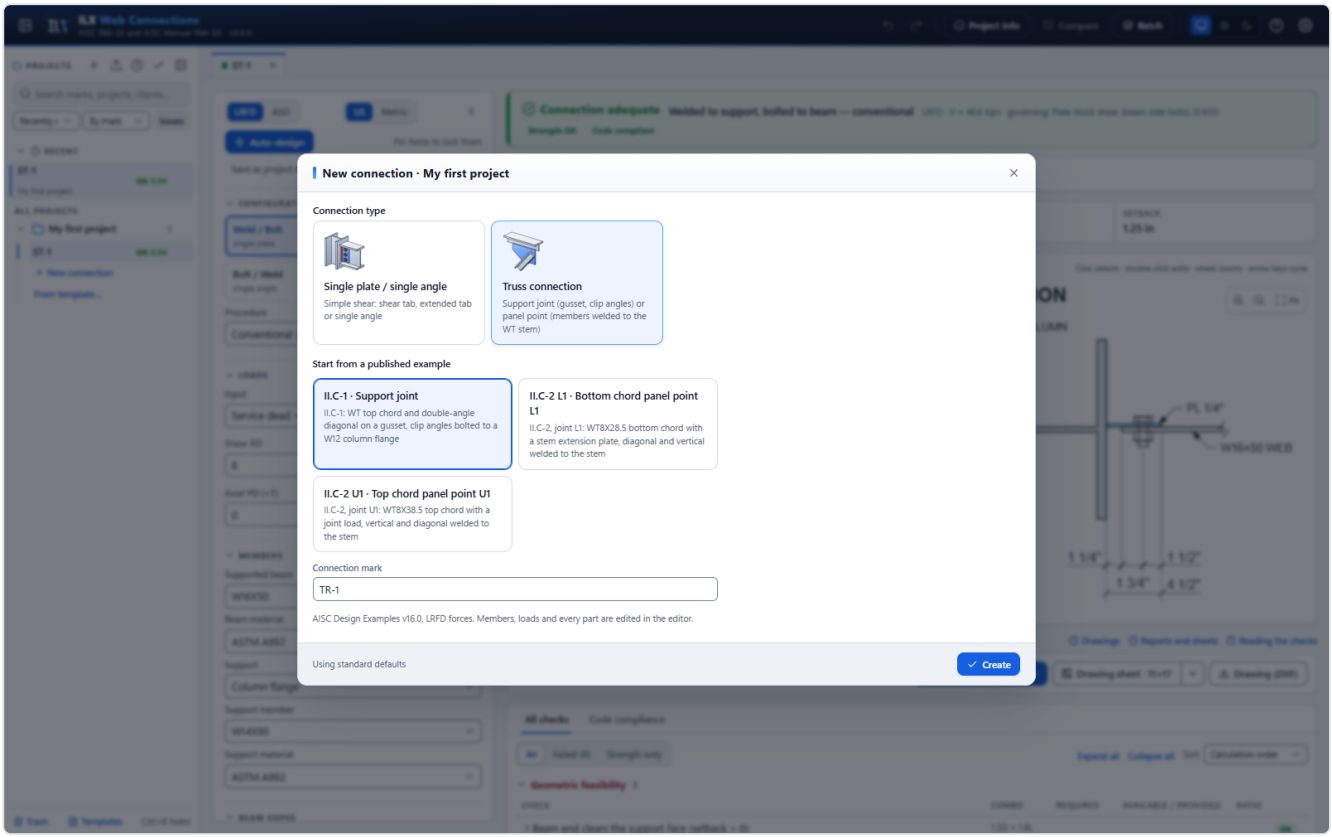
## What the truss connection covers

The truss connection type designs the two joints of AISC Design Examples v16.0, Chapter IIC, for AISC 360-22 and the AISC Manual, 16th Ed. (Parts 7, 8, 9 and 13):

| Joint | Example | What it is |
|---|---|---|
| **Support joint** | II.C-1 | A WT top chord framing into a column flange. A gusset plate is welded (CJP) to the chord stem, the double-angle diagonal is fillet-welded to the gusset, and two clip angles are welded to the gusset and bolted to the column flange. |
| **Panel point** | II.C-2 (joints L1 and U1) | Double-angle (or single-angle) web members fillet-welded at heel and toe directly to the WT chord stem, optionally on a stem extension plate CJP-welded to the stem. |

The editor designs the joint live, like every other type. Every change re-runs the full design and updates the status, the drawings and the checks.

1. **Create it.** In the file manager, click **New connection**, choose **Truss connection**, pick one of the published examples to start from, and click **Create**. New marks are numbered TR-1, TR-2 and so on.
2. **Pick the joint.** Choose **Support joint** or **Panel point**. The example buttons (**II.C-1**, **II.C-2 L1**, **II.C-2 U1**) reload a published joint at any time.
3. **Enter the loads**, the chord and the web members, then the parts of the joint.
4. **Read the status bar** and fix what fails (see Checks and compliance).
5. **Produce the outputs**: the calculation report and the drawing sheet (PDF and DXF).




## The joint and the working point

All geometry is measured from the **working point (W.P.)**. At a panel point the W.P. lies on the chord work line, which is the centroid line of the WT. At a support joint it lies on the column centerline (II.C-1) or, if you choose, on the column face.

- **W.P. at the column centerline.** The reaction is eccentric to the bolts on the column flange by the column half-depth, e = d/2. The bolts then take tension as well as shear (see Bolts).
- **W.P. at the column face.** e = 0 and the bolts take shear only. Move the diagonal end out by the same amount so the welds stay on the gusset.

The column moment from the support eccentricity is reported as a note (see Manual Part 13 truss items).

## Loads at a truss joint

| Field | Meaning |
|---|---|
| **Input** | **Required strength**: forces entered directly as Pu (LRFD) or Pa (ASD). **Service D + L**: dead and live components, combined as max(1.2D + 1.6L, 1.4D) for LRFD or D + L for ASD. |
| **Reaction R** (support joint) | Truss end reaction, upward. |
| **Chord force H** (support joint) | Axial force in the top chord at the column, + compression toward the column. If H is tension, the bolts also take it (equally per bolt). |

| Field | Meaning |
|---|---|
| **Joint load** (panel point) | Load applied at the W.P., + downward (II.C-2 joint U1: 30 kips LRFD, 20 kips ASD). |
| **Member force P** | In each web member, + tension, − compression. |

ILX checks equilibrium of the joint as an advisory. A support joint is checked with the diagonal, the reaction and the chord force. A panel point is checked perpendicular to the chord; the difference in chord force carries the rest. An unbalance above 5 % is flagged so you can catch a sign or a typing error.

## The chord

- **Chord (WT)**: any WT from the AISC Shapes Database v16.0, with its material.
- **Chord position**: **Top** (stem down) or **Bottom** (stem up). A support joint uses a top chord.
- **Chord slope**: rise in 12, + rising to the right (II.C-1: ½ in 12). The chord, its flange, stem and k-line are drawn on the slope.

Welds and member legs must stay on the stem below the k-line. A member that runs into the chord flange makes the joint **NOT FEASIBLE**. One that only reaches into the flange-to-stem fillet (the k-area) is flagged under Code compliance.

## Truss web members

Each web member is a double angle (`2L…LLBB`, `2L…SLBB` or an equal-leg `2L`) or a single angle (`L…`, choosing which leg lies on the plate). A support joint uses one member, the diagonal. A panel point takes up to four.

| Field | Meaning |
|---|---|
| **Bevel rise dv : run dh** | The member direction, as on the drawings. dv rises toward the stem side (into the truss); dh runs to the right (negative to the left). A vertical is 1 : 0. |
| **W.P. to member end** | Distance along the work line from the W.P. to the end of the angles. Negative values are allowed: the member then starts behind the W.P. |
| **Heel to work line** | Gage from the heel to the work line. Leave it blank to use the centroid (II.C-1 uses the detailed 1¼ in.). |
| **Heel side** | Which side of the member the heel is on, looking along the member away from the W.P. |
| **Weld size, heel and toe lengths** | Fillet welds along the heel and the toe of each angle, starting at the member end. Unequal lengths are allowed (II.C-2 U1L2: 7½ in. heel, 4 in. toe). |

What is checked for each member:

- **Weld strength**: φRn = n × 0.60 FEXX × 0.707 w × (l_heel + l_toe), with kds = 1.0 for load along the weld and β for welds longer than 100w (J2.2b(d)).
  - The calculation also prints the required length per angle and per weld line, as the examples do.
  - Balancing the welds about the member's centroid is not required for statically loaded angle members (J1.7).
- **Weld size limits**: minimum from Table J2.4 (thinner part joined), maximum t − 1/16 in. along the angle edges, and minimum length 4w.
- **Base metal**: angle tmin = 3.09 D/Fu (Manual Eq. 9-6). Gusset or chord stem tmin = 6.19 D/Fu (Eq. 9-7), because the angles on both faces share the plate.

- **Tension members**:
  - yielding, Fy Ag;
  - rupture, Fu U Ag, with U from Table D3.1 Case 4: $U = 3l^2/(3l^2 + w^2)(1 - \bar{x}/l)$, where l is the average weld length and w the connected leg;
  - block shear of the gusset or stem around the welds.
- **Compression members**: the member's own buckling belongs to the truss design. At a support joint the gusset is checked as a column strip on the Whitmore section (see below).

## Support joint: column

Choose the column (W shape) and its material. The clip angles are bolted to its flange. ILX checks the following:

- **Prying.** Prying on the column flange, with the same Manual Part 9 procedure as the angles, using tf, b = (g − tw)/2 and p = s.
- **Bearing and tearout.** Bearing and tearout on the flange between holes.
- **Bolt fit.** The bolts must fit on the flange, and the holes must clear the web-to-flange fillet (g/2 − dh/2 ≥ k1).

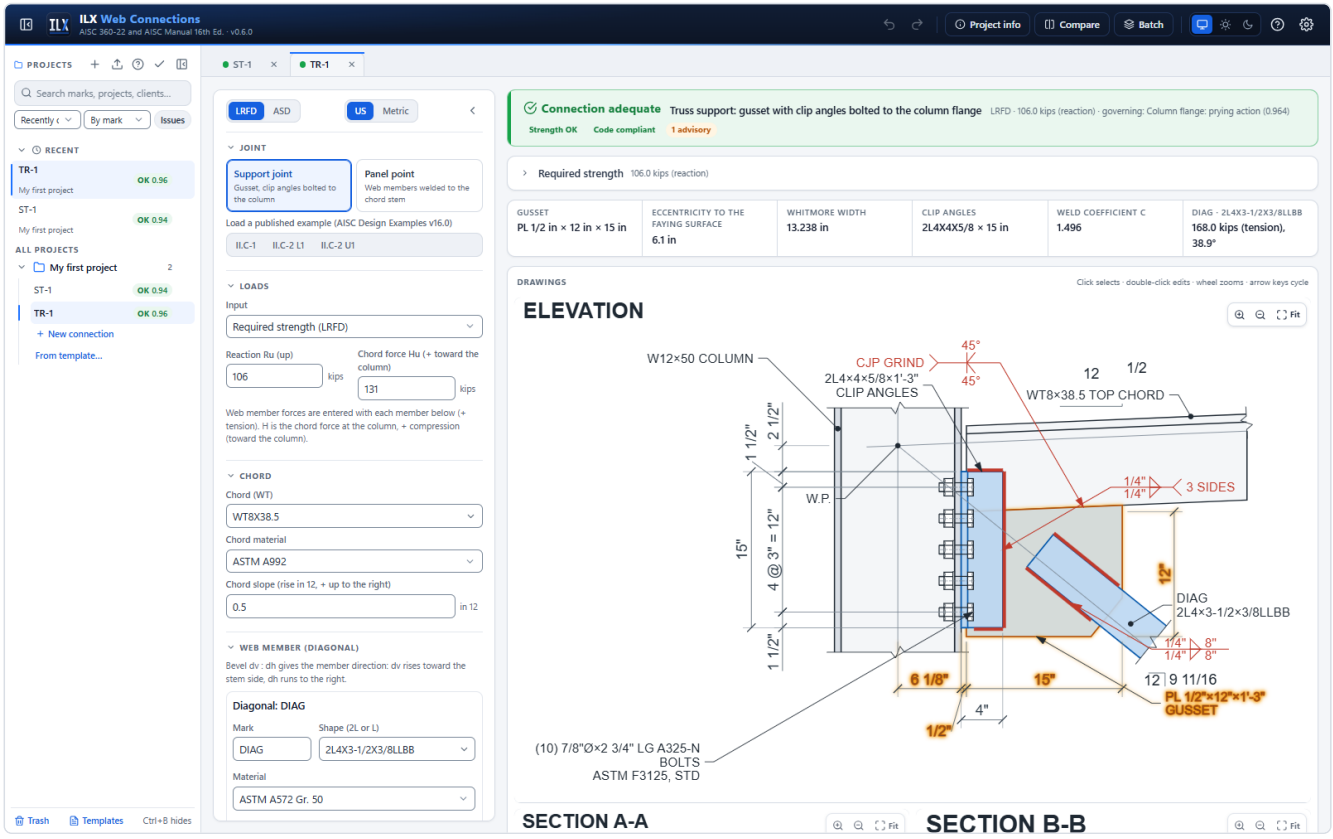
## Support joint: gusset plate

| Field | Meaning |
|---|---|
| **Gusset thickness, material** | II.C-1: PL ½ in., A572 Gr. 50. A recommendation flags a gusset thinner than the chord stem. |
| **Width** | Along the chord, from the setback edge (II.C-1: 1'-3"). |
| **Depth below the stem** | Measured at the column edge (II.C-1: 12 in.). |
| **Setback** | Column face to the gusset and the chord end (½ in.). |
| **Edge beyond the welds** | The corner below the diagonal is cut square to the diagonal, this far past the end of its welds. |
| **Whitmore width** | **Full** (Manual Fig. 9-1, as the example) or **Clipped** (only the part within the gusset and the CJP-welded chord stem). |
| **Gusset buckling K and length** | **DG29 Table C-1** (default) or **Given K with the average length**. With the DG29 setting the support gusset is a corner gusset (see below). |
| **K (compression)** | Shown for **Given K** only: the effective length factor with the Thornton average length (0.5, the earlier ILX default). |

Gusset checks:

- **Whitmore section.**
  - $l_w = w + 2\, l \tan 30°$, and $A_w = l_w\, t$.
  - Tension: yielding, Fy Aw.
  - Compression (AISC Design Guide 29, App. C, Table C-1, after Dowswell 2006): the support gusset is a corner gusset. ILX measures c, the distance from the end of the diagonal's welds to the edge that the centre line meets (DG29 Fig. C-4). If $t \geq t_\beta = 1.5\sqrt{(F_y c^3 / (E\, l_1))}$ the gusset is compact and only yielding applies, Fy Aw. Otherwise it is checked as a column with K = 1.0 and the average $\bar{L}$ of three lines from the Whitmore section back to the gusset edge (Thornton). J4.4: Fy Aw when KL/r ≤ 25; Chapter E otherwise. With **Given K**, K × $\bar{L}$ is used as before.

- **Whitmore section within the plate (advisory).** It reports how much of lw lies inside the gusset and stem. In II.C-1 only part of the 13.2 in. width lies inside the published gusset outline. The example, like Manual Fig. 9-1, uses the full width, so ILX does too by default. Choose **Clipped** for the conservative check.
- **Shear yielding and rupture** of the vertical section through the gusset and the stem at the clip-angle toe.
- **Gusset-to-stem CJP.**
  - Interface forces, as in Design Example II.C-1:
    - compression between the WT and the column, C = n′ rut;
    - horizontal shear H = Hchord − C;
    - vertical tension V = R n′/n;
    - moment about the column face, M = C (w.p. to the top bolt) + H (d − ȳ) − V (width/2 + setback).
  - The gusset base metal at the CJP is then checked for the normal stress V/(tL) + 6M/(tL²) and the shear stress H/(tL).
- **Feasibility.**
  - The diagonal's welds must lie on the gusset or stem.
  - The diagonal must clear the clip angles and the chord flange.
  - The chamfer must leave a usable gusset.



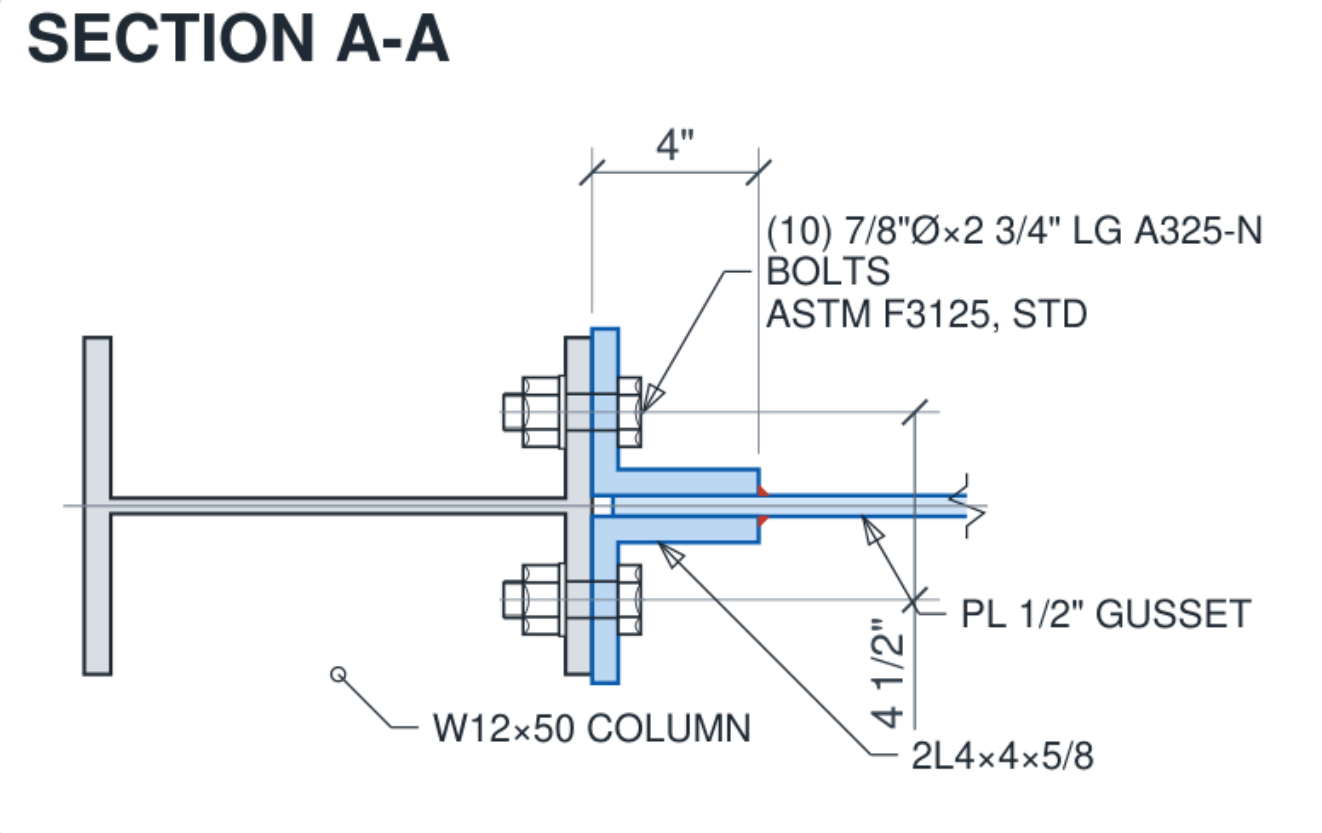

## Support joint: clip angles

Two angles (II.C-1: 2L4×4×⅝ × 1'-3"), one on each face of the gusset, with one leg welded to the gusset and stem and the other bolted to the column flange.

| Field | Meaning |
|---|---|
| **Clip angles** | The angle size (one angle, or the 2L designation), with its material. |
| **Leg on the column** | For unequal legs: which leg is bolted to the column. |
| **Length** | Angle length (II.C-1: 15 in.). |
| **W.P. to top of the angles** | Vertical distance from the horizontal line through the W.P. (II.C-1: 2½ in.). |

Checks:

- **Shear yielding and rupture.** Shear yielding 0.60 Fy Agv (Agv = 2 l t), and shear rupture 0.60 Fu Anv with the holes at dh + 1/16 in. (B4.3b).
- **Block shear.** Shear along the bolt line from the bottom edge, and tension from the bolts to the angle toe.
- **Prying** (Manual Part 9, 16th Ed. with Fu):
  - a is the lesser of (bf − g)/2 on the column flange and the bolt-to-toe distance on the angle;
  - b = (g − tg − t)/2;
  - p = l/n, but not more than s;
  - then δ, tc, α′ and Q, and the available tension per bolt Q Tc ≥ rut.
- **Feasibility.** The angles must fit between the chord flange (clear of the k-area) and the bottom of the gusset.



## Support joint: bolts

Bolt grade, diameter, threads (N or X), hole types in the angles and the column, bearing-type or slip-critical joint (Class A or B), and **hole deformation at service** (bearing 2.4 / tearout 1.2, or 3.0 / 1.5). The pattern has n rows of two bolts (one per angle) at spacing s, a top edge distance lev and a gage g across the column flange.

- **Shear, bearing and tearout.**
  - The group strength is the sum of each bolt's least strength (J3.7 Commentary, as II.C-1).
  - Each bolt takes the least of: bolt shear (with the long-joint factor above 38 in.); bearing and tearout of the angle, where the top bolt's lc = lev − dh/2 and the interior bolts' lc = s − dh; and bearing and tearout of the column flange.
- **Tension with shear** (Manual Part 7, Case II).
  - The neutral axis is at the bolt-group centroid.
  - rut = R e/(n′ dm), where n′ is the number of bolts above the neutral axis and dm is the moment arm between the tension and compression resultants (II.C-1: 4 bolts, 9.00 in.).
  - The tension is combined with the shear stress frv by J3.8: F′nt = 1.3 Fnt − (Fnt/φFnv) frv ≤ Fnt.
- **Slip-critical joints.** μ Du hf Tb ns ksc, with ksc reduced for the bolt tension (J3.9).
- **Detailing.**
  - Spacing: at least 2⅔d, preferably 3d, and at most 24t or 12 in.
  - Edge distances (Table J3.4) to the angle ends, to the angle toe and to the column flange tip; maximum edge distance.
  - Tightening clearance from the gusset face (Manual Tables 7-15/7-16).
  - The column fillet.
  - Slip-critical bolts are required for oversized or slotted holes parallel to the load.

## Support joint: clip-angle weld

A fillet weld on each angle, C-shaped. It runs along the toe of the angle leg on the gusset (the full angle length), with returns along the top and bottom of the leg. With a sloped chord the two returns differ (II.C-1: 3⅜ in. and 2¾ in.).

- **Average the returns** (default, as the example): kl = (top + bottom)/2. Turn it off to solve the true unequal shape.
- **Strength.**
  - ILX solves the weld group by the instantaneous-center method (Manual Part 8, the geometry of Table 8-8, angle 0°).
  - The load is the reaction R/2 per angle, at al = e + (leg on the gusset) − xl from the weld centroid.
  - It reports C, k = kl/l, xl, a = al/l, the minimum weld size Dmin and the capacity 2 C C1 D l.
- **Limits.**
  - Minimum and maximum fillet sizes.
  - The gusset must be thick enough to develop the welds on both faces (6.19 D/Fu).
  - The returns must fit on the part of the angle leg over the gusset; if they don't, the joint is NOT FEASIBLE.

# Panel point: stem extension plate

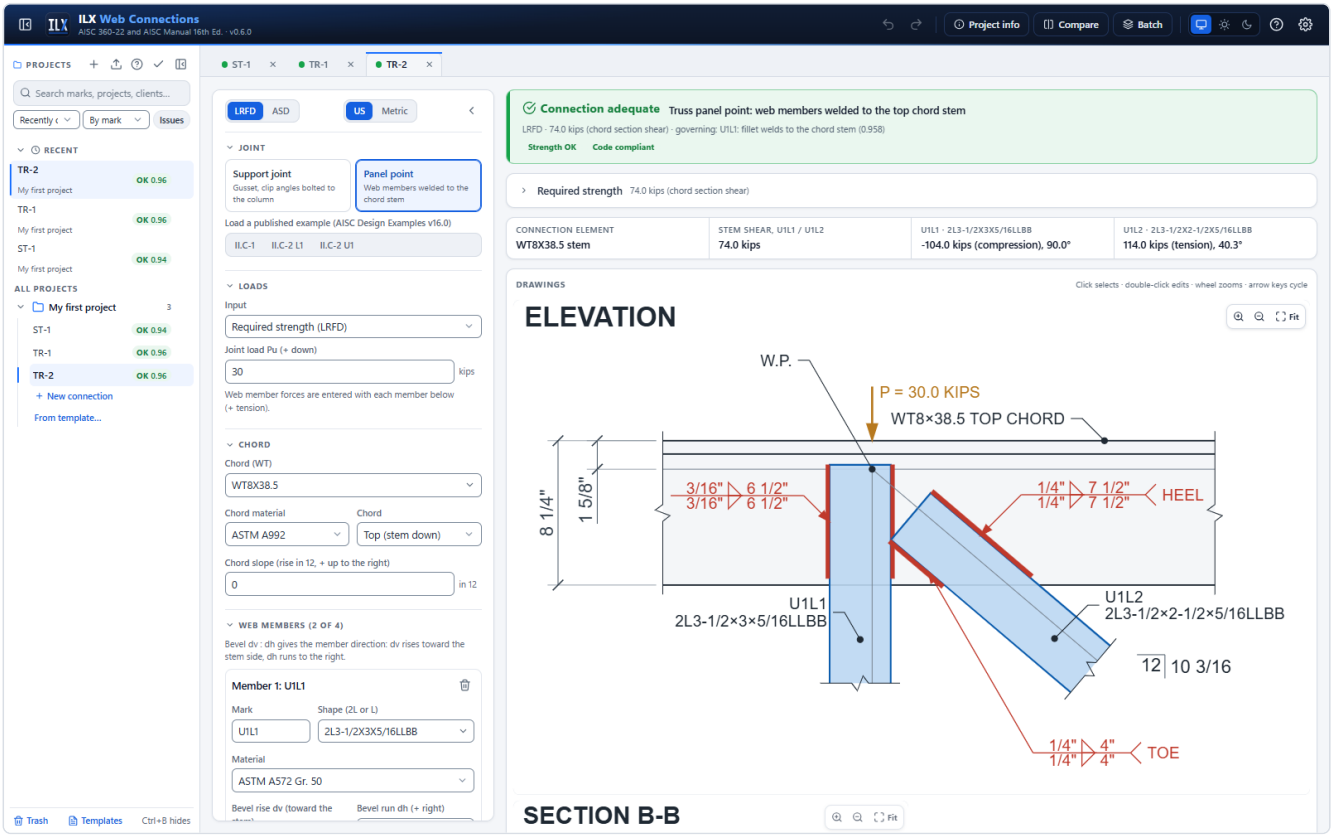
When the welds of a web member do not fit on the chord stem, add a stem plate CJP-welded to the stem tip. II.C-2 joint L1 uses PL 7/16 × 4 × 1'-10. Enter its thickness, its depth beyond the stem tip, the start of the plate along the chord (from the W.P.) and its length.

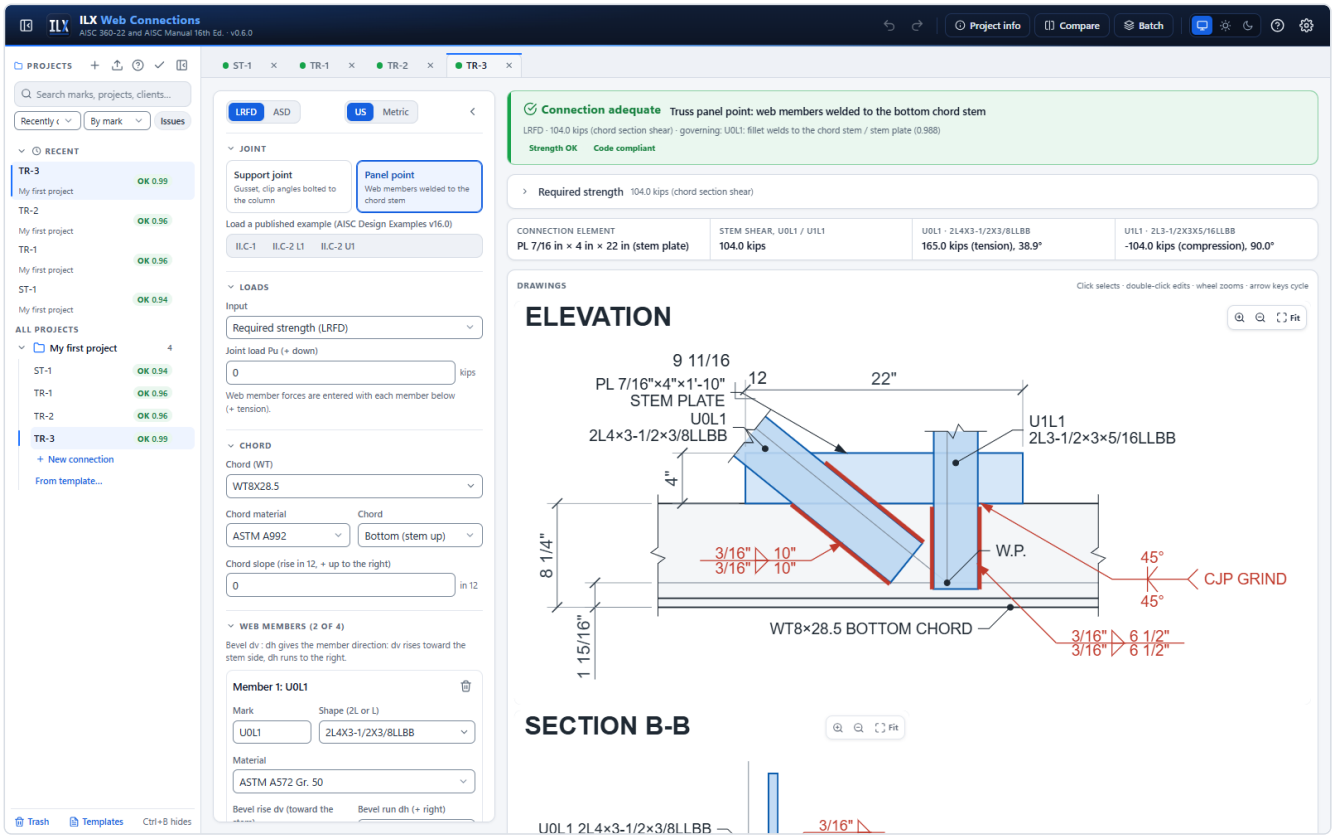
- Welds may run onto the stem plate. The weld base-metal and block-shear checks then use the thinner of the stem and the plate.
- A recommendation flags a plate thinner than the stem.
- Without the plate, the 10 in. welds of U0L1 at L1 do not fit on the stem, and the joint is **NOT FEASIBLE**. That is why the example adds the plate.

## Chord stem shear at a panel point

ILX finds the critical sections between adjacent members, ordered along the chord by their weld centroids. At each section it sums the force components perpendicular to the chord on each side of the cut, including the joint load at the W.P. The larger side governs.

The required shear is checked against shear yielding of the stem, 0.60 Fy d tw, with d the full depth of the WT (J4.2(a)). In II.C-2 this gives 104 kips at L1 (Section A-A) and 104 − 30 = 74.0 kips at U1 (Section B-B), as published.






## Panel point with a gusset plate

Instead of welding the web members to the chord stem, you can connect them to a gusset plate on the stem.

1. Choose **Panel point**, then under **Chord** set **Members connect to** to **A gusset plate on the stem**.
2. Enter up to three web members (for example diagonal, vertical, diagonal).
3. For each member choose **Welded** (heel and toe fillets, as above) or **Bolted**.
4. Set the gusset in the **Gusset plate** section.

The four buttons under **Load a panel-point gusset configuration** load worked setups: three welded members, three bolted members, one bolted and one welded member on a sloped chord, and a single member on a bottom chord with a lapped gusset. They are not published examples.

### Gusset size

- **Automatic** (default). ILX sizes a rectangle along the chord that holds every member's connection (plus the edge distance beyond it) and the ends of each Whitmore section. It rounds the size up to ½ in. The editor shows the computed start, length and depth; **Use computed** copies them into the fields and turns automatic sizing off.
- **Given.** Turn automatic sizing off and enter the start (from the W.P., along the chord), the length along the stem and the depth beyond the stem tip.
- **Chamfers.** The corners are cut square to each diagonal at **Connection end to the gusset edge** beyond its welds or last bolt. A chamfer is skipped if it would cut into another member's connection.
- **Gusset to stem. CJP groove weld** (a butt joint on the stem tip) or **Lapped on the stem** with fillet welds along both edges (enter the fillet size and the lap).

## Bolted members

Enter the bolts per line, one or two bolt lines, the spacing, and the distance from the angle end to the first bolt. The gage is the workable gage of Manual Table 1-7A unless you turn that off and enter it. Bolt grade, diameter, threads, holes and joint type are set once in the **Bolts** section. Double angles put the bolts in double shear.

## What is checked

For each member:

- **End connection.** Welds as for a stem connection. Bolts: shear, plus bearing and tearout on the angles and on the gusset, summed bolt by bolt (J3.7 Commentary). Tearout uses the angle end in tension and the gusset edge beyond the last bolt. Slip resistance is checked for slip-critical joints.
- **Tension.** Yielding, Fy Ag. Rupture, Fu U An: Table D3.1 Case 4 for welds, and the larger of Case 2 ($1 - \bar{x}/l$) and Case 8 (0.80 with four or more bolts per line, 0.60 with three) for bolts.
- **Block shear** of the angle legs (bolted) and of the gusset around the welds or bolts (J4.3).
- **Whitmore section.** $l_w = w + 2\,l \tan 30°$, counted only where it lies on the gusset or the stem. Tension: yielding, Fy Aw. Compression: buckling by DG29 Table C-1. A panel-point gusset is the chevron row: K = 0.65 with the centre-line length $l_1$ from the start of the connection back to the chord. With **Given K**, the K you enter is used with Thornton's average length.
- **Detailing.** Bolt spacing and edge distances, bolt lines inside the angle leg, and a recommended clear distance between members.

For the gusset and the chord:

- **Interface forces.** All member forces act along their work lines through the W.P. ILX resolves them at the gusset-to-stem interface into a normal force N, a shear V and a moment M about the interface centre. The joint load is added when it is applied to the gusset; otherwise it acts on the chord.
- **Interface stresses.** On the weaker of the gusset and the stem: the normal stress $Ne/(tL)$, with the equivalent normal force $Ne = |N| + 4|M|/L$ (DG29 App. B), and the shear stress $|V|/(tL)$. The combination is checked with DG29's plastic interaction (p. 56): $M/Mc + (N/Nc)^2 + (V/Vc)^4 \le 1$, with $Mc = \varphi\, Fy\, tL^2/4$, $Nc = \varphi\, Fy\, tL$ and $Vc = \varphi\, 0.60\, Fy\, tL$.
- **Interface weld.** A CJP develops the base metal. Lap fillets are designed for the larger of the peak stress and 1.25 × the average stress (Manual Part 13), with the directional strength increase.
- **Free-edge buckling** (strength; DG29 App. C.4, the admissible force maintenance method). An edge region of the gusset is an embedded plate a × b with one free edge: $\lambda = (b/t)\sqrt{Fy} / (5\sqrt{(475 + 1120/(a/b)^2)})$, $Fcr = Q\, Fy$ (Q = 1 for $\lambda \le 0.7$, $1.34 - 0.486\lambda$ up to 1.41, $1.30/\lambda^2$ beyond). The compressive stress $fa = Nc/(bt)$ must not exceed $\varphi Fcr$ ($\varphi$ 0.90). Two regions are checked:
  - the chord interface (b = L, a = the gusset depth) under $Nc = -N + 4|M|/L$ when the members push the gusset into the chord;
  - between adjacent members, a section square to the chord (b = its depth, a = the bottom free edge) under the force and moment of the members and the chord reaction on one side (DG29 Ex. 5.9, section b-b). This one can govern when diagonals on both sides push toward each other. The old $Lg/t \le 2.06\sqrt{(E/Fy)}$ rule is no longer used: DG29 explains it as a bridge fatigue rule.
- **Chord stem shear.** The interface force is spread along the stem (uniform from N, linear from M). The stem shear at each section is the force on one side, plus the joint load on the chord. The worst section is checked against $0.60\, Fy\, d\, tw$.
- **Stem local yielding** (J10.2 approach). The equivalent normal force $Ne = |N| + 4|M|/L$ (DG29 App. B) is compared with $Fy\, tw\, L$.

### Drawing

The elevation shows:

- the gusset outline, with its ends dimensioned from the W.P. and its depth;
- the members with their welds and weld symbols, or their bolts (drawn realistically) and bolt callouts;
- the work lines meeting at the W.P.;
- the CJP or lap-fillet symbol, with the lap shown hidden;
- the Whitmore spreads and sections as hidden lines, when **Show the Whitmore sections** is on.

**Section A-A** cuts square to the chord. It shows the WT with the gusset on the stem tip (CJP) or lapped on the stem (fillets), flipped for a bottom chord. **Section B-B** shows the first member on the gusset, with its welds or bolts.

### Limits

- No published example covers this joint. It is verified by hand calculations and equilibrium checks.
- A lapped gusset is offset from the chord plane. That out-of-plane eccentricity is not checked; a note says so.
- The members' own buckling belongs to the truss design and is not checked.

## Geometry not feasible

ILX marks the joint **NOT FEASIBLE** instead of NG whenever it cannot be built:

- a weld runs off the gusset, the stem or the stem plate;
- a member runs into the chord flange;
- two web members collide on the stem;
- the diagonal collides with the clip angles;
- the clip angles do not fit between the chord flange and the gusset bottom;
- a bolt hole breaks out of an angle or lies off the column flange;
- the clip-weld returns are longer than the angle leg over the gusset;
- the gusset chamfer leaves no usable plate.

Each item names the input to change and is highlighted on the drawing.

## Truss drawings

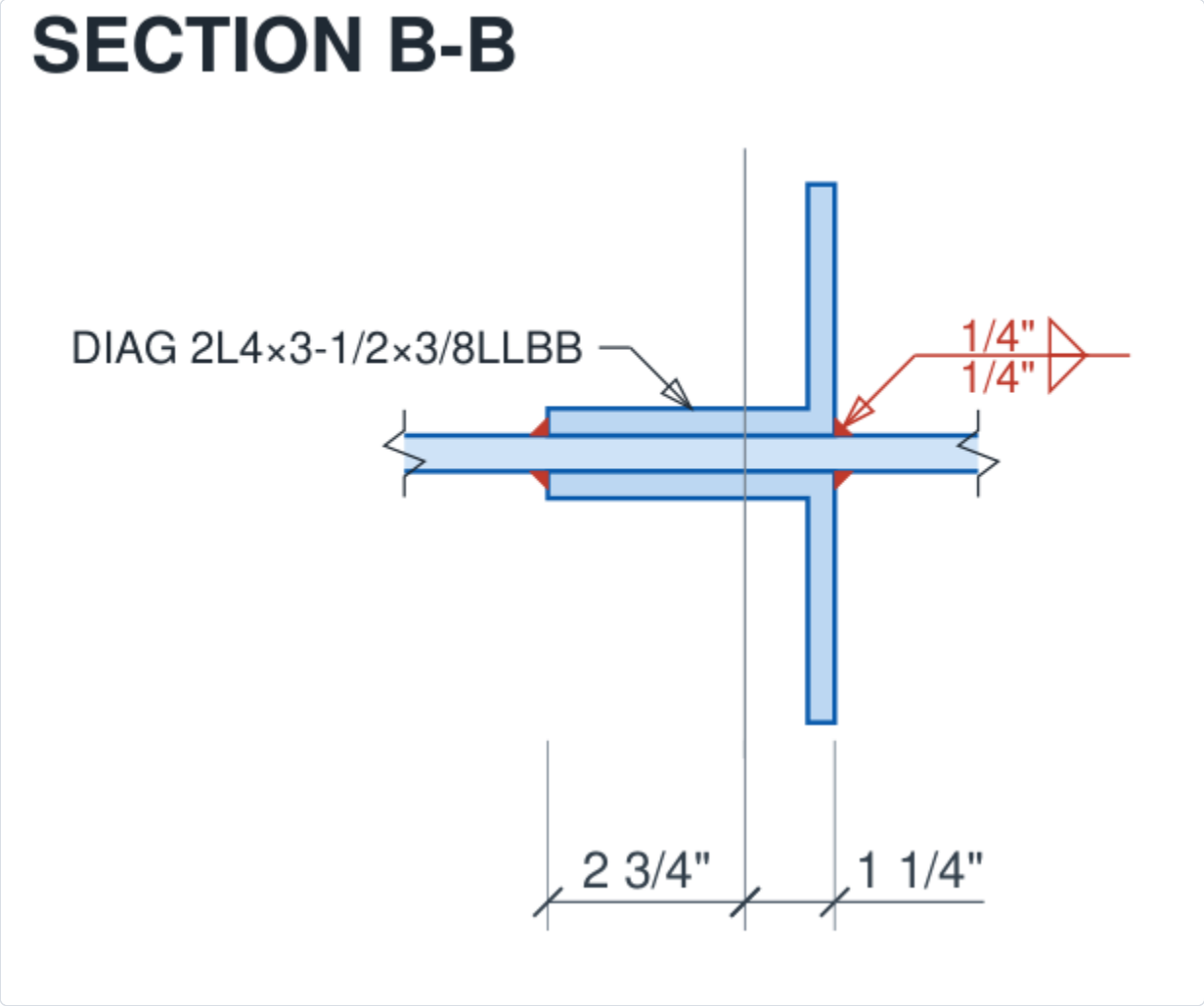
**Elevation.** The view shows:

- the chord with its real flange and stem;
- the web members as their connected legs, cut with break lines, with their work lines meeting at the W.P.;
- the heel and toe welds, with AWS A2.4 symbols that carry the weld lengths (separate HEEL and TOE symbols when the lengths differ);
- bevel triangles;
- at a support joint, also the column flanges and centerline, the gusset outline and its dimensions, the clip angles, the bolts (drawn realistically, through the angle leg and the column flange), the C-shaped weld, the CJP symbol, and the vertical bolt dimensions from the W.P.;
- at a panel point, also the stem plate, its CJP and its dimensions.

**Section A-A** (support joint): the column I-section, both clip angles, the gusset, the bolts and the fillets at the angle toes.

**Section B-B**: a typical web member. The gusset or stem lies between the two angles, with the heel and toe fillets and the work-line gage.

Click any part to select it and edit its inputs. The drawing sheet carries the elevation and one section on 8½ × 11 and 11 × 17 sheets; the report also shows Section B-B.




## Manual Part 13 truss items

These follow AISC Manual 16th Ed. Part 13, "Truss Connections" (pp. 13-22 to 13-28). Where the Manual leaves a value to the engineer it is an input with a stated default, and the report shows it.

| Item | Input (default) | What ILX does |
|---|---|---|
| **Minimum connection strength** (p. 13-24; Spec. Commentary J1.1) | *Minimum connection strength* on; value 0 = 10 kips (LRFD) / 6 kips (ASD) | A web member with no force defined (P = 0) is connected for the minimum as tension. Its welds, base metal and member end are checked for it; the chord stem shear and the equilibrium advisory keep the analysis force (0). Enter a smaller value for small elements. |

| Item | Input (default) | What ILX does |
|---|---|---|
| **Working-line offset at a panel point** (p. 13-24) | *W.P. offset from the chord work line* (0), + toward the stem | The web members' work lines meet at the offset point (the drawing shows the W.P. there). The chord takes M = ΔN e, ΔN being the change in chord force at the joint; it is reported for the chord design. The web members' own connections neglect the eccentricity (Spec. J1.7). |
| **Support eccentricity in the column** (pp. 13-25, 13-26, Figs. 13-14 / 13-15) | *Column share of R e* (1.0, the full eccentricity) | W.P. on the column face: the column takes R d/2 and the connection has no eccentricity. W.P. at the column centreline: R d/2 is shared by stiffness; the Manual's good practice, the default, gives the column all of it. The gusset-to-column connection is always designed for the full e = d/2. The column moment is reported for the column design. |
| **Tension chord elongation** (p. 13-27, Eq. 13-44) | *Check the tension chord elongation* (off); average service force, chord length, chord section (blank = this chord), erection clearance (0) | Δ = P l / (A E). Half of the total arrives at each support. A recommendation compares it with the erection clearance provided (shims for a shop-attached connection, slotted holes, reaming or a welded field connection). |

## Verification of the truss examples

The three published joints are reproduced end to end, LRFD and ASD, as golden tests. That is every limit state the examples print, 124 comparisons in all, each within ±1 % or 0.5 kip. They appear under **Settings → Verification** (connection type "Truss connection"). There are no documented deviations. Notes:

- **II.C-1 weld coefficient.** AISC interpolates Manual Table 8-8 (C = 1.50); ILX solves the IC method on the same weld group (C = 1.496).
- **Rounding.** Small differences in the clip-angle shear areas and in the column-flange prying come from the example rounding intermediate values (18.8 in.², tc = 1.14 in.).
- **Member end distances.** The examples give these only in their figures. ILX's examples use end distances that make the welds fit the stems as drawn. U1L2 at joint U1 fits only in a narrow window (3⅜ to 3 7/16 in.), which is why the example uses unequal welds instead of a stem plate.

## Truss limitations

- **Support-joint gusset weld.** It is CJP, as the example; a fillet-welded interface is designed only for a panel-point gusset.
- **Bolted members.** Bolted web members are available on a panel-point gusset only; at a support joint and on the stem they are welded.
- **Chords.** Chords of W or HSS shapes are planned.
- **Members at a support joint.** A support joint takes one diagonal. Further members in the list are ignored, with a note.
- **Auto-design and Compare.** They are not available for truss connections yet.

CHAPTER 16

# Designing a tension member end connection

The **Tension member end** type (Wizard group *Truss & bracing*, mark prefix `TM`) checks the end of a member in axial tension where it is bolted or welded to a gusset (or splice) plate: the member itself by AISC 360-22 Chapter D, and the connection by Chapter J. It covers the member shapes of the AISC Design Examples, Chapter D:

| Member shape | End connection | Design Example |
|---|---|---|
| W-shape | Both flanges bolted (or welded) to a plate each | D.1 |
| Single angle | One leg bolted or welded to the gusset | D.2 |
| WT | Flange bolted, or fillet-welded along both edges | D.3 |
| Rectangular HSS | Slotted for a single concentric gusset, four fillet welds | D.4 |
| Round HSS | Slotted for a single concentric gusset, four fillet welds | D.5 |
| Double angle | Back to back on the gusset (separation = gusset thickness), bolted or welded | D.6 |
| Plate | Bolted, straight or staggered bolt lines, or welded along both edges | D.9 |

## Tension member inputs

- **Loads**: service dead and live tension (LRFD 1.2D + 1.6L or 1.4D, ASD D + L), a required strength, ASCE 7 combinations or load cases.
- **Member**: shape, section (the list follows the shape), material, the connected leg of an angle, and the member length. The length is used only for the recommended slenderness limit L/r ≤ 300 (enter 0 to skip it).
- **Bolt pattern** (bolted): bolt lines and bolts per line, pitch *s*, gage *g* (W and WT: the gage across the web; four lines per flange use the outer line gage), the first line from the angle heel, the plate side edge distance, the middle gage of a plate with four or more lines, the stagger (alternate lines offset, or the inner lines offset as in Example D.9) with its offset, and the end distances of the member and of the gusset.
- **Welds** (welded, and always for HSS): fillet size, weld length of each line, electrode; for HSS the slotted walls and the slot clearance beyond the gusset thickness (1/16 in. in Examples D.4 and D.5).
- **Gusset**: thickness (for a double angle this is also the separation), material, and the gusset width at the connection when it limits the Whitmore section (0 = wide gusset). A W-shape has one plate per flange.

## Tension member checks

- **Tensile yielding** Pn = Fy Ag (D2a, φ = 0.90, Ω = 1.67).
- **Tensile rupture** Pn = Fu Ae, Ae = An U (D2b, D3, φ = 0.75, Ω = 2.00). The net area deducts the holes (dh + 1/16 in.) or, for HSS, the two slots. With staggered bolts every failure path through one hole per line is listed with its Σ $s^2/4g$ term and the least net width governs. The shear-lag factor U is the largest applicable value of Table D3.1:

Case 1 (bolted plate), Case 2 (1 − x̄/l), Case 4 (longitudinal welds only), Case 7 (W and WT flanges, three or more bolts per line), Case 8 (angles), the slotted-HSS cases, and never less than the connected area over Ag for open sections.

- **Block shear** of the member and of the gusset (J4.3).
- **Bolts**: shear, bearing and tearout per bolt on the member and on the gusset, the group as the sum of the per-bolt least values; slip resistance when slip-critical; spacing, edge distances and maximum limits.
- **Welds**: fillet strength with kds = 1.0 and the long-weld factor β, shear rupture of the member and of the gusset along the welds, minimum and maximum size, minimum length.
- **Gusset**: Whitmore-section yielding and rupture.
- **Slenderness**: L/r ≤ 300 as a recommendation (Spec. D1 User Note).
- **Fatigue (Appendix 3)**: with **Cyclic loading (fatigue)** ticked, the member and the gusset (Whitmore section) at the bolt holes (Table A-3.1 Case 2.1 slip-critical on the gross section, 2.2 pretensioned on the net section, or Case 1.5 for snug-tight joints), or, welded, the member and the gusset at the ends of the longitudinal welds (Case 4.1, E for t ≤ 1/2 in., E′ above) and the fillets (Case 8.2, F). Cycle count and range as for the single plate; see Fatigue (Appendix 3).

## Tension member drawings

The elevation shows the member on its gusset with the member broken off past the connection: bolts in end view (W-shapes from the side, the bolts through each flange and plate in side view), or the welds seen face-on with the weld symbol, and the bolt pitch, end distances and gages dimensioned (staggered bolts at their offsets). Section A-A cuts the member at the connection: the member cross-section on the gusset, the bolts through the grip or the fillet welds in true cross-section (slotted HSS: the gusset through both walls with the four welds).

## Tension member verification

Design Examples v16.0 D.1 to D.6 and D.9 are reproduced row by row (required strength, U and each Table D3.1 case, An, Ae, the available strengths, L/r, the D.9 net widths of every path); see the Verification page. The examples assume that the connection does not govern, so their published conclusion is compared with the member limit states; the connection checks are reported alongside. Pin-connected members and eyebars (D.7, D.8) are a separate type.

## Tension member limits

- The holes are deducted only in the connected elements; W and WT bolt lines are straight (no stagger).
- Block shear of a welded open section is not checked for the member (the base-metal shear rupture along the welds is); the stagger term is not used in the block-shear tension planes.
- The gusset is checked at its Whitmore section only; its connection to the support is a separate design.
- The 2L separation is the gusset thickness; the least radius of gyration of a 2L comes from Manual Table 1-15 when the separation is listed, else from the parallel-axis rule.

CHAPTER 17

# Designing a vertical bracing gusset (UFM)

The **Vertical bracing gusset (UFM)** type (Wizard group *Truss & bracing*, mark prefix `VB`) designs a diagonal brace connected by a gusset plate at a beam-to-column joint. The gusset is fillet welded to the top flange of the beam. At the column flange (or web) it is either fillet welded or bolted through an end plate, two clip angles or a single plate. The brace force is shared between the two gusset edges and the beam-to-column connection by the Uniform Force Method (UFM) of AISC Manual Part 13.

## Bracing gusset inputs

- **Loads**: the brace force *P* (tension positive), the beam end reaction *R*, and the axial transfer force *A* in the beam. Enter them as required strengths (the default), service loads, ASCE 7 combinations or load cases. Tick *Brace force reverses* to check the brace force in both tension and compression (seismic or wind bracing).
- **Beam and column**: the W shapes and their materials, and whether the gusset is on the column flange or the column web. For the column web, ec = 0 (Manual Part 13).
- **Brace**: double angle (back to back on the gusset), single angle, WT (flange on the gusset), a rectangular or round HSS slotted for the gusset, or a W-shape with its web in the gusset plane (see W-shape braces). Also enter the material, the connected angle leg, and the bevel as a vertical and a horizontal leg (for example 12 and 9 9/16). *Brace clearance* sets the gap between the brace end and the beam flange and column face.
- **Brace connection**: bolted (lines, bolts per line, pitch, gage, end distances on the brace and the gusset) or welded (fillet size and length; slotted HSS are always welded).
- **Gusset and UFM**: the thickness and material. The length along the beam and the height along the column are sized automatically unless you enter them. Further inputs:
  - the setback from the column face and the corner clip;
  - the UFM case;
  - optional lengths of the interface connections (when they are shorter than the gusset edges);
  - the fillet sizes on each edge, which are welded on both faces of the gusset;
  - the gusset buckling K (0 means automatic) and the buckling length rule;
  - whether the Whitmore width may extend beyond the gusset edges;
  - the gusset shape: a corner plate, optionally with its column-side free edge cut parallel to the brace at a set distance from the bolt line, or a flat bar on the brace line (special case 3 only, AISC Design Guide 29 Ex. 5.8).
- **Gusset to column**: *Welded to the column* (the default) or bolted to the column:
  - *End plate*: the plate is welded to the gusset edge and bolted to the column with two bolts per row at the cross-centre gage. Enter the plate thickness and width; the setback must be at least the plate thickness.
  - *Double clip angles*: the angles are welded to the gusset (C-shaped welds) and bolted to the column through the outstanding legs.
  - *Single plate*: the plate is welded to the column and bolted to the gusset with one line of bolts. Enter the distance from the column face to the bolt line. For each, enter the bolts (grade, diameter, threads, holes, rows,

pitch) and the element material and fillet. The bolt rows are centred on the gusset-to-column connection length.

- **Gusset to beam**: *Welded to the beam flange* (the default) or *Bolted to a tab plate welded to the beam* (see Gusset bolted to tab plates on the beam).
- **Beam-to-column connection**: *Required forces only* (the default) reports the forces. Choose *Double angles*, *Single plate* or *Shear end plate* to design it here:
  - double angles: bolted or welded to the beam web, bolted or welded to the column; the angle size and material;
  - single plate: welded to the column, bolted to the beam web (one or two bolt lines, the distance from the weld line to the bolts);
  - shear end plate: welded to the beam web, bolted to the column;
  - the bolts (grade, diameter, threads, pretensioned), the rows and spacing, the edge distance, the distance from the top of the beam, the gage at the column, the welds and the beam setback.

## Gusset bolted to tab plates on the beam

Use this when the gusset cannot be welded to the beam in the field, as at a truss chord (AISC Design Guide 29 Example 5.11). A tab plate is welded to the beam flange along both faces, and the gusset laps it and is bolted to it. The tab is as long as the gusset-to-beam connection.

Enter:

- the tab thickness, material and fillet;
- the bolts: grade, diameter, threads, bearing or slip-critical (with the slip class), and the holes in the tab and the gusset. Oversized holes need slip-critical bolts;
- the bolts per line along the beam, the number of lines, the pitch and the line spacing;
- the distance from the beam flange up to the centroid of the bolt group;
- the edge distance.

The UFM forces on the beam edge act at the flange. ILX checks:

- **the bolt group**, by the instantaneous center method with the force applied at the flange. Each bolt takes the least of slip, bolt shear, and bearing or tearout on the tab and the gusset. Tearout uses the edge distance for every bolt, as the guide does;
- **the tab plate** along the bolt line:
  - shear and tension yielding and rupture;
  - flexure from the moment Hb × (flange to bolt centroid);
  - the Neal interaction of moment, normal force and shear, for yielding and for rupture;
- **the tab-to-beam welds**, without the 1.25 ductility factor. Design Guide 29 omits it here because bolt slip lets the force redistribute.

ILX still checks the gusset along its beam edge, and the beam web under the gusset force. The drawing shows the tab face-on with its bolts. Section A-A shows the gusset lapped on the tab.

## W-shape braces

Choose *W-shape* as the brace shape. The brace web lies in the plane of the gusset. Pick one of two connections (AISC Design Guide 29):

- **Web WT splices** (Example 5.5). Two WTs, one on each face, lap the brace web and the gusset with their flanges; their stems stand out. The bolts are in double shear. Enter the tee size and material, the bolt lines and gage, the bolts per line, the pitch on the gusset (*s*) and on the brace web, the end distances, and the gap between the gusset end and the brace end.
- **Flange plates** (Example 5.11). A plate on the outside of each brace flange is bolted to the flange and slotted over the gusset, then fillet-welded to the gusset on both faces of the plate along the slot. Enter the plate width, thickness, material, fillet, weld length and the slot clearance. The bolt lines and bolts per line are per flange. Slip-critical bolts with oversized holes in the plates are allowed.

ILX checks:

- the brace in tension: yielding, rupture and block shear. The shear lag factor U is the larger of $1 - \bar{x}/l$ and the Table D3.1 Case 7 value. For a web connection $\bar{x}$ is that of half the W; for a flange connection it is that of the WT.
- the bolts on each side: per bolt the least of shear, bearing and tearout on each ply, and slip when slip-critical.
- the tees: yielding, rupture ($U = 1 - \bar{y}/l$) and block shear out to the flange edges.
- the plates: yielding, rupture at the bolts and at the gusset slot, and the plate-to-gusset welds. Each half plate carries P/4 at a quarter of the net plate width from the slot (instantaneous center method).
- the gusset: block shear and the Whitmore section. For web tees the section is at the first gusset bolt row, with width $g + 2\,l \tan 30°$. For flange plates it is at the weld ends, with width $d + 2\,t_{pl}$.

The brace end is placed by *Brace clearance*. Design Guide 29 places it by its own detailing, so the Whitmore width that runs into the beam or column web can differ.

## How the UFM forces are found

- **Geometry.** eb is half the beam depth. ec is half the column depth for a column flange, or 0 for a column web. tan θ is the horizontal leg of the bevel divided by the vertical leg (θ is measured from the vertical). $\bar{\alpha}$ and $\bar{\beta}$ are the centroids of the gusset-to-beam and gusset-to-column connections. $\bar{\alpha}$ is measured from the column face and $\bar{\beta}$ from the beam flange face.
- **General case.** With $\beta = \bar{\beta}$, Manual Eq. 13-1 gives $\alpha = eb \tan\theta - ec + \beta \tan\theta$. Then $r = \sqrt{((\alpha + ec)^2 + (\beta + eb)^2)}$ (Eq. 13-6). The interface forces are $V_c = \beta P/r$, $H_c = ec\,P/r$, $V_b = eb\,P/r$ and $H_b = \alpha P/r$ (Eqs. 13-2 to 13-5).
- **Auto-sizing.** When the gusset is sized automatically and *Auto size for α = ᾱ* is on, ILX picks the gusset length (or raises its height) so that $\bar{\alpha}$ matches α, which leaves no moment on the interfaces.
- **Moment on the gusset-to-beam edge.** When $\alpha \neq \bar{\alpha}$, that edge carries $M_b = V_b(\alpha - \bar{\alpha})$. You can set a small eccentricity that is treated as negligible. This is the Manual's prescribed-geometry analysis with the gusset-to-beam edge taken as the more rigid one (Eqs. 13-20 to 13-22), the default *General case*.
- **Prescribed geometry, other edge.** For a gusset of given size (enter its length and height, or the connection lengths) two more choices follow Manual p. 13-14:
  - *Moment on the gusset-to-column edge* (the column edge is the more rigid): $\alpha = \bar{\alpha}$ and $\beta = (\bar{\alpha} - K)/\tan\theta$ with $K = eb \tan\theta - ec$ (Eqs. 13-21, 13-23). The column edge carries $M_c = H_c(\beta - \bar{\beta})$ (Eq. 13-24) and the beam edge none.
  - *Minimised eccentricities* (both edges equally rigid): α and β minimise $((\alpha - \bar{\alpha})/\bar{\alpha})^2 + ((\beta - \bar{\beta})/\bar{\beta})^2$ on the line of Eq. 13-1: $\alpha = (K' \tan\theta + K(\bar{\alpha}/\bar{\beta})^2)/D$ and $\beta = (K' - K\tan\theta)/D$, with $D = \tan^2\theta + (\bar{\alpha}/\bar{\beta})^2$ and $K' = \bar{\alpha}(\tan\theta + \bar{\alpha}/\bar{\beta})$ (Eqs. 13-25 to 13-29). Both edges carry a moment, $M_b = V_b(\alpha - \bar{\alpha})$ and $M_c = H_c(\beta - \bar{\beta})$.

The column-edge moment goes into the gusset section, its welds and, at a column flange, the column web (as Ne = Hc + 4 Mc / L). A bolted gusset-to-column connection takes it as the normal force Hc + 6 Mc / L on its bolt rows. With an auto-sized gusset ($\bar{\alpha} = \alpha$, $\bar{\beta} = \beta$) the three choices give the same forces.

- **Special case 1.** The work point is at the gusset corner. Then Vc = P cos θ, Hc = 0, Vb = 0 and Hb = P sin θ, and the beam must be designed for Hb eb.
- **Special case 1, Design Guide 29 form.** The work point is still at the gusset corner. The guide keeps the concentric UFM forces and adds the couple M = P e of the offset brace line, with e = eb sin θ − ec cos θ (DG29 §4.2.2):
  - H′ = (1 − η) M / (β + eb) and V′ = (M − H′ β) / α;
  - the column edge gets Hc + H′ and Vc + V′, and the beam edge gets Hb − H′ and Vb − V′;
  - η = Zb / (Zb + 2 Zc) at a column flange (you can enter it), and η = 1 at a column web;
  - the beam must carry the extra couple η M, and the column (1 − η) M / 2 above and below.

  The guide recommends this form over the Manual's.
- **Special case 2.** A part ΔVb of Vb (all of it by default) moves to the gusset-to-column edge. The gusset-to-beam edge then carries Mb = ΔVb α.
- **Special case 3.** There is no gusset-to-column connection: β = 0 and α = eb tan θ − ec.
- **Special case 4** (Manual pp. 13-10 to 13-12, Fig. 13-6). The gusset is bolted to a single plate welded to the column, with the bolts outside the column flanges. Choose *Single plate* as the gusset-to-column connection; at a column web set the setback so the gusset starts outside the flange tips. ILX:
  - takes ec from the column centreline to the bolt line (column face + *column face to the bolt line*);
  - finds α = eb tan θ − ec + β tan θ with $\beta = \bar{\beta}$ (Eq. 13-15);
  - starts the gusset-to-beam weld x = sh/2 + leh + ¾ in. past the bolt line (Eq. 13-17; one bolt line, sh = 0, leh the plate's edge distance; the ¾ in. is an input) and, when the gusset is auto-sized, makes it lw = 2 (α − x) long so that $\bar{\alpha} = \alpha$ (Eq. 13-16);
  - adds the couple Fr = Vb ec / (eb + β) (Eq. 13-18): Hc + Fr on the column edge, Hb − Fr on the beam edge, and the moment Fr β on the gusset-to-beam edge (Eq. 13-19). The beam-to-column axial force includes Fr.

  When the beam web is bolted to the same plate (as in Fig. 13-6), design that plate for both bolt groups; ILX designs the gusset bolts and the beam-to-column connection separately. The beam flange is coped clear of the plate.
- **Beam-to-column connection.** The connection must carry the shear R + Vb (− ΔVb) and the axial force A ± Hc. ILX designs for the larger value, |A| + |Hc|, in tension and in compression (Design Guide 29 checks this envelope). In special case 3 at a column flange the connection also carries the moment Vb ec, taken as an extra axial force 4 Vb ec / L on the connection of length L (Design Guide 29 Example 5.4).

## Local web shear in the beam and column (Sabelli et al. 2021)

The gusset delivers Hc to the column and Vb to the beam as local shear in their webs. Sabelli, Saxey, Li and Thornton (2021, AISC Engineering Journal 58(4)) check that shear and adapt the UFM to keep it within the members' strength. The *Local web shear* section of the editor turns this on. It is off by default, since the Manual's UFM does not require it. It is always on when you choose the virtual dimension r or a bypass factor λ other than 1.

- **Inputs** (all default to 0, the paper's base case):
  - *Column shear from frame action Vcol* and *beam shear from frame action VBMF*. The gravity shear Vg is the beam end reaction R of each load combination;

  - the *available shear strengths* of the column and the beam (0 = AISC 360-22 G2.1 of the rolled shape; enter a larger value for a web doubler) and of the *beam-to-column connection* (0 = the beam's own);
  - the *vertical brace component on the gusset across the beam* and the *horizontal component on the gusset across the column*, which share the member's strength (Eqs. 22 to 25).

- **Effective strengths.** VefCol = φVn − Vcol (Eq. 17), VefBm = φVn − Vg − VBMF (Eq. 18), VefConn = φRn − Vg − VBMF ≤ VefBm (Eq. 19) and UC = VefConn / VefBm (Eq. 20). With a second gusset each strength is shared in proportion to the brace components (Eqs. 22 to 25). For ASD, Vn/Ω replaces φVn.
- **Checks.**
  - Section C1, the column at the beam flange: Hc ≤ VefC (Eq. 16);
  - section B1, the beam and its connection at the column: Vb ≤ UC VefB (Eq. 27);
  - sections C2 and B2, at the middle of each gusset edge: Vmid = N/2 + M/e ≤ VefC or VefB, with e half the connection length (Eqs. 31, 32, 37, 38);
  - beam and column web local yielding and crippling under the more heavily loaded half of each edge, on a bearing length of half the connection (Eqs. 46, 47);
  - the gusset thickness at each edge by von Mises (Eqs. 44, 45).

  The steps also give the paper's design aids: the shortest edge lengths (Eqs. 33, 39, 78) and the member strengths a given gusset needs (Eqs. 35, 42, 43).
- **Virtual dimension r** (*UFM case*: *Virtual dimension r from member shear*). The UFM forces are taken from the smallest r that keeps the column and the beam within their effective strengths: r ≥ max(ec P / VefC, eb P / (UC VefB)) (Eqs. 26 to 29), then α = r sin θ − ec and β = r cos θ − eb (Eqs. 10, 11). Enter the gusset size; its edges then carry Mc = Hc (β − $\overline{\beta}$) and Mb = Vb (α − $\overline{\alpha}$) (Eqs. 30, 36). You can give a larger r instead (p. 230). ILX also keeps r at least max(ec / sin θ, eb / cos θ), so α and β are not negative.
- **Bypass factor λ** (default 1, no bypass). The UFM takes λ P, and the rest, (1 − λ) P, goes straight to the column: HcBP = (1 − λ) P sin θ, VcBP = (1 − λ) P cos θ and McBP = HcBP (eb + $\overline{\beta}$) − VcBP ec (Eqs. 48 to 50). The edge totals are HcTot = λ Hc + HcBP, VcTot = λ Vc + VcBP, McTot = λ Mc − McBP and λ Hb, λ Vb, λ Mb (Eqs. 68 to 74). They replace the UFM forces in every check, including the welds, J10 and the beam-to-column connection. The steps give the largest λ that keeps the beam within its strength (Eq. 52). The authors do not recommend λ below 0.6, and the report says so. λ applies to the general case, the prescribed-geometry cases and the virtual dimension r.

## Bracing gusset checks

- **Brace end** (in tension). These checks are the same as the *Tension member end* type:
  - tensile yielding, and rupture with the Table D3.1 shear lag;
  - block shear of the brace and of the gusset;
  - bolts (shear, bearing, tearout, slip) or welds (strength, base metal, sizes, length);
  - spacing and edge distances.

- **Gusset at the brace.** In tension, Whitmore yielding, plus Whitmore rupture when the brace is bolted. In compression, Whitmore buckling with Thornton's lengths. K is 0.5 for a corner gusset supported on two edges (Design Example II.C-2), or 1.2 for special case 3. For compression, ILX also recommends limiting the free edge to 0.75 √(E/Fy) t.
- **Each interface.** On the gusset section, normal yielding (N + 6M/L ≤ φ Fy t L) and shear yielding (φ 0.60 Fy t L), plus a combined (von Mises) check. For the fillet welds on both faces, the design force is the larger of the peak stress and 1.25 × the average stress (Manual Part 13 ductility factor), with the directional strength increase. The Table J2.4 minimum size is also checked.

- **Beam web** under the gusset (J10.2 yielding and J10.3 crippling, for either sense of the brace force), and the **column web** under Hc (column flange connection). For the column, **Gusset near the column end (Spec. J10.2, J10.3)** and **Column end to the gusset-to-column connection** (Beam and column) set the member-end condition: cleared (default), Eqs. J10-2 and J10-4; ticked, Eq. J10-3 when the distance is ≤ d and Eqs. J10-5a/b when it is < d/2. The report states the distance and the equation.
- **Bolted gusset-to-column connection** (for the interface forces Vc and Hc; AISC Design Guide 29 Ex. 5.1, 5.5 and 6.1b):
  - the bolt group: per bolt, the least of the bolt shear and the bearing or tearout on the element and the column;
  - in tension, the bolts in tension with shear (J3-3a / J3-3b) and prying of the end plate or angle legs and of the column flange (Manual Part 9);
  - the element in shear (yielding and rupture) and block shear; for angles, the tearout of the gusset under the angle welds; for a single plate, its tension and the bolt group by the instantaneous-center method;
  - the welds to the end plate, angles or single plate, without the 1.25 ductility factor (the element is flexible).
- **Single bolt line.** The gusset block tears out down the bolt line and across to the nearest free edge (Design Guide 29 Ex. 5.4 and 5.8).
- **Beam-to-column connection.** With a connection chosen, ILX designs it with the same checks as the *Double angle*, *Single plate* or *Shear end plate* type, for the shear and the axial force above. This includes bolts in combined tension and shear, prying of the angles or end plate and of the column flange, the welds at the load angle, block shear, and the column web. The checks are listed as *Beam-to-column …*. On a column web, ILX also checks the Design Guide 29 "oil-can" yield line of the web under the transfer force. With *Required forces only*, ILX reports the forces; design the connection with one of those types.

## Seismic (SCBF) mode

Set **Seismic design** to **SCBF** to design the connection of a special concentrically braced frame, as AISC Design Guide 29 Example 6.1a does. The brace force you enter is then replaced by the expected strengths of the brace (AISC 341-22 F2.3 and F2.6c):

- **Tension**: Ry Fy Ag. AISC 341-22 F2.6c.1(b) lets you use the largest tension the system can deliver to the brace, found by analysis, when it is less. Enter it as **Max. brace tension the system can deliver** (at the LRFD or ASD level); ILX uses the lesser of the two. Leave it at 0 to use Ry Fy Ag.
- **Compression**: the lesser of Ry Fy Ag and (1/0.877) Fne Ag, with 1/0.877 = 1.140 (Design Guide 29 rounds it to 1.14). Fne is the Chapter E buckling stress with Ry Fy in place of Fy (341-16 and Design Guide 29 call it Fcre), for the **brace length** you give (end to end, that is the work-point length less the two pull-offs) and **K**. With no length, the compression is Ry Fy Ag.
- **Post-buckling**: 0.3 times the expected compression (AISC 341-22 F2.3(b)). It is a third load case. It does not govern the connection itself, but it is reported with the others.
- For ASD all three are divided by 1.5 (αs). The beam reaction R and the transfer force A stay as entered.

Ry and Rt default to AISC 341-22 Table A3.2 (Table A3.1 in 341-16; the values are the same):

| Material | Ry | Rt |
|---|---|---|
| A500 Grade B HSS | 1.4 | 1.3 |
| A500 Grade C HSS | 1.3 | 1.2 |
| A1085 HSS | 1.25 | 1.15 |
| A53 pipe | 1.6 | 1.2 |

| Material | Ry | Rt |
|---|---|---|
| A992, A572 Grade 50 shapes | 1.1 | 1.1 |
| A36 shapes | 1.5 | 1.2 |
| A572 Grade 50 plates | 1.1 | 1.2 |
| A36 plates | 1.3 | 1.2 |

The reinforcing plates use the plate rows. The editor shows the table value for the brace grade under the Ry input. You can type your own values; they replace the table.

The mode adds these checks:

- **Net-section reinforcement** (slotted rectangular HSS). The brace's effective net area must be at least its gross area. You can add two reinforcing plates, one on each wall parallel to the gusset. ILX adds them to An and to the shear-lag $\bar{x}$. The brace rupture is then checked at Rt Fu Ae. The plate welds (two along the plate, over the length a, and one across it) must develop Ry Fy Ar of each plate. The plate's Fy must be at least the brace's.
- **Brace yielding (information only)**. The expected strengths are what the connection must carry, not the brace. The brace itself is designed for the brace force from your analysis (AISC 341-22 F2.6c.1(a) Exception), outside this connection. ILX shows the brace's gross yielding against the brace force you entered, for information. It does not change the status.
- **Net section, other braces**. Braces other than a slotted rectangular HSS must also have an effective net area at least equal to their gross area. Their rupture is checked at Rt Fu Ae.
- **Gusset hinge zone**. AISC 341-22 F2.6c.3(b) asks for rotation capacity at the brace end; the Commentary (Fig. C-F2.18) shows clearance models for it. Choose the one ILX enforces with **Hinge-zone clearance enforced**; the other is reported as an advisory:
  - **2t linear** (the default, as Design Guide 29 Ex. 6.1a). The brace end must stop at least 2 tp past Point A. Point A is the end of the beam or column edge that lies farthest along the brace. ILX's gusset is rectangular, so a large gusset can push Point A far out. If the check fails, the message gives the pull-off the brace needs and the brace clearance that works with the gusset size on auto. Increase the brace clearance, or make the gusset smaller. A gusset shaped to stop its edges short of the brace, as in DG29 Example 6.1a, is not modeled.
  - **8t elliptical** (corner gussets, Fig. C-F2.18(b), Roeder et al. 2011). The restrained gusset edges are drawn as a quarter ellipse through the ends of the beam and column edges, centred at the end of the beam edge and the top of the column edge. The brace end must lie at least 8 tp inside it, that is inside the ellipse whose semi-axes are 8 tp shorter. The report gives the offset of the brace-end corner nearest the restrained edges.
  - A gusset restrained along the beam only (special case 3, flat bar) has no corner ellipse: the 2t check stays enforced, and ILX reports the 6t horizontal clearance (brace end at least 6 tp from the beam flange, Fig. C-F2.18(c), drawn for midspan gussets) for information.
- **Moment-release beam splice** (optional). Two splice plates in the beam near the column carry the beam shear R to the column face. ILX checks the eccentric bolt group (IC method, eccentricity from the column face, every bolt at its least strength), plate shear yielding and rupture, plate flexure at the first bolt line, and the Manual Part 10 tmax ductility limit.
- **Brace slenderness**. With a brace length, Lc/r must not exceed 200 (AISC 341-22 F2.5b(a)).
- **Built-up braces** (double angles). Enter the number of **stitch connectors**. ILX spaces them evenly, a = L/(n + 1), and checks a/ri ≤ 0.4 Lc/r, at least two connectors and none in the middle quarter of the brace (F2.5b(b)). The connectors themselves are not designed; the report gives the tensile strength of one angle they must match.

- **Brace buckling** (F2.6c.3). Choose how the connection accommodates it. **Rotation** (the default) is the hinge zone above. **Flexure** designs the connection for 1.1 Ry Mp / αs of the brace about its critical axis: ILX checks the gusset's weak-axis plastic moment over the Whitmore width, and the hinge-zone checks become advisories.
- **Gusset-to-flange welds** (F2.6c.4). By default the welds of the gusset to the beam flange and to the column flange must have 0.6 Ry Fy tp / αs of shear strength per unit length (the 341-22 User Note gives about 0.74 tp for A572 Grade 50 and 0.62 tp for A36 with E70 double fillets). Choose **Exception** to design them instead for the edge forces of the expected compression combined with the gusset's weak-axis flexural strength. ILX reduces the weak-axis plastic moment by the Neal interaction for those forces and resolves it on the two fillets with a lever arm of tp + 2w/3 (its own model of the Exception; the report shows each step).
- **Beam-to-column connection** (F2.6b, F3.6b, F4.6b). Choose (a) a simple connection that can rotate 0.025 rad (the default; the moment-release splice provides it), (b) a moment connection for the lesser of 1.1 Ry Mp / αs of the beam and Σ 1.1 Ry Fy Z / αs of the columns (with **Column continues above the joint** on, two column sections), or (c) an OMF connection to E1.6b(c). For (b) and (c) ILX reports the required moment with the UFM forces; the shear connection types designed here do not provide it, so set the beam-to-column type to none and design the moment connection with the FR moment connection type.
- **Bolted joints** (D2.2). Holes must be standard or short slots across the load; oversized holes are accepted only in one ply of a slip-critical brace joint. Every bolt is pretensioned with Class A (or better) faying surfaces; ILX checks the brace joint type and asks for it on the drawings where the joint is designed in bearing. **Brace bolt bearing and tearout** follows D2.2(a): deformation at service considered (2.4 d t Fu) by default, or the D2.2(a) Exception (3.0 d t Fu) where the demand is an expected brace strength.
- **V-braced bays** and **multi-tiered frames**. Tick these to get the F2.4b beam requirements, F2.3 Exception (d) (the expected brace tension on the beam limited to the expected compression, your choice) and the F2.4e requirements (with the torsional moment 1.1 Ry Mp / αs of the brace) in the report.
- **Protected zones** (F2.5c). The report lists them, and the elevation hatches the gusset, the brace end over the brace depth and, for a V-braced beam designed with Exception (d), the beam next to the gusset edge.

### Other braced-frame systems

**Seismic design** also offers:

- **OCBF** (F1.6a). The brace force you enter must be the overstrength seismic load effect. **Limit the force to the expected brace strength** applies the F1.6a Exception: at most Ry Fy Ag / αs in tension and min(Ry Fy Ag, Fne Ag) / αs in compression. In a V-braced bay the brace must satisfy Lc/r ≤ 4 √(E/Fy) (F1.5b).
- **EBF** (F3.6c). Enter the brace force from the capacity-limited analysis of F3.3. If the brace resists part of the link end moment, tick it: the connection must then be a fully restrained moment connection, which this gusset connection is not, and the report flags it.
- **BRBF** (F4.6c). Enter the steel core yield strength Pysc and the factors β and ω from the brace qualification tests. The connection is designed for ω Ry Pysc / αs in tension and β ω Ry Pysc / αs in compression. Enter Ry = 1 when Pysc comes from the measured yield stress (F4.2a Exception). The report adds the F4.6c.2 gusset bracing note and the F4.5c protected zone.

## Bracing gusset drawings

The elevation shows:

- the column and beam with breaks, and the gusset outline;
- the brace and its bolts or welds;
- the interface weld symbols and the work lines meeting at the work point;

- the bevel, eb and the gusset dimensions;
- the Whitmore section, drawn hidden;
- a note with the brace force, α, β, ec and the beam-to-column forces;
- the beam-to-column connection when it is designed: the angles or plate, their bolts and welds, with callouts;
- in a seismic mode (SCBF, BRBF), the protected zones hatched, with a note.

Section B-B shows the brace on the gusset. On screen, Section A-A shows the gusset welded to the beam flange (or lapped on the tab plate).

## Bracing gusset verification

The UFM forces are checked against AISC Design Examples V14.1:

- **II.C-3**, all four cases (general and special cases 1, 2 and 3);
- **II.C-2**: the UFM forces, the gusset-to-beam weld with the 1.25 factor, beam web yielding and crippling, the beam-to-column forces, and Whitmore buckling.

Both examples use W braces with claw angles, which ILX does not design. A surrogate brace carries the force, and only the rows that do not depend on the brace are compared. Design Guide 29 Example 5.2 (special case 1, the guide's η form) is compared in full. The W braces of Examples 5.5 (web WT splices) and 5.11 (flange plates) are compared too: the brace, the bolts, the tees or plates and their welds, and the Whitmore section. Rows with 1 in. bolt holes are 1 to 3 % lower, because of the hole difference described below.

AISC Design Guide 29 Examples 5.1 to 6.1b are also compared, including the bolted gusset-to-column end plate (5.1, 5.3), clip angles (5.5, 5.7) and single plate (6.1b), the single-line gusset block shear (5.4) and the flat-bar gusset (5.8), and the tab plate of Example 5.11 (bolt group, tab plate and Neal interaction). For 1 in. bolts ILX uses the 1-1/8 in. hole of AISC 360-22, so two shear rupture rows are 2 to 3 % lower than the guide (360-10, 1-1/16 in.). The beam-to-column connections of 5.1 (end plate), 5.4, 5.5 and 5.8 (double angles) and 6.1b (single plate) are compared too. Two differences are documented: Example 5.8 halves the column web for bolt bearing (ILX uses the full web for a one-sided connection), and Example 6.1b takes the single-plate block shear over the full plate length (ILX is 14 % lower). The seismic rows of Example 6.1a are compared in the SCBF mode: the expected strengths (1,060 and 783 kips), the reinforcing-plate net area and welds, and the moment-release splice flexure (2,840 kip-in.).

The local web shear option is compared with Design Examples 1 and 2 of Sabelli et al. (2021), LRFD (the paper has no ASD example). In Example 1 (beam web doubler) these agree within 1 %: r = 27.9 in., α = 14.2, β = 8.80, the edge forces 146 / 119 / 149 / 234 kips, Mb = 479 kip-in., Vmid = 72.8 and 118 kips, and tg = 0.390 and 0.460 in. Mc (100 kip-in.) is within 2 %, because the paper rounds β before it. In Example 2 (bypass, λ = 0.592) these agree within 1 %: r = 34.2 in., HcTot = 202, VcTot = 222, McTot = −1,310 kip-in., Vmid = 62.6 and 88.5 kips, and tg = 0.897 in. The design aids of the unreinforced Example 1 (Eqs. 35, 42, 43: 101, 135 and 150 kips) are compared too.

## Bracing gusset limits

- W-shape braces with claw angles are not designed (web WT splices and flange plates are). A gusset bolted to a tab plate is drawn with its edge on the flange, not raised to the tab bolts. The chord-to-chord tab plate of Design Guide 29 Example 5.11 (Tab Plate C) is a beam-to-column connection and is not designed. The beam-to-column connection is designed for the beam alone; one end plate shared by the beam and the gusset (Design Guide 29 Example 5.1) is not modelled.
- The beam-to-column axial force is |A| + Hc; the frame-action reduction of Design Guide 29 (Hc − HD + A) is not applied (conservative).

- The column web under the gusset-to-column bolt tension (the Design Guide 29 yield line) is noted, not checked.
- The brace member itself (compression buckling), the beam (axial Hb and any moment) and the column are designed separately.
- The UFM is concentric: the brace work line passes through the work point at the beam and column centrelines (or at the gusset corner for special case 1). A work point anywhere else (Design Guide 29 Fig. 4-13) is not offered.
- Special case 4 uses one bolt line on the single plate (sh = 0 in Eq. 13-17). No published example uses the Manual's special case 4 or the prescribed-geometry options of Eqs. 13-23 to 13-29; they are verified by hand calculations, and the brace term of the gusset-to-column force against Design Guide 29 Example A.1.
- Local web shear (Sabelli et al. 2021): the haunch method (the gusset as part of a moment connection, Eqs. 57 to 67, 75, 76) is not implemented. The two gussets above and below the beam are designed one at a time, with the other one's brace component entered to share the strength. For Eqs. 46 and 47 ILX uses the more heavily loaded half of the edge, which equals the paper's Vmid when the moment loads the far half.
- Chevron (V and inverted-V) gussets are a separate type: see Designing a chevron bracing gusset.

CHAPTER 18

# Designing a chevron bracing gusset

The **Chevron bracing gusset (chevron effect)** type (Wizard group *Truss & bracing*, mark prefix `CV`) checks the beam and the gusset where two braces meet a gusset welded to the beam: V-bracing with the gusset above the beam, or inverted-V (chevron) bracing with it below. It follows the chevron-effect section of AISC Manual 16th Ed. Part 13 (pp. 13-15 to 13-19), which is based on Sabelli and Saxey (2021) and Fortney and Thornton (2015, 2017).

The work point where the brace centre lines meet is usually on the beam centre line, half the beam depth away from the gusset. That offset puts a moment on the gusset-to-beam joint. Beam analysis that ignores the connection dimensions misses the local forces it causes; the beam shear is the one that matters most.

## Chevron gusset inputs

- **Loads**: the brace forces F1 (brace 1, on the left) and F2 (brace 2, on the right), tension positive, as required strengths, service D + L, ASCE 7 combinations or load cases. *Brace forces reverse* also checks both forces with the opposite sign. The common seismic or wind case has one brace in tension and the other in compression.
- **Beam and braces**:
  - the beam and its material, and whether the gusset is below or above it;
  - the bevel of each brace as a vertical and a horizontal leg. γ, the angle from the beam axis, comes from it;
  - *Work point offset toward the gusset* (default 0, the beam centre line). Moving the work point toward the gusset reduces the moment (Manual p. 13-15); design the beam for the moment the offset causes;
  - *Other beam shear at the gusset midpoint* (default 0). For a symmetric, symmetrically loaded frame with one gusset, 2Mf/Lg is the only shear there (Manual p. 13-18). Enter gravity shear, or the shear from a gusset on the other flange (X-bracing: the shears add, p. 13-15), here.
- **Gusset and welds**: the gusset length Lg along the beam, its depth dg at the midpoint, the thickness and material, the fillet size (welded on both faces), the electrode, and the weld ductility factor (default 1.25, Manual p. 13-15).
- **Stress distribution** (default: the *Uniform Stress Method*, the Manual's base method). Choose the *Concentrated Stress Method* when the gusset cannot be as long as Eq. 13-32 asks (Manual p. 13-17, step 2). It then asks for the *concentrated stress length z* at each gusset end. The default, 0, lets ILX choose the shortest z (in ¼ in. steps) that passes gusset yielding, beam web local yielding and crippling, and the weld.

## How the chevron forces are found

- **Forces at the gusset-to-beam joint** (Manual p. 13-16, Fig. 13-7), at the midpoint of the interface:
  - FN = F1 sin γ1 + F2 sin γ2, normal to the beam;
  - FV = F1 cos γ1 + F2 cos γ2, along the beam;
  - Mf = FV em, with em = d/2 for a concentric work point (Eq. 13-31), or d/2 less the work point offset.

- **Uniform Stress Method** (pp. 13-17 to 13-19). The moment is carried by two blocks of normal stress, one on each half of the gusset length (Fig. 13-8). ILX checks:
  - **beam shear** at the gusset midpoint, Vr = 2Mf/Lg (Eq. 13-30), plus any other shear you enter, against the beam's shear strength by AISC 360-22 G2.1. The steps also give the minimum gusset length Lg ≥ 2Mf/Vc (Eq. 13-32) and the web thickness this needs (Eq. 13-33a for LRFD, 13-33b for ASD). When the beam shear fails, ILX suggests the minimum length;
  - **beam web local yielding** (J10.2) and **crippling** (J10.3, Eq. J10-4) under two concentrated forces Rr = FN/2 ± 2Mf/Lg, each on a bearing length Lg/2 (Eq. 13-34). The larger magnitude is used for both, since the brace forces can reverse. The gusset is taken to be away from the beam ends;
  - the **gusset horizontal section** (the higher-stressed half of the interface): N = |FN/2| + |2Mf/Lg| and V = FV/2 (Eqs. 13-35, 13-36) on Lg/2 × t, by normal yielding, shear yielding and Manual Eq. 9-1;
  - the **gusset vertical section** at the midpoint: Ng (Eq. 13-37), Vg (Eq. 13-38) and Mg (Eq. 13-39) on dg × t, by shear yielding and Manual Eq. 9-1 $(M/Mc + (N/Nc)^2 + (V/Vc)^4 \le 1)$. ILX finds these forces by statics of the half-gusset (Fig. 13-8(b)), which is what the Manual's equations express;
  - the **gusset-to-beam welds**, with the stress of the Uniform Stress Method (step 5, p. 13-17): per inch of each weld line fa = (|FN|/2 ± 2|Mf|/Lg)/Lg on the two halves and fv = |FV|/(2 Lg). The design force is the larger of the peak and the ductility factor × the average (p. 13-15); the directional strength increase of J2-5 applies. The minimum fillet size of Table J2.4 is checked too.
- **Concentrated Stress Method** (p. 13-19, Fig. 13-9). The moment is carried at the two ends of the gusset over lengths z, and the middle Lg − 2z carries FN. The lever arm is ez = Lg − z, longer than the Uniform Stress Method's Lg/2, so the force at each end is smaller: Rz = Mf/ez. ILX checks:
  - **beam shear** Rz plus any other shear you enter, against G2.1. The steps show 2Mf/Lg for comparison;
  - **beam web local yielding** and **crippling** under Rz on a bearing length z, and under |FN| on the middle length Lg − 2z;
  - the **gusset** in each end length (Rz on z × t with the shear FV z/Lg) and in the middle, by normal yielding and Manual Eq. 9-1, and shear yielding on half the length;
  - the **gusset vertical section** from statics of the half-gusset of Fig. 13-9(b): Vg = F1 sin γ1 − Mf/ez + FN/2 and Mg with FN/2 on the middle half;
  - the **welds**, with the end-length stress Rz/z as the peak and the length-weighted average of the end and middle stresses (the Manual asks for welds that develop this distribution, p. 13-15).

  With z = Lg/2 the Concentrated Stress Method gives the Uniform Stress Method's forces (for FN = 0); a test checks this. With FN ≠ 0, z must leave a middle length.

## Chevron gusset drawings

The elevation shows the beam (broken at both ends), the gusset outline, the two brace work lines through the work point, the gusset-to-beam fillets with their symbol, the dimensions Lg (in halves), dg and em, the gusset callout, the brace forces and a note with FN, FV, Mf and the beam shear (2Mf/Lg, or Mf/(Lg − z) and z with the Concentrated Stress Method).

## Chevron gusset verification

The forces are compared with two published chevron examples, LRFD and ASD:

- **AISC Design Guide 29, Example 5.9** (W27X114, two HSS8X8X1/2 at 45°, ±289 / ±193 kips, a 3/4 × 18 × 64 in. gusset): the interface shear 408 / 272 kips and moment 5,560 / 3,720 kip-in., and the vertical section forces N′ =

0, V′ = 30.3 / 19.8 kips, M′ = 0. ILX is within 0.5 % (the guide rounds each brace component to 204 / 136 kips). The beam's shear strength 467 / 311 kips matches Manual Table 3-2.

- **AISC Design Examples V14.1, Example II.C-5** (W18X35, two HSS6X6X1/2, ±158 / ±105 kips): the interface shear 234 / 156 kips and moment 2,080 / 1,380 kip-in., and V′ = 11.5 / 7.57 kips at the vertical section.

Neither example applies the Manual 16th Ed. beam-shear rule of Eq. 13-30 (both are older than it). That rule and Eqs. 13-32 to 13-34 are verified by hand calculations. No published example applies the Concentrated Stress Method: it is verified by hand against Fig. 13-9, by statics of both half-gussets, and by its limit z = Lg/2.

## Chevron gusset limits

- The Concentrated Stress Method follows the Manual's description and Fig. 13-9. The Manual refers to Sabelli and Saxey (2021) for the full design procedure; that paper is not yet on file, so the choice of z (auto: the shortest passing length) and the checks in the end lengths are ILX's reading of p. 13-19.
- The brace-to-gusset connections, the Whitmore section and gusset buckling are not checked here: use the vertical bracing gusset or tension member types for them.
- The beam's own flexure and axial force, and the moment from a work point offset, are designed separately.
- The gusset is assumed away from the beam ends (interior J10 equations).

CHAPTER 19

# Designing a horizontal bracing wrap-around gusset

The **Horizontal bracing wrap-around gusset** type (Wizard group *Truss & bracing*, mark prefix `HB`) designs a horizontal brace connected by a gusset plate where two beams frame into a column at right angles. The gusset is cut out around the column, so it has two legs: leg 1 along beam 1 and leg 2 along beam 2 (AISC Manual 16th Ed. Part 13, "Wrap-Around Gusset Plates", Fig. 13-10). Each leg is lapped on its beam's top flange and bolted or welded to it.

## Wrap-around gusset inputs

- **Loads**: the brace force *P* (tension positive), as a required strength (the default), service loads, ASCE 7 combinations or load cases. Tick *Brace force reverses* to check tension and compression.
- **Minimum connection strength**: on by default. When no brace force is defined (P = 0), the connection is designed for 10 kips (LRFD) or 6 kips (ASD), Manual p. 13-24 and Specification Commentary J1.1. Enter a smaller value for small elements ("a required strength more appropriate to the size and use of the part").
- **Beams and column**: beam 1 and beam 2 (W shapes) and their material, the column, and whether beam 1 frames to the column flange (beam 2 then frames to the web, as in Fig. 13-10(a)) or to the web.
- **Brace**: double angle, single angle, WT, or a slotted rectangular or round HSS, its material and the bevel as a run along beam 1 and a run along beam 2. The angle θ is measured from beam 1. The brace end is placed clear of the cutout by the *Brace clearance to the cutout*, unless you enter the distance from the work point.
- **Brace connection**: bolted (lines, bolts per line, pitch, gage, end distances) or welded (fillet and length).
- **Wrap-around gusset**: thickness and material, the cutout e1 (along beam 1) and e2 (along beam 2), the leg lengths d1 and d2, the clearance from the cutout to the column, the leg buckling method and Cb (see below), the Whitmore buckling K, an optional chamfer of the outer corner square to the brace (the "optional cuts" of Fig. 13-10(a)), and an optional **diagonal cut** at the reentrant corner: its length along beam 2 (along leg 1's inner edge, back from leg 2) and along beam 1 (down leg 2's inner edge, back from leg 1).
- **Gusset to the beams**: *Bolted* (one bolt line along each beam, centred on the leg; bolt grade, diameter, threads, joint type, holes, bolts per leg, spacing, and the distance from the beam centreline to the bolt line) or *Welded* (fillet per leg, one line along the gusset edge or two lines adding the flange tip, and the weld ductility factor).

The gusset edges lie on the beam centrelines, as in the Manual's free-body diagram (Fig. 13-10(b)).

## What is checked

The brace force splits into Pr1 = Pr cos θ, delivered by leg 1 to beam 1, and Pr2 = Pr sin θ, delivered by leg 2 to beam 2. Each leg is a rectangular beam with its largest moment at the reentrant corner:

| Check | Basis |
|---|---|
| Leg 1 flexure, Mr1 = Pr1 e2 | Manual Eq. 13-40; Specification F11 |

| Check | Basis |
|---|---|
| Leg 2 flexure, Mr2 = Pr2 e1 | Manual Eq. 13-41; Specification F11 |
| Leg lateral-torsional buckling, general method: tension Lb1 = e2, Lb2 = e1, Cb = 1.84; compression Lb1 = e2 + d2/2, Lb2 = e1 + d1/2, Cb = 1.84 when the critical load ratio α is above 1.6 or below 1/1.6, else 1.00 | Dowswell et al. (2017) pp. 161-162, Eq. 2; Manual Eqs. 13-42, 13-43; Spec. F11 |
| Simplified method (default): tension legs fully braced, Mn = Mp = Fy Z ≤ 1.5 Fy Sx; compression as above with Cb = 1.00 | Manual p. 13-21; Dowswell et al. (2017) p. 162; Spec. Eq. F11-1 |
| Diagonal cut: buckling moment at the midpoint of the cut, Mr1 = Pr1 em2 and Mr2 = Pr2 em1 (em = work point to the cut's midpoint), with the square-cutout Lb, Cb and leg depths; plastic moment at both ends of the cut (the leg depth there, the moment to that section) | Dowswell et al. (2017) pp. 160, 162, Fig. 10, Example 3 |
| Leg shear yielding, Rn = 0.60 Fy t d (with a diagonal cut, d on the plane just beyond the gusset-to-beam connection) | Specification J4.2(a) |
| Brace end: member yielding and rupture (shear lag), bolts or welds, block shear | Specification D2, D3, J3, J4 |
| Whitmore section in tension (yielding, and rupture when bolted) and in compression (K L / r with the length back to the cutout) | Manual Part 9, Fig. 9-1; Spec. J4.1, J4.4 |
| Legs to beams, bolted: bolt shear, bearing and tearout on the gusset and the flange, slip (slip-critical), block shear of the gusset strip at the beam, spacing and edge distances | Specification J3, J4.3 |
| Legs to beams, welded: fillet strength for the larger of the force and 1.25 × the average force, not more than the force that develops the leg in shear; base metal; fillet size limits | Manual p. 13-15; Spec. J2 |

When the plate is Fy = 50 ksi, 3/8 or 1/2 in. thick and a leg length is tabulated (6½, 9½, 12½, 15½ or 18½ in., and in compression an Lb from 8 to 30 in. with Cb = 1.0), the calculation also quotes the **Manual Table 13-1** value. ILX reproduces every value of that table with the F11 calculation (see Verification).

The beams receive Pr1 and Pr2 as axial forces at the beam-to-column connections. ILX reports them; the beams, the column and the beam-to-column connections are designed separately.

## Engineer inputs and their defaults

The Manual and Dowswell et al. (2017) leave these to the engineer. Each is an input with a stated default, and the report states it:

- **Leg buckling method: simplified** (the method the Manual adopts, p. 13-21): tension legs fully braced, compression Cb = 1.0. You can choose the general method of Dowswell et al. (2017), the Manual's source, which takes Cb from the critical load ratio and can give a higher compression capacity; it matched the 15 tests better (mean test / calculated 1.40 at the maximum load). For 3/8 in. and thicker plates in tension both methods usually give Mp.
- **Cb = 0, by the method**: 1.84 or 1.00 as above. Enter a value to replace it.
- **Weld ductility factor 1.25** (Manual p. 13-15).
- **Minimum connection strength 10 kips (LRFD) / 6 kips (ASD)** when no force is defined (Manual p. 13-24).
- **Whitmore buckling K = 0.5** with the length from the Whitmore section back to the cutout. This check comes from Manual Part 9; Part 13 gives no Whitmore rule for wrap-around gussets.

## Verification against Manual Table 13-1

Manual Table 13-1 (p. 13-31) is the golden data: all 400 tabulated values (two thicknesses, five leg lengths, the tension row and 19 compression rows, ASD and LRFD) are reproduced within the table's rounding by `backend/tests/test_horizontal_bracing.py`. The AISC Design Examples v16.0 have no wrap-around gusset example.

The three worked examples of Dowswell et al. (2017) are also golden tests, LRFD and ASD, tension and compression: Example 1 (simplified method), Example 2 (general method; α = 0.639, so Cb = 1.00 in compression) and Example 3 (diagonal cut; em1 = 4⅜ in., em2 = 10 in.). Every leg moment and available strength matches the paper within its three-figure rounding.

## Limits

- The Manual notes that Eqs. 13-40 and 13-41 "may be conservative for some gusset plate geometries".
- Neither the Manual nor Dowswell et al. (2017) give a flexure-shear interaction for the legs; flexure and shear are checked separately, as in the paper.
- In its Example 3 the paper takes the shear depth on the plane beyond a clip angle on the beam web. ILX laps the legs on the top flanges and takes the plane beyond its own connection (the bolt line, or the gusset edge or flange tip for welds).
- Gussets connected through angles to the beam webs (the detail of Fig. 13-10(a)) are not modelled; the legs are lapped on the top flanges.

CHAPTER 20

# Designing built-up member connectors

The **Built-up member connectors** type (Wizard group *Components & calculators*, mark prefix `BU`) designs the welds or bolts that join the parts of a built-up steel member so that the parts act as one member. It covers three kinds of member:

| Member | Configurations | What is designed | Design Example |
|---|---|---|---|
| Flexural (shear flow) | Plate girder, W-shape with cover plate(s), box section, W-shape with a channel cap | Flange-to-web, cover-plate or cap welds (continuous or intermittent) or bolts from q = V Q / I; partial-length cover-plate termination | F.14 |
| Compression (E6) | Double angle with fillers, double channel back to back, double WT flange to flange, W column with flange plates | Intermediate connector spacing, modified slenderness, end connection, stitch spacing of outside plates | E.5, E.6, E.13 |
| Tension (D4) | Double angle, double channel, W with plates, two channels with tie plates | Connector spacing (L/r ≤ 300), J3.5 spacing of parts in contact, tie plates | — |

## Built-up member inputs

- **Loads**: flexural members take the shear V (and, for a partial-length cover plate, the moment M at the section a' from the plate end); compression and tension members the axial force P (shown for reference: the E6 and D4 rules are geometric). Service dead and live loads, a required strength, or load cases.
- **Member**: the member kind and configuration; the rolled parts (W-shape, channel, angle, WT) from the shape database; the materials of the rolled parts and of the plates.
- **Plates** (plate girder, box): web depth and thickness, the top and bottom flange plates, and for a box the flange overhang past the outside faces of the webs.
- **Cover plates**: width and thickness, plates on one or both flanges, which flange is in compression (it selects the E6.2 or the D4 / J3.5 spacing limit), and the partial-length termination (weld across the plate end, extension past the theoretical cutoff).
- **Two-shape members**: the separation (filler or connector plate thickness), the angle legs back to back, the member length with Kx and Ky, and the number of intermediate connectors (they divide the length into equal spaces a).
- **Tie plates**: the out-to-out width of the two channels (toes in), the tie plate length, thickness and spacing, and the spacing of the welds or fasteners at each tie plate.
- **Connectors**: fillet welds (continuous or intermittent: segment length and pitch center to center; one or two fillets per web joint for plate girders and boxes) or bolts (bolts per row, pitch, grade, diameter, joint type, holes); staggered bolts and unpainted weathering steel select the alternative spacing limits.

## Built-up member checks

Flexural members (AISC 360-22 F13.3):

- **Section properties** from the parts: the neutral axis, Ix, and Q of the part(s) outside each joint.
- **Shear flow** q = V Q / Ix at each joint (kip/in.).
- **Fillet welds**: strength 0.60 FEXX × 0.707 w per inch with kds = 1.0 (the weld is parallel to the shear flow), times the number of fillets, times l / s for intermittent welds; the required size in sixteenths (Manual Eqs. 8-2a/b) and the required segment length at the given pitch.
- **Base metal**: shear rupture 0.60 Fu t of the element whose faces or edge carry the welds (the web of a plate girder or box, the cover plate edge, the W flange tip under a channel cap), Eq. J2-2 and J4.2(b).
- **Fillet size limits**: Table J2.4 minimum from the thinner part, J2.2b(b) maximum along an edge; intermittent segments at least 4 w and 1-1/2 in. long (J2.2b(e)).
- **Bolts**: the least of shear, bearing and tearout per bolt (or slip for slip-critical joints), the required pitch s ≤ n φrn / q, the minimum spacing 2-2/3 d.
- **Maximum spacing** of intermittent welds or bolts: E6.2 at the compression flange (0.75 t √(E/Fy) ≤ 12 in.; staggered 1.12 t √(E/Fy) ≤ 18 in.), D4 / J3.5 at the tension flange (24 t ≤ 12 in.; weathering 14 t ≤ 7 in.).
- **Partial-length cover plates**: the termination length a' = w with an end weld of at least 3/4 tcp, 1.5 w with a smaller end weld, 2 w without one; the welds in a' (both edges plus the end weld, J2.6 combination) must develop the plate force F = M Q / Ix, and the plate must extend at least a' past the theoretical cutoff. Bolted cover plates must be slip-critical, the bolts in the extension develop F, and the cover plate area may not exceed 70 % of the total flange area.

Compression members (E6):

- **Connector spacing**: a / ri ≤ 3/4 of the governing slenderness of the built-up member (ri = the least radius of gyration of one component: rz of an angle, ry of a channel or tee), with the number of connectors needed.
- **Modified slenderness** about the axis that shears the connectors (E6.1): Eq. E6-1 for snug-tight bolts, Eq. E6-2a (a / ri ≤ 40) or E6-2b with Ki = 0.50 (angles), 0.75 (channels), 0.86 (other) for welded or pretensioned connectors. The result (and the snug-tight alternative) is reported for use in the member design (E3 / E4).
- **End connection** of members that bear on base plates or milled surfaces: weld length ≥ the maximum width of the member, or bolts at ≤ 4 d over 1.5 times the maximum width.
- **Outside plates** of a plated column: the E6.2 stitch spacing.

Tension members (D4):

- **Connector spacing**: the component slenderness between connectors a / ri ≤ 300 (User Note, a recommendation).
- **Parts in continuous contact**: the J3.5 spacing (24 t ≤ 12 in.; weathering steel 14 t ≤ 7 in.).
- **Tie plates**: length ≥ 2/3 and thickness ≥ 1/50 of the distance between the lines of welds or fasteners, welds or fasteners at the tie plates at most 6 in. apart, and the channel slenderness between tie plates ≤ 300.

## Built-up member drawings

The **section** shows the parts with every fillet weld in true cross-section (outside the base metal) or the bolts through the plies, the weld symbol (intermittent fillets with their length-pitch, for example 6"-12"), and the part marks. The **elevation** shows a length of the member broken at both ends: continuous welds or the intermittent segments at their pitch seen face-on with the segment length and pitch dimensioned, the bolts at their pitch, the connectors or tie plates at their spacing, and for a partial-length cover plate the end weld, a' and the extension to the theoretical cutoff.

## Built-up member limits

- The member itself (flexural, compressive or tensile strength) is designed elsewhere; this type designs the connectors and reports the modified slenderness for the compression design.
- Bolted plate girders (flange angles), laced members, perforated cover plates and the shear on intermediate connectors of two-shape compression members are not modelled.
- Welds joining parts at an angle to the shear flow, and loads applied directly to a flange (F13.3: welds that also transfer those loads to the web), are not included; add them to V or check them separately.
- Verification: Design Examples F.14 (continuous and intermittent flange-to-web welds), E.5, E.6 and E.13 (connector spacing and modified slenderness); every other configuration is checked against hand calculations (backend/tests/test_built_up.py).

CHAPTER 21

# Designing a beam web opening

The **Beam web opening** type (Wizard group *Components & calculators*, mark prefix `WO`) checks a rectangular or circular opening in the web of a W-shape beam, without reinforcement or with horizontal bars welded above and below the opening on one or both faces of the web. It follows AISC Design Guide 2, *Steel and Composite Beams with Web Openings* (Darwin, 1990, revised printing 2003), for **non-composite steel beams**. The opening is checked at its centreline for the moment and shear there.

| Configuration | What is checked | DG2 example |
|---|---|---|
| Unreinforced rectangular opening (concentric or eccentric) | Mm (Eq. 3-6), Vm of both tees, interaction, guidelines | Examples 1 (LRFD) and 1A (ASD) |
| Bars on one side of the web | Mm (Eq. 3-7 or 3-8), Vm with the bar term μ, one-sided limits, bar welds and extension | Example 2 |
| Bars on both sides of the web | As above, with the bar area at each edge doubled | — |
| Circular opening | The equivalent rectangle of DG2 Eqs. 3-29 / 3-30 | — |

## Web opening inputs

- **Loads**: the shear V and the moment M at the opening centreline (service dead and live loads, required strengths, ASCE 7 combinations or load cases). Enter the values at the centreline of the opening from your beam analysis.
- **Beam**: the W-shape and its material, which flange is in compression (it selects the tee for the tee-buckling guideline), and the unbraced length of the compression flange (0 when braced continuously).
- **Opening**: rectangular (length a0, depth h0, corner radius) or circular (diameter D0); the eccentricity e of the opening centre above mid-depth (negative below); optionally the clear space to the next opening and the distance from the opening edge to the support.
- **Reinforcement**: none, bars on one side or on both sides of the web; bar thickness and width (projection from the web), material, the gap between the opening edge and the bar, the extension ℓ1 past each side of the opening (0 = the DG2 minimum rounded up to 1/4 in.), the fillet size and electrode, and whether the bar is welded on one edge (the tee side) or both edges within the opening. In the extensions both edges are always welded (DG2).

## Web opening checks

Strength (DG2 Chapter 3):

- **Maximum moment capacity Mm** at the opening: Eq. 3-6 for unreinforced openings, Eq. 3-7 (tw e < Ar) or 3-8 (tw e ≥ Ar) for reinforced openings, never more than Mp = Fy Z.
- **Maximum shear capacity Vm** = Vmt + Vmb (Eq. 3-12): each tee carries αv Vp with Vp = Fy tw s / √3 and αv = (√6 + μ) / (ν + √3) ≤ 1 (Eq. 3-13), ν = a0 / s (a0 / s̄ when reinforced), μ = 2 Pr dr / (Vp s) (Eq. 3-14), Pr = Fy Ar ≤ Fy tw a0 / (2√3). Vm is limited to (2/3) V̄p for stocky webs and 0.45 V̄p otherwise (§3.7a2).

- **Moment–shear interaction**: R = [(Mu / φMm)³ + (Vu / φVm)³]^(1/3) ≤ 1 with φ = 0.90. In ASD, DG2 §3.8 uses φ = 1.00 with a load factor of 1.7 on dead and live load; ILX applies it as Ω = 1.70 on Mm and Vm with your ASD loads, which gives the same ratio 1.7 Ma / Mm.

Guidelines (DG2 §3.7): Fy ≤ 65 ksi; compression flange and bar b/t ≤ 65/√Fy; web (d − 2tf)/tw ≤ 520/√Fy; p0 = a0/h0 + 6 h0/d ≤ 5.6; a0/h0 ≤ 3.0 (2.2 for webs above 420/√Fy); h0 ≤ 0.7 d; each tee at least 0.15 d deep and a0 / s ≤ 12; corner radius ≥ 2 tw and ≥ 5/8 in.; the opening edge at least d from a support; the clear space between openings (Eqs. 3-37 / 3-38). Bars on one side only also need Ar ≤ Af/3, a0/h0 ≤ 2.5, s/tw ≤ 140/√Fy and Mu / (Vu d) ≤ 20 (Eqs. 3-33 to 3-36). Notes: no concentrated load over the opening, the tee-buckling guideline (ν ≤ 4 for unreinforced openings), and the reduction of J for lateral-torsional buckling (Eq. 3-26) when an unbraced length is entered.

Reinforcement (DG2 §3.7b5): the extension ℓ1 ≥ a0/4 and ≥ Ar √3 / (2 tw); the welds within the opening develop φ 2Pr (Eq. 3-31) and those in each extension φ Fy Ar (Eq. 3-32), with the fillet strength of AISC 360-22 J2.4 (longitudinal, kds = 1.0) and the required size in sixteenths; the bar's shear rupture (as DG2 Example 2) and the web's (J4.2); the Table J2.4 minimum and J2.2b(b) maximum fillet sizes; room for a fillet between the opening edge and the bar when both bar edges are welded within the opening.

## Web opening drawings

The **elevation** shows the beam broken at both ends, the opening with its corner radii (or the circular hole), the bars extended ℓ1 past the opening with their fillets seen face-on and the AWS fillet symbol, the dimensions ℓ1, a0, ℓ1 above the beam, the tee depths and h0 at the right, e from the beam centreline when the opening is eccentric, and the corner-radius note. **Section A-A** through the opening centre shows the top and bottom tees, the bars on one or both faces of the web and their fillets in true cross-section outside the base metal.

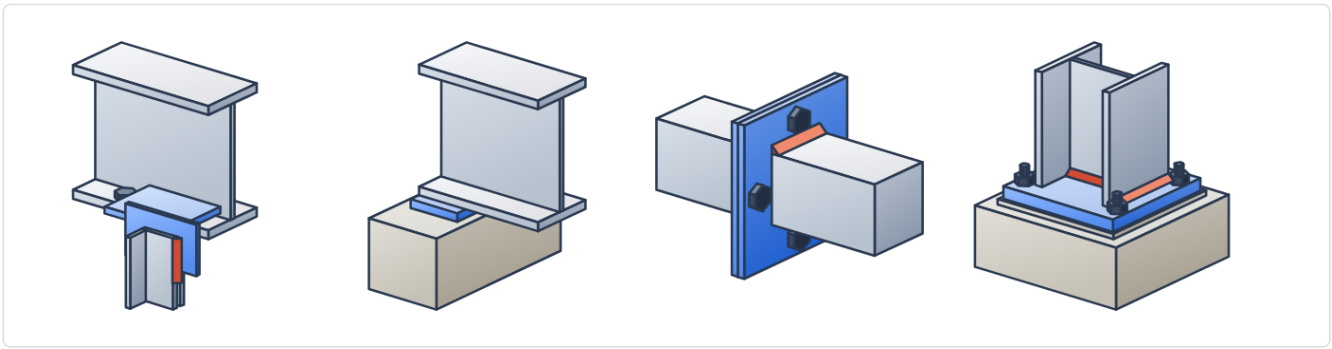
## Web opening limits

- Composite beams (DG2 §3.5b, §3.6b, the slab and shear-connector rules) are not covered; nor are castellated or cellular beams, or several openings acting together (only the spacing guideline is checked).
- The compression tee is not checked as a column (DG2 §3.7a3): ILX flags unreinforced openings with ν > 4.
- The beam's own strength away from the opening (flexure, shear, lateral-torsional buckling) is designed elsewhere.
- Verification: DG2 Examples 1, 1A and 2 (95 published values, LRFD and ASD); other configurations by hand calculations (backend/tests/test_web_opening.py). See research/38_web_openings.md.

CHAPTER 22

# Hangers, bearing plates and HSS strut end plates

These three connection types carry an axial force (tension in a hanger or strut, a bearing reaction on a wall). They share the single-plate editor's layout: the input panel on the left with collapsible sections and pins, the status bar, the drawings with click-to-edit, the output bar, and the **All checks** and **Code compliance** tabs. They are described here type by type. For how to read the checks, see Checks and compliance.



## Creating one

1. Click **New connection** in a project.
2. On the first step of the wizard, under **Tension and bearing connections**, click **Hanger**, **Beam bearing plate** or **HSS strut end plate**.
3. ILX creates the connection with the standard defaults and your project's defaults for that type, and opens it in its own editor. The defaults reproduce the AISC Design Example of each type, so a new connection starts from a verified design.

**Save as project defaults** stores materials, bolts, welds and the method for this type only. Members and loads are never saved as defaults.

## Loads for axial connections

The **Loads** section works the same way for the three types. The load symbol is **P** (tension) for the hanger and the strut and **R** (reaction) for the bearing plate.

| Input | What you enter | What ILX uses |
|---|---|---|
| **Service dead + live** | PD, PL (or RD, RL) | LRFD: max(1.2D + 1.6L, 1.4D). ASD: D + L. |
| **Service components → ASCE 7 combinations** | D, L, Lr, S, W, E | Every ASCE 7-22 2.3 (LRFD) or 2.4 (ASD) combination. W and E are applied with both signs. Each check reports its governing combination. |

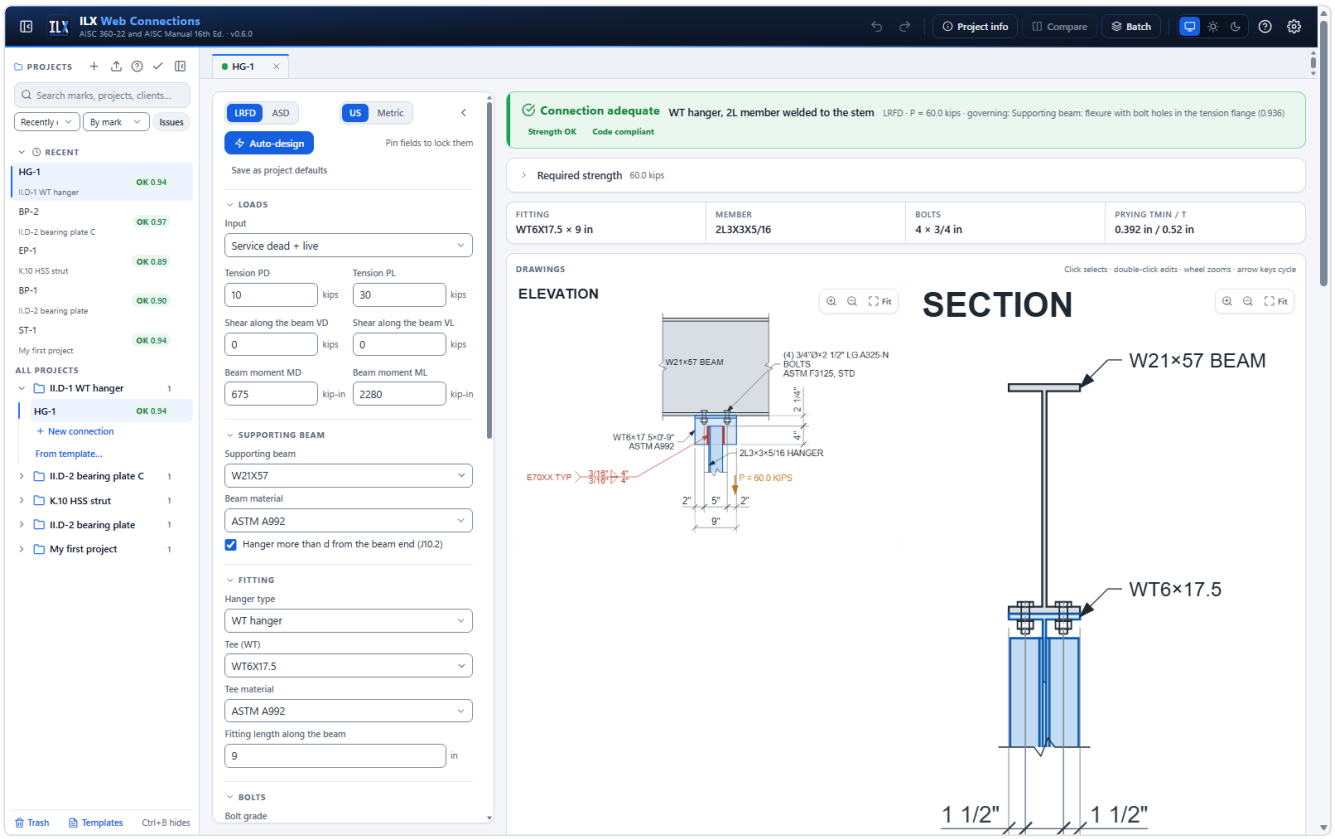
| Input | What you enter | What ILX uses |
|---|---|---|
| **Required strength** | Pu or Pa, entered directly | That value |
| **Factored load cases** | A table of named cases | Each case. The checks are enveloped across the cases. |

For the hanger there are two extra inputs:

- **Shear along the beam (V)**, for an inclined hanger. It reduces the bolt tension through AISC 360-22 Eq. J3-3a/b (F'nt).
- **Beam moment at the hanger (MD, ML or M)**. It drives the flexure check of the supporting beam with bolt holes in its tension flange (AISC 360-22 F13.1).

For the bearing plate with anchor rods, enter the required **uplift** and **horizontal force** on the rods directly.

## Hanger connections




### Hanger fitting

| Hanger type | Fitting | The hanging member attaches to |
|---|---|---|
| **WT hanger** | A WT flange bolted to the beam bottom flange. There are two bolt lines at gage g and n rows at spacing s along the beam. | The tee stem |
| **Double-angle hanger** | Two angles back to back. Their outstanding legs are bolted to the beam flange. | A plate between the vertical legs. It is bolted in double shear, or fillet-welded along the leg toes. |

| Hanger type | Fitting | The hanging member attaches to |
|---|---|---|
| **Welded plate hanger** | A plate fillet-welded to the beam flange (both faces). It runs along the web, or across the flange. | The plate |

**Fitting length along the beam** is L. The end distance is (L − (n − 1) s)/2 and is checked against AISC 360-22 Table J3.4.

**Bolt layout** (WT and double-angle hangers):

- **Rows along the fitting** (default). n rows at spacing s, centred on the fitting length.
- **4 bolts at the corners.** One bolt near each corner of the fitting flange. The two rows sit at the **corner end distance** e from each end of the fitting, so s = L − 2e. The gage g across the beam is still yours to set. If L − 2e is less than 2⅔ d, the bolts clash and ILX reports that the layout is not feasible.
  Corner bolts are end bolts in Manual Figure 9-4, so p = min(e, 1.75b) + min(s/2, 1.75b). ILX also checks that the hanger force reaches the corner bolts. The force spreads at 30° from the member attachment up through the stem to the flange, over a length Ls ≤ L:
  - the two bolts on a gage line share that length, so p ≤ Ls/2;
  - when a bolt lies beyond the loaded length, b is measured diagonally to the end of it: $b = \sqrt{b^2 + (s/2 - Ls/2)^2}$.

  With shear V, ILX also checks bearing and tearout at the fitting ends and block shear of the fitting flange. The load-spread limit is ILX's own conservative addition to the Manual method; the calculation shows each step.

## Hanging member

- **Member**:
  - **Double angle (2L)**: two angles straddling the stem, as in II.D-1.
  - **Plate**: lapped on one face of the stem, or placed between the fitting angles.
- **Attachment**:
  - **Fillet-welded**. For a 2L, the welds are on the heel and toe of each angle, with the weld length entered per line.
  - **Bolted**. Enter the number of bolts, their spacing, the member end distance and the stem bottom edge distance.

## Hanger checks

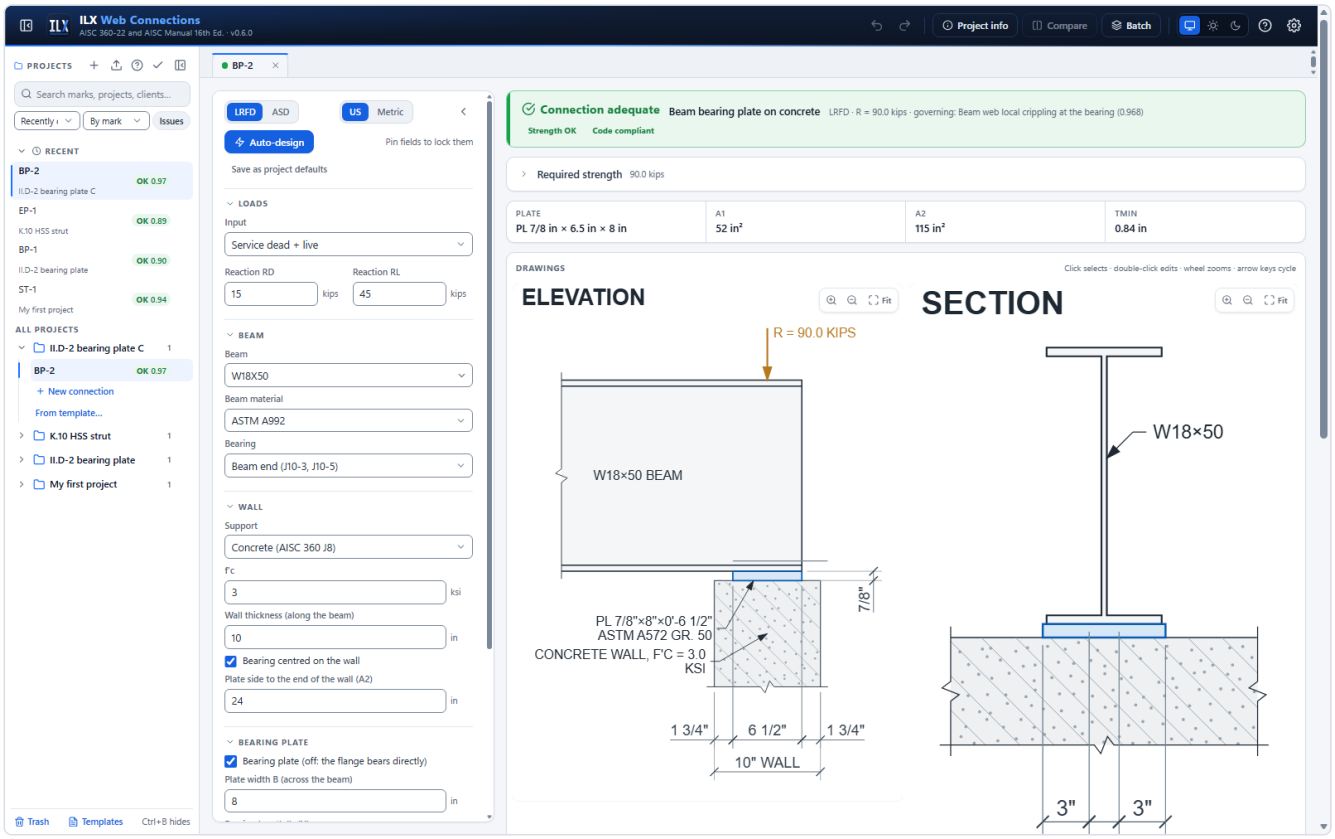
- **Bolts to the beam, tension with prying (Manual Part 9).** ILX uses both forms of the Manual method:
  - **Solution Method 2:** the minimum fitting flange thickness tmin from β and α' (Manual Eqs. 9-35 to 9-37). It is compared with the flange thickness. This is the II.D-1 calculation.
  - **Method 1:** the available tension per bolt, Q × Tc (Manual Eqs. 9-38 to 9-40), compared with the tension per bolt.

  Both use the tributary length per bolt from Manual Figure 9-4: p = min(le, 1.75b) + min(s/2, 1.75b) for an end bolt. With shear, Tc uses F'nt (AISC 360-22 J3.8).
- **The supporting flange.** Prying in the beam flange, with b and a measured on the beam: the local bending of the supporting flange. For a welded plate across the flange, flange local bending is checked by AISC 360-22 J10.1 ($6.25\ F_{yf}\ t_f^2$; halved near the beam end; not required when the plate is narrower than 0.15 bf).
- **Bolt shear and slip.** Bolt shear applies only with V. Slip-critical joints use ksc = 1 − Tu/(Du Tb nb) (Eq. J3-5).
- **The fitting.** Shear yielding and shear rupture of the tee or angle flanges (J4.2).

- **The stem or plate:**
  - tensile yielding on the Whitmore section ($l_w = w + 2\,l \tan 30°$, not more than the fitting length);
  - shear rupture of the base metal at the welds;
  - block shear, U- and L-shaped (J4.3).

  For a bolted member, rupture on the Whitmore section and L-shaped block shear at the bolts are checked as well.
- **The member.**
  - Tensile yielding (D2).
  - Tensile rupture with the shear lag factor U from AISC 360-22 Table D3.1: Case 4 for longitudinal welds, $U = 3l^2/(3l^2 + w^2)(1 - \bar{x}/l)$; Case 2 for bolts.
  - Base metal shear rupture along the welds.
  - Bolt shear, bearing and tearout, and block shear when bolted.
- **The welds.** Strength (J2.4, $k_{ds} = 1.0$ for the longitudinal member welds; 1.5 for the transverse welds of a plate hanger to the flange). The minimum length $l_{min}$ (Manual Eq. 8-2) is reported. The minimum and maximum fillet size and the 4w minimum length are checked.
- **The supporting beam.** Web local yielding under the hanger force (J10.2: $5k + l_b$, or $2.5k + l_b$ within d of the beam end). Flexure with holes in the tension flange (F13.1) when a beam moment is entered.
- **Detailing.**
  - Spacing, edge distances (Table J3.4) and the J3.6 maximums.
  - The bolt lines must lie on both flanges and clear the web and stem fillets (feasibility).
  - Tightening clearance (Manual Tables 7-15/7-16).
  - A fitting no wider than the beam flange.
  - Pretensioned bolts under cyclic loading.
- **Fatigue (Appendix 3).** With **Cyclic loading** ticked, the cycle count and the cyclic tension range (a fraction of the live tension PL, or the range of P entered) as for the single plate; see Fatigue (Appendix 3). The details: the bolts in tension with prying (Table A-3.1 Case 8.5, G, on the tensile stress area At, Eq. A-3-7: 20 % of the cyclic load for pretensioned bolts, else the load plus prying, Section 3.4(b)); this replaces the earlier design-load bolt check. A welded plate hanger: the plate at its fillets (Case 5.7, C and C″) and the fillets (Case 8.2, F). The hanging member: welded, the member at the ends of its longitudinal welds (Case 4.1) and the fillets (Case 8.2); bolted, the member at the holes (Section 2).

# Beam bearing plates




## Bearing plate inputs

- **Beam**, **Beam material** and **Bearing**:
  - **Beam end** (an end reaction): AISC 360-22 Eqs. J10-3 and J10-5a/b.
  - **Interior**: Eqs. J10-2 and J10-4.
- **Wall**:
  - **Support**: concrete (AISC 360-22 J8) or masonry (TMS 402-22).
  - **f'c** or **f'm**.
  - **Wall thickness** along the beam.
  - The bearing position: **centred**, or a **wall face to the bearing** distance.
  - The **side distance** to the end of the wall.

  These set A2, the largest area geometrically similar to and concentric with A1 that fits on the wall.
- **Bearing plate**:
  - Untick it to check the beam flange bearing directly, as in II.D-2 Solution A.
  - Width **B**, bearing length **lb**, thickness and material.
  - **Count the beam flange with the plate** is an ILX option, off by default. With it on, the plate and flange share the cantilever moment at the web, and the plate alone carries it beyond the flange tips. The Manual's method, which ignores the flange, is conservative.
- **Anchor rods** (optional). The rods run through the plate outside the flange, or through the flange.
  - Inputs: 2 or 4 rods, diameter, grade (F1554 Gr. 36, 55 or 105, or A36) and gage.

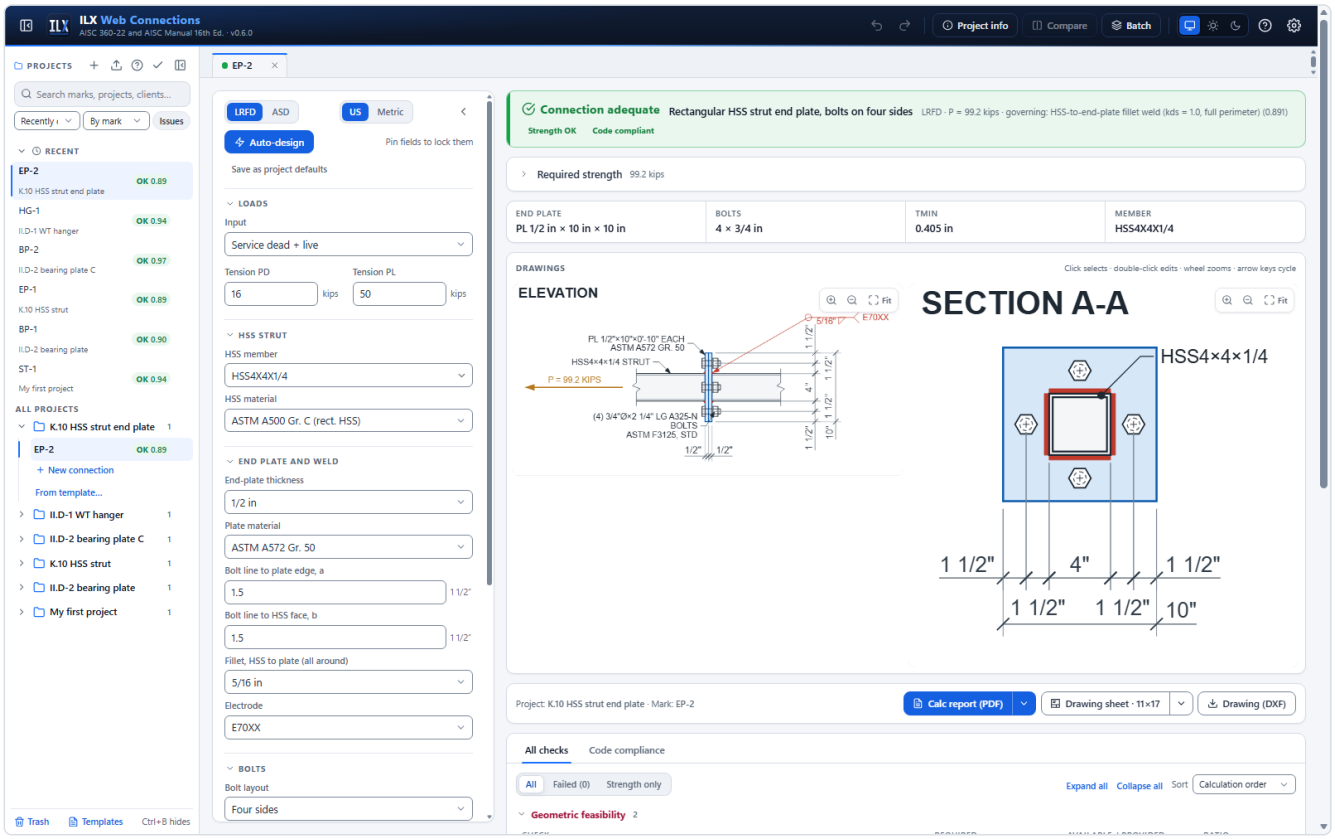
    - Hole sizes and plate washers follow Manual Table 14-1.

- **Beam bearing stiffeners (J10.8)**: **None** (default; saved connections are unchanged), **Design when required** (when web local yielding or crippling fails) or **Provided (as entered)**, with the thickness (provided only), width (0 = max), **Full** or **Half depth (from the bottom flange)**, corner clip, fillets, material, **Finished to bear on the bottom flange (J7)** (on by default) and the electrode.

## Bearing plate checks

- **Web local yielding** and **web local crippling** at the bearing (J10.2 and J10.3). The Manual Table 9-4 constants (φR1, φR2, φR5, φR6, or R3/R4 when lb/d ≤ 0.2) and the required bearing length lb,min (Manual Eqs. 9-60 to 9-63) are shown in the calculation.
- **Bearing on the support.**
    - Concrete: Pp = 0.85 f'c A1 √(A2/A1) ≤ 1.7 f'c A1 (J8-1, J8-2), φc = 0.65, Ωc = 2.31. The calculation also gives the required area A1,req and plate width B,req.
    - Masonry: TMS 402-22. Bearing area A1 √(A2/A1) ≤ 2 A1; strength design 0.8 f'm with φ = 0.60; allowable stress design 0.33 f'm.
- **Plate thickness** (Manual Part 14): n = B/2 − kdes, tmin = √(2 Ru n² / (φ Fy B lb)). Without a plate, the beam flange is checked with B = bf.
- **Geometry.**
    - The bearing must lie on the wall (feasibility).
    - lb ≥ kdes.
    - B ≥ bf (advisory).
    - The rod holes must clear the flange and fit in the plate.
- **Bearing stiffeners** (when selected). The pair over the centre of the bearing carries the whole reaction: two stiffeners plus a 12tw (end reaction) or 25tw (interior) web strip as a column, Lc = 0.75h (J10.8, J4.4, Chapter E); J10.8 width, thickness (≥ half the plate thickness, ≥ width/16) and length; corner clips; fitted bearing on the flange (J7, 1.8 Fy Apb) or flange fillets; web fillets; beam web shear rupture along the welds. When the stiffeners are adequate, web local yielding and crippling become recommendations (*bearing stiffeners required*). The drawings show the pair edge-on in the elevation and face-on in the section. The plate thickness and the bearing on the support are unchanged by the stiffeners.
- **Anchor rods.** Tension with shear (J3.8, Fnt = 0.75 Fu for threaded rods) and shear. ILX does not check the anchorage in concrete (ACI 318 Chapter 17); use the base-plate type or an anchor design program for that.

# HSS strut end plates




## End plate inputs

- **HSS member** (rectangular or round) and its material.
- **End plate**:
  - thickness and material;
  - **a**, from the bolt line to the plate edge;
  - **b**, from the bolt line to the HSS face.

  The plate size follows from the HSS size plus 2(a + b).
- **Bolts.**
  - Rectangular HSS, **Bolt layout**:
    - **Along the sides**: **four sides** or **two sides**, and the bolts per side and their spacing.
    - **4 bolts at the corners**: one bolt beyond each HSS corner, at b from both faces. The plate is B + 2(a + b) by H + 2(a + b).
  - Round HSS: the number of bolts on the circle of radius D/2 + b.
- **Weld.** A fillet all around the HSS.

The drawing shows a bolted splice of two identical end plates, as in K.10.

### End plate checks

- **Rectangular HSS.** Manual Part 9 prying with Fy in place of Fu (Willibald, Packer and Puthli 2003). The tributary length p is the full plate width divided by the bolts per side (Packer and Olson 2023, DG24 2nd Ed.). ILX checks:
  - the minimum end-plate thickness (Solution Method 2, as K.10);
  - the available bolt tension with prying (Method 1);
  - that the bolts lie within the HSS corners;
  - a ≤ 1.25 b (a larger a is not effective).
- **Rectangular HSS, corner bolts.** DG24 (1st and 2nd Ed.) covers only bolts that are not beyond the HSS corners, so there is no published method for corner bolts. ILX uses the same Manual Part 9 prying model with these conservative choices:
  - **Lever arm.** b is the diagonal distance from the bolt to the rounded HSS corner: bd = √2 (b + r) − r, with r = 2t.
  - **Edge distance.** a is the orthogonal edge distance, the shorter of the two prying arms.
  - **Tributary length.** p = min[2(B + H + π b)/4, 4 √(b′(a + bd))], from DG24 2nd Ed. Eqs. 7-17 and 7-18.
  - **Plate strength.** Fy in place of Fu, as for the other rectangular layouts.

  The tightening clearance is measured to the HSS corner. Expect a thicker plate than with bolts on four sides. For the K.10 strut, the corner layout needs a 3/4 in. plate where four sides need 1/2 in.
- **Round HSS.** DG24 Eqs. 5-5 to 5-13:
  - tp ≥ √(2 Pr / (φ Fyp π f3));
  - n ≥ (Pr / rn)[1 − 1/f3 + 1/(f3 ln(r1/r2))].

  Bolt pitch on the circle is checked as well.
- **Weld.** Rn = 0.60 FEXX (√2/2) w l kds with kds = 1.0. AISC 360-22 J2.4(a)(2) gives kds = 1.0 for fillet welds at the end of a rectangular HSS in tension. l is the full perimeter, 2(B + H) or π D, as in K.10. The calculation also gives the required weld size in sixteenths. The minimum fillet follows Table J2.4.
- **HSS member.** Tensile yielding and rupture (D2; U = 1.0 for an all-around weld to a cap plate).
- **Detailing.** The edge distance a must meet Table J3.4. The bolt must clear the HSS wall and its weld, and ILX also checks the tightening clearance b ≥ C1. Bolts in fatigue must be pretensioned.
- **Fatigue (Appendix 3).** With **Cyclic loading** ticked: the bolts in tension with prying (Case 8.5, G), the HSS wall at the toe of its weld (Case 5.7 toe crack, C; the weld is on one side of the wall, and Table A-3.1 has no root-crack case for it, so the engineer verifies the root) and the weld (Case 8.2, F); see Fatigue (Appendix 3).

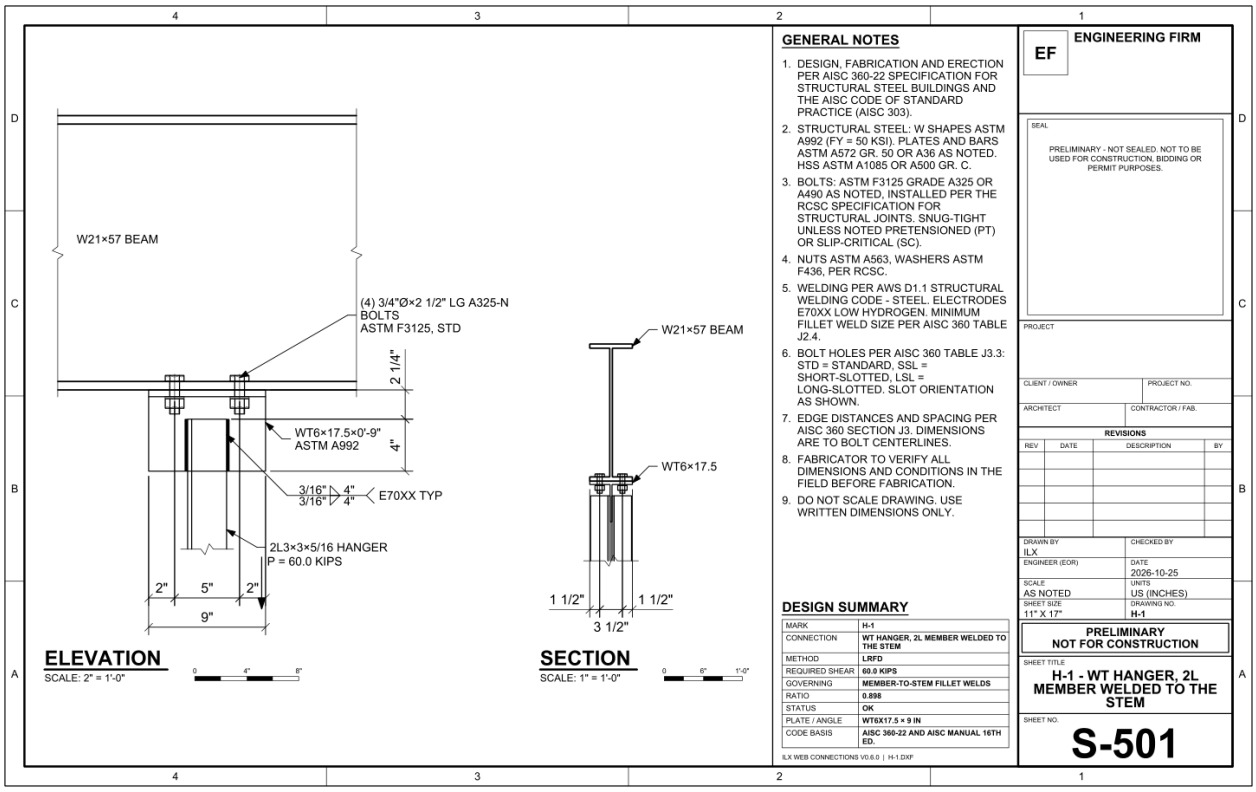
## Drawings, reports and verification

- **Drawings.** Every type has two views:
  - Hanger: an **elevation** along the beam and a **section** across it.
  - Bearing plate: an **elevation** with the wall cut in section and hatched (concrete, or masonry lines), and a **section** across the beam.
  - HSS strut: an **elevation** of the splice and **Section A-A** at the end plate.

  Members are drawn from their real AISC profiles. Bolts are drawn realistically (head, washer, shank, nut). Welds carry AWS A2.4 symbols, including the all-around symbol on the HSS weld. Click an element to select it; double-click to edit its inputs in a popover.
- **Sheets and reports.** The drawing sheet (PDF or DXF, all three sizes) and the calculation report work as for the single plate. The sheet's design summary reads **REQUIRED TENSION** or **REQUIRED REACTION**.

- **Auto-design.**
  - Hanger: the tee, the bolt rows and diameter, and the weld size and length.
  - Bearing plate: the plate thickness, width and length.
  - HSS strut end plate: the plate thickness, the bolt diameter and the weld.

  Pinned fields are kept.
- **Verification.** Design Examples II.D-1, II.D-2 (Solutions A, B and C) and K.10 are reproduced end to end in LRFD and ASD. Every limit state is within 1 % or 0.5 kip of the published value. Two rows carry the example's own rounding as a documented tolerance: the II.D-1 Whitmore available strength and the K.10 α'. The round HSS method is benchmarked against Packer (2023). See Verification.



CHAPTER 23

# Designing a column bearing on a beam

The **Column bearing on beam** type (Wizard group *Hangers & bearing*, mark prefix `CB`) checks a beam or girder where a column sits on its top flange: the column's base or cap plate (and an optional transfer plate under it) bears on the flange, and the beam web must take the concentrated force (AISC 360-22 Section J10). When the web is not adequate, ILX designs bearing stiffeners (or, if you ask for one, a web doubler).

## Column on beam inputs

- **Loads**: the column load *P* (compression positive; a negative value is uplift) and the lateral shear *V* at the base, as service dead and live loads (LRFD 1.2D + 1.6L or 1.4D, ASD D + L), required strengths, ASCE 7 combinations or load cases.
- **Beam**: the W shape and its material; whether the load is interior or near the beam end, with the distance from the end to the column centreline; whether that end is framed (a web connection). Bracing for J10.4 sidesway buckling: the largest unbraced length *Lb* of either flange at the load, whether the loaded (top) flange is restrained against rotation (normally yes for a welded or bolted base), whether the bottom flange is braced at the load (then J10.4 does not apply), and the beam moment *Mr* at the load (it sets Cr; αs = 1.0 LRFD, 1.5 ASD). Tick *Bottom flange also loaded here* when a support or another column acts on the bottom flange at the same section (J10.5).
- **Column**: a W shape (web parallel or perpendicular to the beam web), a rectangular HSS, or a bearing footprint you give (length along the beam × width). The column may be offset from the plate centre along the beam.
- **Plates**: the base / cap plate (length along the beam *N*, width *B*, thickness, material), and optionally a transfer plate under it. **Load dispersion**: 1:1 (default), 2.5:1 (web local yielding only) or none.
- **Flange bending model**: cantilever strip (default), the J10-1 analogy, or not checked.
- **Attachment**: fillet welds along both plate edges (optionally all around), or bolts in two lines through the plate and the flange.
- **Reinforcement**: bearing stiffeners (designed when required, none, or as entered) and an optional web doubler.

## How the bearing length is found

The column footprint over the beam web (the column depth when its web is parallel to the beam web; the column web thickness when perpendicular; the HSS dimension along the beam; or your footprint) spreads through each plate at the chosen slope: 1:1 adds twice the plate thickness, 2.5:1 adds five times the plate thickness. The spread never runs past the ends of the plate (nor the beam end). The J10 equations then add the spread through the beam flange (the 5k or 2.5k term). Web crippling always uses 1:1 (or none). The result shows *lb* and how it was found.

## Column on beam checks

| Check | Reference |
|---|---|
| Web local yielding (interior J10-2, within d of the end J10-3) | AISC 360-22 J10.2 |

| Check | Reference |
|---|---|
| Web local crippling (J10-4; within d/2 of the end J10-5a/b) | J10.3 |
| Web sidesway buckling (J10-6 restrained, J10-7 not restrained) | J10.4 |
| Web compression buckling (only when both flanges are loaded) | J10.5 |
| Flange local bending and web yielding in tension (uplift only) | J10.1, J10.2 |
| Flange bending under the column (no AISC limit state; the model you choose) | Manual Part 14 strip (Example II.D-2) |
| Plate thickness (cantilever beyond the column, base-plate analogy) | Manual Part 14 / Design Guide 1 |
| Bearing of the plate on the flange | J7 |
| Plate-to-flange welds, or bolts (shear, bearing, tension, no-prying flange thickness) | J2.4, J3, Manual Part 9 |
| Compression only: connectors hold the parts in place | J1.4(a) |
| Unframed beam end: full-depth stiffener pair | J10.7 |

**Bearing stiffeners carry the full column load.** A pair of stiffeners and a strip of web 25tw long (12tw at a member end) act as a column with an effective length 0.75h (J10.8, J4.4); the unstiffened web checks they replace are then shown as "bearing stiffeners required". Fitted stiffeners (the default) are checked in bearing on the loaded flange (J7); otherwise the stiffener-to-flange welds carry the load. The stiffeners are also checked for J10.8 width and thickness, length, corner clips, web welds and web shear rupture along the welds.

A **web doubler** (J10.9) can be chosen for web yielding and crippling: yielding uses the web plus doubler thickness; for crippling the doubler is added as a compression element (Commentary J10.9: the strengths add). Its welds develop its share of the force.

## Column on beam verification

Design Example J.7 (v16.0) reproduces exactly: web local yielding 248 / 165 kips, crippling 212 / 141 kips and web sidesway buckling 142 / 94.9 kips (ILX is 0.9 % higher on sidesway buckling because the Example rounds a ratio). Example II.D-3 (V14.1) checks web yielding under a plate. The stiffener column follows a published plate-girder example within 2 %.

## Column on beam limits

- Beam flexure and shear, the column itself and the column-to-plate weld are not checked here.
- The column is assumed centred over the beam web across the beam (a transverse offset adds torsion, not checked).
- The flange bending check under compression, the plate dispersion and the doubler crippling contribution are engineering models (no AISC equation); they are stated in the calculation.
- The moment *Mr* is not converted when you switch units in the editor: re-enter it.

CHAPTER 24

# Designing a crane-rail connection

The **Crane-rail connection** type (Wizard group *Hangers & bearing*, mark prefix `CR`) details a crane rail on the top flange of a runway girder: how the rail is fastened to the girder, and how two rail lengths are joined. It follows the AISC Steel Construction Manual, 16th Edition, Part 15 (*Crane Rail Connections*, pp. 15-5 to 15-9), the rail dimensions of Table 1-21 and the crane-rail notes of Part 2 (p. 2-26), with AISC Design Guide 7, 3rd Edition (*Industrial Building Design*: crane loads, CMAA service classes, rail attachments, rail joints, crane stops and runway tolerances) and the CMAA 70 crane specification (70-2020 where it is on file, else 70-1983).

Part 15 describes crane-rail connections as detailing practice. It gives recommendations, not design equations. ILX therefore reports most of its checks as **recommendations**: they quote the Manual and are flagged when not met, but they do not make the connection *Not adequate*. The strength checks are the AISC 360-22 checks of the bolts that hold the fastenings to the girder, for a crane lateral force that you enter or that ILX computes from the crane data by Design Guide 7.

| Fastening | Manual | What ILX checks |
|---|---|---|
| Hook bolts | Fig. 15-4; DG7 14.7.1 | Light rails (40 to 60 lb), CMAA Classes A to C, 20 tons or less, runway up to 500 ft, pairs 4 to 6 in. (Manual) or 3 to 4 in. (DG7) apart at 24 in. or less; hook bolt tension (J3.7) |
| Rail clips | Fig. 15-5; DG7 14.7.2 | Low-duty cranes, infrequent use, runway under 500 ft, pairs at 3 ft or less; bolt shear (J3.7), flange bearing and tearout (J3.11), edge distance |
| Tight rail clamps | Fig. 15-6; DG7 14.7.3 | 1/4 in. maximum adjustment, filler matching the rail toe, two high-strength bolts 3 in. apart with 1-1/2 in. ends, runway up to 500 ft, pairs at 3 ft or less; bolts as above |
| Floating rail clamps | Fig. 15-6; DG7 14.7.3 | Float of 1/2 to 9/16 in. (Manual) or the DG7 modified clamp, 1/4 in. or less (1/32 in. on pads); CMAA 70 and CMAA 74 advise against floating rails; bolts as above |
| Patented adjustable clips | Fig. 15-7; DG7 14.7.4 | The manufacturer's rating (entered); bolts as above |

## Crane-rail inputs

- **Loads**: the crane **lateral force H per fastening pair** (service dead and crane live loads, required strength, ASCE 7 combinations or load cases). The Manual leaves the spacing of the fastenings and the force each one takes to the crane supplier or the structural engineer (p. 15-7), so you enter it from the crane data or the design criteria, or you let ILX compute it (**Crane data**, below).
- **Rail, girder and crane**: the rail from Table 1-21 (ASCE 30 to 100 lb, ASTM A759 104 to 175 lb), the W-shape runway girder and its material, the **CMAA crane service class** A to F (or, when not entered, the crane **service duty**, low or heavy), the **runway length** and the **rail eccentricity** to the girder web centerline (default 0). With a class, ILX reads Classes A, B and C as light duty and D, E and F as heavy-duty cycle cranes (DG7 14.7.1).
- **Crane data (Design Guide 7)**: tick **Crane loads from the crane data** to replace the load input. Enter the criteria (ASCE/SEI 7-16, default, or AIST TR-13), the crane operation (cab or remote, pendant, powered monorail, hand geared), for AIST the crane type of Table 3.2, the rated capacity, the bridge weight with end trucks, the trolley and hoist weight, the number of wheels (default 4), and optionally the maximum wheel load from the crane data and

the driving wheels (default half the wheels). The defaults are the crane of DG7 Example 14.1.1 (20 tons, bridge 57.2 kips, trolley and hoist 10.6 kips, 4 wheels). ILX computes:

- the maximum wheel load: the bridge shared by all wheels, the trolley, hoist and lifted load by the wheels of one end truck (DG7 Example 14.1.2); an entered wheel load replaces it for service and ASD; LRFD is always 1.2 × the crane weights + 1.6 × the lifted load (DG7 Chapter 12);
- the vertical impact: ASCE 7 25 % (cab, remote, powered monorail), 10 % (pendant), 0 (hand geared); AIST 25 % (20 % for motor room maintenance cranes);
- the side thrust: ASCE 7 20 % of the capacity plus trolley and hoist (none for hand-geared cranes); AIST the greatest of Table 3.2, 20 % of the lifted load plus trolley and 10 % of the lifted load plus the crane (40 % and 15 % for stacker cranes), or 20 % of the load on the driving wheels for pendant cranes. It is shared equally by the wheels (DG7 Example 14.1.1);
- the tractive force per rail: ASCE 7 10 % of the maximum wheel loads on the rail; AIST 20 % of the load on its driving wheels;
- the bumper force: entered, or for a spring bumper DG7 Eq. 14-3, $F = W V^2 / (g\ ct)$, with V = 50 % of the rated bridge speed and ct the stroke, or the DG7 default, the greater of twice the tractive force and 10 % of the crane weight.

The lateral force per fastening pair is your **share of a wheel's lateral force** (default 1.0, one pair takes it all: DG7 calls the design of rail attachments "largely empirical" and gives no distribution). It is a crane live load: LRFD 1.6 (DG7's recommendation), ASD 1.0. **Fatigue inputs from the service class** (default on) sets the cycles from DG7 Table 11-1 (A 20,000; B 50,000; C 100,000; D 500,000; E 1,500,000; F over 2,000,000, taken as an indefinite life) times the **stress cycles per crane passage** (default 1), and cycles half the side thrust (DG7 12.7, AIST fatigue loads). Turn it off to enter your own fatigue inputs.

- **Rail fastening**: the type, the spacing of the pairs along the rail, the hook-bolt offset, the bolt gage across the rail, bolts per clip or clamp and their pitch, the clamp end distance, the clamp plate and filler thicknesses (filler 0 = the rail toe thickness n of Table 1-21), the tight-clamp adjustment, the floating-clamp float, an optional **manufacturer's available lateral strength** per fastening, and the elastomeric rail pad.
- **Fastener bolts**: grade (default ASTM F3125 Grade A325: DG7 recommends high-strength bolts for clamps; Fig. 15-6 shows a machine bolt), diameter and threads.
- **Rail splice**: bolted joint bars (splice, angle or joint bars of Fig. 15-3), the gap between the rail ends (default 1/16 in., the CMAA 70-2020 maximum), the bolts (track bolts or fully tensioned high-strength bolts), whether the bars are welded to the rail, clips or clamps at each end of the splice; or a flash-butt, thermite or electric-arc welded joint. Then the distance from the joint to the nearest girder end or splice, the stagger of the joints on the two rails (at least 1 ft, DG7 14.8), the crane wheel spacing and your minimum difference between the two (neither the Manual nor DG7 gives a number), the rail length and the shortest odd length. The joint bar length, the splice bolts per rail end and their diameter are for the drawing only.

## Crane-rail checks

Strength (AISC 360-22), with H applied by statics: the rail bears on the fastening on the side it is pushed toward, so that clip or clamp carries all of H.

- **Bolt shear** of the clip or clamp bolts, H / n per bolt (J3.7, Table J3.2).
- **Bearing and tearout** at the bolt holes in the girder flange, toward the flange tip (J3.11, Eqs. J3-6a and J3-6c).
- **Hook bolt in tension**: one hook bolt of a pair, the one hooked on the far flange edge, carries H (J3.7).
- **Manufacturer's rating**, when entered, against H.

Detailing: the rail base fits on the flange; the bolt holes clear the rail base and lie within the flange (*Not feasible* otherwise); the edge distance from the bolts to the flange tip and the clamp end distance (Table J3.4); the bolt pitch in a clamp (J3.4).

Recommendations (Manual Part 15 and Part 2, Design Guide 7, CMAA 70): the fastening limits in the table above; the splice gap of 1/8 in. or less (Manual) and 1/16 in. or less (CMAA 70-2020 1.4.5); splice bars not welded to the rail; clips or clamps next to each end of the splice; rail joints at least 5 ft from the girder ends and not at girder splices; joints staggered at least 1 ft between the two rails and the stagger not equal to the wheel spacing; odd rail lengths of 10 ft or more; the rail lengths available (40 lb: 30 ft; 60 lb: 30, 33 or 39 ft; standard rails: 33 or 39 ft; crane rails up to 80 ft); the rail centered on the girder within 3/4 of the web thickness (DG7 Chapter 15); a lateral float of 1/32 in. or less on rail pads. Notes repeat the guidance on welded joints (AREMA, DG7 14.8), heavy cranes, keeping bolted joints tight, the rail selection by the crane manufacturer, the rail pad, never welding rails to girders, and the RCSC Commentary on pretensioned bolts in buildings with cranes.

With a CMAA class, a note gives the class, the duty ILX reads from it and the DG7 Table 11-1 design life (CMAA 70-1983 definitions; the current edition is 70-2020, confirm the class against it). With the crane data, notes give the crane loads with every step, the **forces the runway girder must take** (wheel load with impact, lateral force per wheel, tractive force per rail, bumper force, DG7's deflection limits) and the **crane stop** guidance. ILX reports these forces; it does not design the girder or the stop.

## Crane-rail drawings

**Section A-A** cuts across the girder: the top flange with the web broken below it, the rail profile from Table 1-21 (on the pad when there is one), and the fastening on both sides with its bolts. The gage and flange width are dimensioned below and the rail depth at the right. The **elevation at the rail joint** shows the girder broken at both ends, the two rail ends with the joint bars and the splice bolts on the Table 1-21 gage line (or the welded joint), and the fastenings at their spacing. The bar length and the spacing are dimensioned above the rail.

## Crane-rail limits

- Neither Part 15 nor Design Guide 7 gives a bearing length through the rail for web local yielding or crippling (J10.2, J10.3) under a crane wheel, so ILX does not check them. DG7 checks web sidesway buckling (J10.4) under the wheel load with impact as part of the runway girder design, which ILX does not do (biaxial bending, deflection, fatigue of the girder details). The rail itself and the clip, clamp plate, filler or hook as components are not checked either (by the supplier or the engineer). The rail is selected by the crane manufacturer.
- **Fatigue (Appendix 3).** Tick **Cyclic loading (fatigue)** and enter the cycle count and the cyclic range (a fraction of the live lateral force HL, or the range of H), or let the crane data set them from the service class; see Fatigue (Appendix 3). Hook bolts: the bolt in tension (Table A-3.1 Case 8.5, Category G, on the tensile stress area; not pretensioned, so the whole range). Clips and clamps carry H in shear: bolts in shear are not checked (Section 3.4(a)), and their holes in the girder top flange are details of the runway girder under its own stress range, which ILX does not design; the result says so.
- The distribution of the crane lateral force to the fastenings is your input (DG7 gives none).
- The CMAA 70 class definitions come from the 1983 edition and the 2020 preview; the full 2020 edition is not on file. AIST TR-13 is quoted only as DG7 quotes it.
- There is no published numeric example of a crane-rail connection. The tests check the Table 1-21 data as printed, the AISC 360-22 bolt strengths by hand (backend/tests/test_crane_rail.py), and the crane loads against DG7 Examples 14.1.1 and 14.1.2 and Table 12-2 (backend/tests/test_crane_rail_dg7.py). See research/63_crane_rail_connections.md.

CHAPTER 25

# Designing a column base plate

The base plate type designs a column base plate on a concrete pier or footing. It covers axial compression and uplift, major-axis moment, shear, the anchor rods, the column-to-plate welds and the OSHA erection check. Concrete anchorage is checked to ACI 318-19 Chapter 17.

## When to use it

Use it for an exposed column base: a W, M, S, HP, HSS or Pipe column on a base plate, with four or more cast-in anchor rods in a concrete pier or footing. Shear goes by friction, by a shear lug or through the anchor rods.

It does not cover embedded column bases, stiffened base plates or moment about the column minor axis. See Limitations.

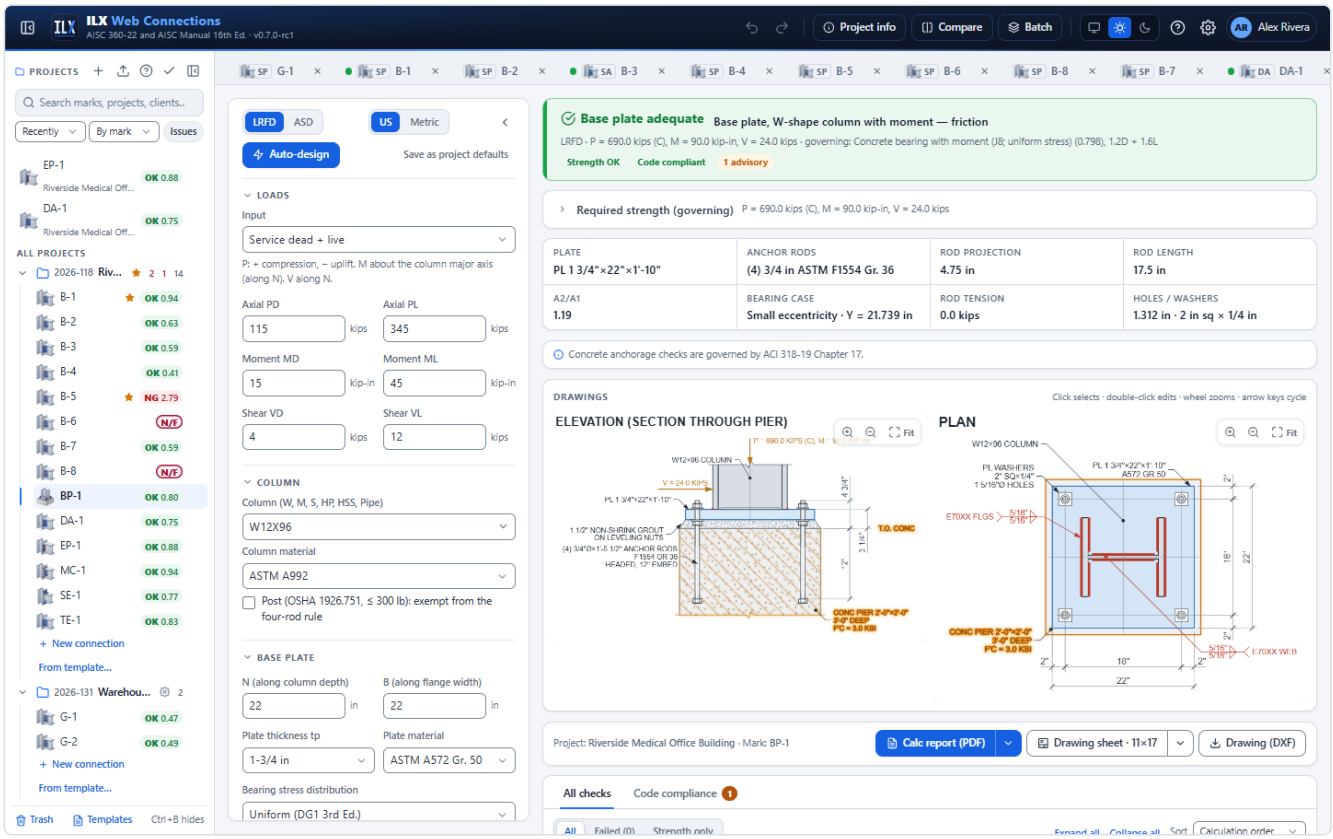
## Creating a base plate in the wizard

1. Select **New connection** and, in the **Connection type** step, pick **Base plate** in the **Column bases** group.
2. The base-plate wizard opens, titled **New column base plate**. It has one page:
   - **Connection mark**: the next free mark, **BP-1**, **BP-2** and so on.
   - **Column**: type to search the W, HP, HSS and Pipe shapes.
   - **Loads**: **Service D + L** (dead and live PD, PL, MD, ML, VD, VL) or **Required strength** (P, M, V). P is + compression, − uplift.
   - **Shear transfer**: **Friction**, **Shear lug** or **Anchor rods**.
3. Select **Create**. **Connection type** takes you back to the type choice.

Everything else starts from the standard base-plate defaults and is edited in the editor. The method, units and electrode come from the project defaults. For an HSS or Pipe column the wizard sets fillet welds and A500 Gr. C.

The base-plate wizard has no **Auto-design the connection** option. Run **Auto-design** from the editor after you create the connection.

## Inputs in the editor




The editor works like the other connection editors (see The editor workflow). The toolbar has **LRFD** or **ASD**, **US** or **Metric**, **Auto-design** and **Save as project defaults**. Save as project defaults stores this type's materials, hardware and methods, not the members and loads.

In metric, all lengths are in mm, forces in kN, moments in kN-m and stresses in MPa. Rod diameters stay in inches, with the mm value shown.

Above the drawings, a key-results strip shows the **Plate**, **Anchor rods**, **Rod projection**, **Rod length**, **A2/A1**, **Bearing case** (with the bearing length Y), **Rod tension** and **Holes / washers**. A note states that the concrete anchorage checks are governed by ACI 318-19 Chapter 17.

### Loads on the base plate

Choose how you enter the loads with **Input**. Signs: P is + compression, − uplift. M acts about the column major axis (along N). V acts along N.

**Biaxial loads.** Every input mode also takes a **minor-axis moment My**, which compresses the +B edge, and a **shear along B, Vy**:

- Service D + L: MyD, MyL, VyD, VyL.
- ASCE 7 components: the My and Vy columns.
- Required strength: My, Vy.
- Cases: the My and Vy columns.

Leave them at 0 for a uniaxial design; the results are then exactly the uniaxial ones. With My, the base plate is designed to AISC Design Guide 1, 3rd Ed. §4.3.11 (Example 4.7-14). See **Biaxial bending** under the checks.

| Input | Enter | Combinations used |
|---|---|---|
| **Service dead + live** (default) | **Axial PD**, **Axial PL**, **Moment MD**, **Moment ML**, **Shear VD**, **Shear VL** | LRFD: 1.2D + 1.6L, plus 1.4D when its axial or moment is larger. ASD: D + L. |
| **Service components → ASCE 7 combinations** | P, M and V for D, L, Lr, S, W and E | Every ASCE 7-22 LRFD (2.3) or ASD (2.4) combination. W and E act in both directions. |
| **Required strength (Pu, Mu, Vu)** | **Axial Pu**, **Moment Mu**, **Shear Vu** (Pa, Ma, Va in ASD) | None (used as entered) |
| **Factored load cases (table)** | Named cases, each with P, M and V. Select **Add case**. | Each case is checked |

- Units: forces in kips (kN), moments in kip-in (kN-m).
- The axial load keeps its sign. Moment and shear are designed by magnitude, because the rod pattern and pier are symmetric.
- Negative ASCE 7 components model reversal. The 0.9D and 0.6D combinations give the net-uplift cases.
- With more than one combination or case, a **Load combinations** table lists each one with P (+C), M, V, ratio and status. The governing one is tagged **governs**, and each check shows its governing combination.
- The OSHA erection checks run as their own case, **OSHA 1926.755 erection (300 lb at 18 in.)**.

## Column member

| Field | Options and notes |
|---|---|
| **Column (W, M, S, HP, HSS, Pipe)** | AISC shape. Rectangular and round HSS and Pipe are welded all around. |
| **Column material** | Any steel for W, M, S and HP. A500, A1085 and A53 Gr. B for HSS and Pipe. |
| **Post (OSHA 1926.751, ≤ 300 lb): exempt from the four-rod rule** | Exempts the column from the four-rod rule and the erection check. Also lowers the recommended plate thickness to 1/2 in. |

## Base plate dimensions and bearing method

| Field | Options and notes |
|---|---|
| **N (along column depth)** | Plate length in the moment direction |
| **B (along flange width)** | Plate width |
| **Plate thickness tp** | 1/2 in. to 6 in. |
| **Plate material** | A36, A572 Gr. 50 and so on |
| **Bearing stress distribution** | **Uniform (DG1 3rd Ed.)** (default) or **Triangular (DG1 2nd Ed. App. B.5)** |
| **Plate flexibility, effective area (DG1 App. B.3)** | Concentric compression only. Bearing is checked on Aeff, the column profile grown by c = 1.5 tp on every side and clipped to the plate: φ fp(max) Aeff ≥ Pu (Eq. B-24). The plate-yielding check is then shown for information only. Ignored with a moment, a lug moment or uplift. |

| Field | Options and notes |
|---|---|
| **Two-way bending width for n (DG1 App. B.4)** | With a moment, the n strip bends over beff = Y/2 + n when the bearing length Y < 2n (Eq. B-27); Mpl = fp n² Y/(2 beff). Off: beff = Y (conservative). |
| **Report the rotational stiffness (DG1 App. C.2.1)** | Adds the secant stiffness β = M/θ of the base at the governing load (kip-in./rad), for the frame model. Rod stretch, plate bending on both sides and footing compression (high eccentricity), or the footing strains (low eccentricity). Ec = 145^1.5 √f'c. **Footing depth** is the total depth under the plate (0 = pier height). Combine it in series with the footing and soil stiffness (Eq. C-1). |

## Concrete support

The section header reads **Governing code: ACI 318-19 Chapter 17**.

| Field | Options and notes |
|---|---|
| **Support** | **Concrete pier** or **Footing**. This changes the labels and the drawing; the calculation is the same. A footing is drawn with break lines. |
| **Pier length (along N)**, **Pier width (along B)**, **Pier height** | Footing length, width and thickness for a footing. The pier is concentric with the plate. |
| **Clear cover to rods** | Used for the edge distance and embedment checks |
| **f'c** | Concrete strength |
| **λa (lightweight)** | 1.0 for normal weight (ACI 318-19 17.2.4) |
| **Cracked concrete (service)** | Default ticked |
| **Supplementary reinforcement (Condition A)** | φ = 0.75 instead of 0.70 for breakout and blowout |
| **Seismic: SDC C-F (ACI 17.10)** | Concrete tension strengths × 0.75, plus a ductility note |
| **Edge reinforcement (ψc,V)** | **None**, **No. 4 bar or larger** or **Bar + stirrups ≤ 4 in** |
| **ASD: ACI strength ÷ α** | ASD only. ACI design strengths are divided by α (default 1.6) for comparison with ASD loads. |
| **Nb = 16√f'c hef^5/3 for 11 ≤ hef ≤ 25 in (17.6.2.2.3)** | Uses the long-embedment basic breakout form (default ticked) |
| **Anchor reinforcement (ACI 17.5.2.1, advisory)** | Shows **As, tension**, **As, shear** and **fy**. The reinforcement then replaces the concrete breakout. |
| **Design bars (DG1 Ex. 4.7-15)** | Pick a bar size to design the tension bars instead of entering As. **Legs crossing the breakout** sit on the two faces parallel to B, **Legs outside the rod pattern** each way (DG1 Fig. 4-36: 2 in., so g = 2√2 in. from a corner rod). ILX checks As,req = Nu/(0.75 fy), legs within 0.5 hef, the hook (or straight bar) developed above the 35° breakout plane, la = hef − cc − g/1.5 ≥ ldh (ACI 25.4.3), and the straight length below it, le = ld As,req/As ≥ 12 in. (25.4.2, 25.4.10; no reduction when Seismic is ticked). It reports hef,req and the bar length lreinf; **Bar length** 0 = rounded up from lreinf. |

## Grout and leveling

| Field | Options and notes |
|---|---|
| **Grout thickness**, **Grout strength** | Default 1-1/2 in. and 8 ksi. Bearing uses the concrete f'c, conservatively. |
| **Leveling** | **Leveling nuts**, **Leveling plate**, **Shim stacks**, **None** or **Double-nut, no grout (DG1 App. A)** |
| **Leveling plate thickness** | Leveling plate only |

**Double-nut joints (DG1 App. A.1.1).** The plate stands on the leveling nuts and is not grouted; **Grout thickness** becomes the clear distance under the plate. The rods carry all the axial load and moment as an elastic rod group of a rigid plate; concrete bearing is shown for information only. Added checks: rod compression Rc = Fy Ag (Eq. A-1, φ = 0.90) with buckling (AISC 360-22 E3, K = 1.2) when the clear distance below the leveling nut exceeds 4 da, combined with the rod bending V lc/2 by Eq. H1-1; the embedded head in compression (8 Abrg f'c, φ = 0.70); plate bending at the compression rods; a recommendation that the clear distance not exceed 2-1/2 in. Shear must go to the rods or a lug.

## Anchor rods

The rods are in two lines, one each side of the column along N. The pattern is symmetric.

| Field | Options and notes |
|---|---|
| **Rod grade** | ASTM F1554 Gr. 36, 55 or 105, ASTM A449, ASTM A36 rod |
| **Rod diameter** | 1/2 in. to 3 in. |
| **Rods per line** | 2 to 5 |
| **Rod lines spacing (along N)** | Distance between the two rod lines |
| **Rods out-to-out (along B)** | Distance between the outer rods of a line |
| **Hole diameter** | Leave blank for the AISC Manual 16th Ed. Table 14-1 hole (p. 14-20), the default. Enter a value for a smaller (or other) hole: the report states it as engineer-entered |
| **Washers** | **Plate washers (Table 14-1)** (default) or **ASTM F844 washers (Table 14-1 Note 4)** |
| **General contractor has agreed to anchor-rod placement tolerances tighter than ACI 117 (Note 4)** | F844 washers only. Note 4 says the F844 exception should not be used without this agreement; unticked, the check fails |
| **Plate washer size (square)**, **Plate washer thickness** | Leave blank to use the Table 14-1 minimum. With F844 washers they become **F844 washer outside diameter** and **F844 washer thickness**, which you enter |
| **Plate washers welded to the base plate** | All rods then share the shear, and the rods are checked for bending |
| **Embedded anchorage** | **Heavy hex nut (headed)**, **Hooked rod** or **Embedded plate washer** |
| **Embedment hef** | Always shown |
| **Hook extension eh** | Hooked rod only |
| **Embedded plate size (square)** | Embedded plate washer only |
| **Rods torqued (ACI 17.9: 6 da spacing / edge)** | Raises the minimum spacing and edge distance to 6 da |

## Shear transfer

| Shear carried by | Fields | What is checked |
|---|---|---|
| **Friction** | **Friction coefficient μ** (0.45 on a grout pad per DG1 3rd Ed.) | Friction under the concurrent compression. The rods are not checked for shear. |
| **Shear lug** | **Lug width**, **Lug depth (below plate, incl. grout)**, **Lug thickness**, **Lug material**, **Lug fillet weld** | The lug and its weld, the flexure-shear interaction of the lug (Manual Eq. 9-1) and the base plate bending at the lug (DG1 Ex. 4.7-5). The anchors are checked for tension only. The lug bears for shear along N; a shear Vy along B is checked by friction. |
| **Anchor rods** | **Setting plate** (option): **Setting plate thickness**, **Field fillet at each rod**, **Field fillet length at each rod** | Rod shear and bending, and the ACI 17.7 shear checks. With welded plate washers: washer bearing (Eq. J3-6b) and tearout (Eq. J3-6d). With a setting plate (standard holes, field welded to the base plate, DG1 Ex. 4.7-7): all rods share the shear without rod bending; setting-plate bearing (Eq. J3-6a) and field welds (kds = 1.5) are checked. With Vy the resultant shear is used, and the breakout is checked toward each edge. |

## Column-to-plate welds

| Field | Options and notes |
|---|---|
| **Flange weld** | W, M, S, HP only: **Fillet** or **CJP groove** |
| **Flange fillet** / **Fillet all around** | Flange fillet size, hidden for CJP. For HSS and Pipe, the fillet all around. 3/16 in. to 3/4 in. |
| **Web fillet** | W, M, S, HP only |
| **Fillets on both faces of each flange** | Fillets only. Adds the inside flange faces, clear of the web fillets. |
| **Electrode** | E70XX or E80XX |

## Concrete pier (option)

Tick **Design concrete pier (ACI 318-19)** in the **Concrete pier** section to design the reinforced pier under the plate. The section is offered when the support is a pier. The pier size (pier length along N × pier width along B), its height, f′c, the anchor rods and the loads come from the base plate; the pier fields set the reinforcement.

| Field | Options and notes |
|---|---|
| **Pier type** | **Reinforced** (default) or **Plain pedestal** (ACI 318-19 Chapter 14, only when the height is at most 3 × the least side) |
| **Vertical bars**, **Choose bar size** | Bar size #5 to #11; tick to let the program pick the size too |
| **Bars per face along N / along B** | Corners included; 0 = auto (the lightest layout that passes every check) |
| **Ties**, **Tie spacing** | Auto picks #3 (#4 for #11 bars) and the largest spacing allowed by ACI 25.7.2.1 and the shear checks |
| **Pier height** | Blank = the pier height of the concrete support |
| **Clear cover to ties**, **Exposure** | ACI Table 20.5.1.3.1 minimum cover is checked |

| Field | Options and notes |
|---|---|
| **fy**, **Max. aggregate size** | Bar spacing and the bar-to-rod clearance use max(1.5 in., 1.5 db, 4/3 aggregate) |
| **Footing thickness**, **Dowels**, **Standard 90° hooks** | Development of the bars into the footing (ACI 25.4); 0 = only the required length is reported. **Lapped** adds the lap splice check |
| **Rod sleeve diameter** | When the rods are sleeved the sleeve is kept clear of the bars |
| **Use pier bars (tension) and top ties (shear) as anchor reinforcement** | Pier bars within 0.5 hef of a tension rod and developed above the breakout plane (hooked with **Standard hooks at top**, else straight) carry the breakout tension; their development below the plane is checked to the pier height plus the footing. The closed top ties within 0.5 ca1 of the top count two legs each for shear. The bars used are ringed on the pier top view, the breakout planes dashed on the elevation, and the sheet summary lists them. When no bar qualifies the concrete breakout is checked as usual. |

**What it checks.** Axial load with biaxial bending by strain compatibility (ACI 22.2 to 22.4, φ per 21.2.2), checked at the top of the pier and at its base, where the shear adds V × H. Also the reinforcement limits 0.01 to 0.08 Ag and the 0.005 Ag dowel area, bar spacing and cover, tie size, spacing and lateral support, and the ties around the anchor rods within 5 in. of the top (ACI 10.7.6.1.6). Shear in both directions (ACI 22.5), slenderness and dowel development round it out.

**Bars and anchor rods.** Bars are never placed at an anchor rod. Each rod counts with its largest plan size: the rod itself, its sleeve or its embedded nut or plate. Intermediate bars shift along their face, symmetrically where they can, to clear the rods. When no layout fits, the result is **Geometry not feasible**, with a hint: move the rods inward, change the bar count or size, or enlarge the pier. Rods must also lie inside the tie cage.

**Drawings.** The pier gets its own sheet, numbered after the base plate sheet (S-501A). It has three views:

- **Pier elevation**: the plate and rods with their embedment, the ties and their spacing, and the dowels into the footing.
- **Section A-A**: the ties with hooks and crossties, the cover and the bar callout.
- **Pier top view**: the bars, the base plate outline and the rods together, with the rod positions chained and the minimum bar-to-rod clearance called out.

On letter-size paper the views go on two sheets when they don't fit on one. The pier checks and a **Concrete pier** block of input rows appear in the report.

Method, clauses and the verification record: research/43.

## Seismic design (weak base / strong base)

The **Seismic design** section designs the base for seismic loading by Design Guide 1, 3rd Ed., Chapter 6 and ACI 318-19 17.10. Pick one:

| Option | What it does |
|---|---|
| **None** (default) | Ordinary design. Results do not change. **Seismic: SDC C-F** in the concrete section still applies the ACI 17.10.5.4 × 0.75 on the concrete tension strengths. |
| **Weak base: ductile anchor rods** | The anchor rods are the part that yields (DG1 §6.4.2, Example 6.4-1). Each rod can be turned down to a **reduced shank** over a **stretch length**. LRFD and ASD (see below). |
| **Strong base: 1.1 Ry Fy Z** | The base stays elastic. The moment of every seismic load case becomes the column capacity 1.1 Ry Fy Z (DG1 §6.4.1, AISC 341-22 D2.6c(b)(1)); the axial force and shear are the ones you enter. |

| Field | Options and notes |
|---|---|
| **Reduced shank diameter** | Weak base. Always in inches, like the rod diameter. 0 = no reduced shank: the plain shank is the ductile element, and ACI 17.10.5.3(a)(v) then needs futa/fya ≥ 1.3 (Gr. 36 and 55 pass, Gr. 105 fails). |
| **Stretch length** | Weak base. Must be at least the larger of f (half the distance between the rod lines, DG1) and 8 da (ACI 17.10.5.3(a)(iii)). |
| **Rod Rt** | Weak base. Expected-strength factor on the yielded rod, 1.2 by default (DG1 Ex. 6.4-1, from ACI 17.10.5.3(a)). |
| **Column Ry** | Strong base. Expected-yield ratio of the column steel. You enter it; the default is 1.1. The line under the input shows the AISC 341-22 Table A3.2 value for the column grade (for example 1.1 for A992, 1.3 for A500 Grade C), for reference only. ILX never replaces your value. |
| **Ω0 on E** | With **ASCE 7 combinations** only: the seismic load E is multiplied by Ω0 and the combinations read Ω0E. With given loads or load cases, enter the overstrength combinations yourself. |
| **Strong-base moment** | Strong base. **1.1 Ry Fy Z / αs** (default, AISC 341-22 D2.6c(b)(1)), or **the lesser of that and the overstrength moment** (D2.6c(b)(2)). Choose (2) only when a ductile limit state of the base or the foundation controls the design: then each seismic case keeps its own moment, up to the column capacity. |
| **Story height H** | 0 (default) uses the shear you enter. With H, ILX generates the column shear of AISC 341-22 D2.6b: the lesser of 2 Ry Fy Z / (αs H) and your (overstrength) shear, but at least 0.7 Fy Z / (αs H). Strong-axis shear only; enter the weak-axis shear yourself. |
| **Minimum shear 0.7 Fy Z / (αs H)** | On by default. Turn it off for D2.6b Exception (a) (a single-story column with simple connections at both ends) or (b) (OMF, OCBF and the other systems it lists). |
| **Limit of the minimum by analysis** | D2.6b Exception (c): the largest shear the column can transfer to the foundation, from a nonlinear analysis or one at 0.025H story drift. 0 = not used. |
| **Special cantilever column system** | AISC 341-22 E6. Checks the column axial force of the seismic cases against 0.15 Pyc = 0.15 Fy Ag (E6.4a), marks the protected zone 2 d above the plate (E6.5c, hatched on the elevation) and makes the column-to-plate welds demand critical (E6.6(b)). |

Any seismic design turns on the ACI 17.10 concrete factor (× 0.75).

**Weak base, what it checks.**

- **Rod size.** The rod-group tension from the bearing analysis must not exceed n φt Fy Areduced (φt = 0.90, n = rods per line). The check reports the required area and diameter.
- **Threads.** The strain-hardened shank force Treduced = Areduced Fu must not exceed the strength of the threaded section, φNsa. This makes the rod yield in the shank, not in the threads.
- **Capacity design.** Each yielded rod delivers Rt Treduced. ILX raises the demand of the pullout, side-face blowout and breakout checks to that force, and of the tension-side plate bending to n Rt Treduced. These rows show the combination as "Weak base: yielded rods". The plate bending on the bearing side becomes information only (DG1 Ex. 6.4-1 checks the tension side only).
- **Detailing.** Stretch length; a shear lug when there is shear (DG1; ACI 17.10.6.3 does not allow anchor ductility for shear); leveling nuts under the plate; rods in tension under the loads (with no uplift, no rod yields); demand-critical welds with CJP flanges.

**Weak base in ASD.** DG1 Example 6.4-1 is LRFD only. In ASD, ILX converts it as follows and says so in a report note:

- the shank is sized with Ω = 1.67 (AISC 360 D2(a)): Ta ≤ n Fy Areduced / 1.67;

- the yielded-rod forces Treduced and Rt Treduced are expected strengths, so they are divided by αs = 1.5 (AISC 341-22, the same factor as the strong-base moment);
- those forces are then compared with the ASD strengths used elsewhere in the base plate: the ACI strengths divided by α, and the plate bending with Ω = 1.67.

LRFD results are unchanged.

**Strong base.** The base is designed by the normal Chapter 4 checks for M = 1.1 Ry Fy Z. In ASD the moment is divided by αs = 1.5 (AISC 341-22 D2.6c(b)(1)). The anchorage meets ACI 17.10.5.3(c): the anchors take the largest tension a non-yielding base can transmit. Two notes appear in the report:

- **Ry is entered by you.** The note gives your value and the AISC 341-22 Table A3.2 value for the column grade.
- **The strong-base moment replaces the moment of every load case.** With given loads or load cases this is every case. With ASCE 7 combinations it is every combination that includes E. The sign of each case is kept, and P and V are used as entered.

**Drawings.** In a weak base the elevation shows the reduced shank (dashed) from the top of concrete over the stretch length, dimensioned. The rod callout adds "REDUCED SHANK ... STRETCH, TAPED".

Method and sources: research/46.

## Checks the base plate runs

Each check cites the reference shown here. Failing checks link back to their inputs, and many have a fix hint. See Checks and compliance.

**Feasibility.** If the column does not fit on the plate, the plate on the support, the rod holes in the plate, the rods in the concrete or clear of the column steel, the holes overlap, hef is not less than the concrete depth, or the shear lug clashes, the base plate is **NOT FEASIBLE** (AISC Design Guide 1 detailing).

**Concrete bearing** (AISC 360-22 J8 (Eqs. J8-1, J8-2))

- Pp = 0.85 f′c A1 √(A2/A1) ≤ 1.7 f′c A1, φc = 0.65, Ωc = 2.31. A2 is the largest concentric area of the support similar to the plate, limited by the support plan and a 1:2 frustum within the support depth.
- Without moment, P is checked against the bearing strength.
- With moment, the Design Guide 1 analysis finds the case: **Small eccentricity** (bearing length Y = N − 2e, no rod tension), **Large eccentricity** (Y and the rod tension from statics), or **No solution** when the plate is too small for the moment. Net uplift puts the rods in tension. The reference adds "AISC Design Guide 1 … uniform-stress moment method" or the triangular App. B.5.

**Base plate flexure**

- At the bearing interface (AISC Manual 16th Ed. Part 14 (Eqs. 14-2 to 14-7)): cantilevers m, n and λn′. HSS use 0.95 or 0.80 of the wall dimension and no λn′. With moment, the m and n directions are both checked. Mn = Fy tp²/4 per unit width, φb = 0.90, Ωb = 1.67.
- At the tension interface (Design Guide 1 moment method): Mpl = T x / B, x = f − d/2 + tf/2. Rods between the flanges bend the plate about the web, with b_eff = 2x (DG1 Ex. 4.5).
- An advisory repeats the tension check over a 45° effective width.

**Anchor rods, AISC** (AISC 360-22 J3.7, J3.8 and J9 (threaded rods, Table J3.2))

- Tension: Fnt = 0.75 Fu, φ = 0.75, Ω = 2.00.

- Rods in shear: Fnv = 0.450 Fu. Without welded washers only two rods are effective, and on a grout pad the shear is multiplied by 1.25.
- Combined tension, bending and shear by Eq. J3-3a/b. With welded washers the rod bends in reverse curvature over (tp + t_washer/2)/2, with Z = d³/6.

**Concrete anchorage, ACI 318-19 Chapter 17**

| Check | Reference |
|---|---|
| Anchor steel in tension and in shear (0.80 on a grout pad) | ACI 318-19 17.6.1 / 17.7.1 (anchor steel strength) |
| Concrete breakout in tension of the tension rods | ACI 318-19 17.6.2 (concrete breakout in tension) |
| Pullout: headed, embedded plate or hooked | ACI 318-19 17.6.3 (pullout) |
| Side-face blowout, headed or plate only, when hef > 2.5 ca1 | ACI 318-19 17.6.4 (side-face blowout) |
| Concrete breakout in shear toward the edge. Front row; with welded washers the back row too. | ACI 318-19 17.7.2 (concrete breakout in shear) |
| Pryout | ACI 318-19 17.7.3 (pryout) |
| Tension-shear interaction | ACI 318-19 17.8 (tension-shear interaction) |
| Anchor reinforcement in tension and shear (φ As fy, φ = 0.75); with design bars or pier bars: placement within 0.5 hef and development above and below the breakout plane | ACI 318-19 17.5.2.1, 25.4.2, 25.4.3, 25.4.10 |

The shear checks run only when the rods carry the shear. In ASD the ACI strengths are divided by α.

**Shear transfer**

- Friction (AISC Design Guide 1: shear transfer): Vn = μ P, with P the concurrent compression, ≤ min(0.2 f′c A1, 800 psi A1). φ = 0.75, Ω = 2.00. Without compression it fails.
- Shear lug (ACI 318-19 17.11 (shear lugs)): bearing, concrete breakout toward the edge, hef/hef,sl ≥ 2.5 and a vent-hole note. Lug flexure V(G + hef,sl/2) and the lug weld couple follow Design Guide 1. Shear yielding is AISC 360-22 J4.2(a).

**Biaxial bending** (AISC Design Guide 1, 3rd Ed. §4.3.11, Eqs. 4-68/4-69; Example 4.7-14)

- **Connection check.** (Mrx/Mcx,Pr)² + (Mry/Mcy,Pr)² ≤ 1. The moment strength about each axis is taken at the applied axial load as the least of three limits:
  - Anchor rod yielding: the outer tension row develops the least of the AISC rod tension, ACI steel, pullout and row breakout. Then Y = (T + P)/q and M = q Y (L/2 − Y/2) + T f.
  - Plate yielding at the tension interface.
  - Plate yielding at the bearing interface.

  The steps list each value: Y and e at anchor capacity, M anchors, and T for plate yielding.
- **Rod forces.** For the concrete checks (breakout, pullout, blowout), ILX finds the bearing zone, a uniform fp(max) block cut by an inclined line, and the rod forces of a rigid plate. Rods on the tension half carry force in proportion to their distance from the centroidal axis (an elastic analysis, as ACI 318-19 17.2.1 requires for brittle modes).
- **Rod steel in tension.** Under compression with the shear on a lug or friction, the steel is governed by the interaction (ductile, plastic redistribution as DG1 assumes). The per-rod steel checks are then advisory. Under uplift, or with the rods in shear, they are strength checks.

- **Plate.** The bearing interface uses the full cantilever max(m, n). The tension interface takes the larger of the strong-axis rod line over B and the outer rod row along B over N.
- **Welds.** The weak-axis moment also goes to the flange fillets (each flange carries My/2) or the HSS weld group.
- **Drawings and pier.** The plan view outlines the bearing zone and gives the number of rods in tension. The pier option receives My and Vy.

**Column-to-plate welds** (AISC 360-22 J2.4, Table J2.5; AISC Design Guide 1 §2.4)

- W columns: the tension-flange force M/(d − tf) − P/2 on the flange fillets (load normal to the weld, × 1.5) or the CJP weld (base-metal yielding). The web fillets carry the shear, and the rod tension for rods between the flanges. The column web next to the web welds is checked for shear rupture, 0.60 Fu tw T (AISC 360-22 Eq. J4-4; DG1 Ex. 4.7-5 and 4.7-8).
- HSS and Pipe: the fillet all around, q from M/Sw − P/Lw and the shear. A note asks for welded corners under tension or moment.
- Minimum fillet size (AISC 360-22 J2.2b, Table J2.4).

**Detailing** (AISC Design Guide 1 detailing, AISC Manual 16th Ed. Table 14-1, ACI 318-19 17.9)

- Holes and plate washer size and thickness per Table 14-1 (p. 14-20). An entered hole larger than the table hole is a recommendation NG. A smaller hole is your choice; the report notes that the table holes correlate with ACI 117 tolerances (Note 1).
- ASTM F844 washers (Table 14-1 Note 4): the hole must be at most d + 5/16 in. for rods up to 1 in., d + 1/2 in. over 1 in. up to 2 in., and d + 1 in. over 2 in.; the tolerance agreement must be ticked; the F844 washer diameter and thickness must be entered. The plate-washer minimum checks are then not made.
- Rods not in Table 14-1 (1/2, 5/8, 1-1/8, 1-3/8, 2-1/4, 2-3/4 and 3 in.): enter the hole and the washer. Left blank, ILX uses interpolated or extrapolated values and the recommendation **rod is not in Table 14-1: hole and washer entered by the engineer** stays open, stating they are not Manual values.
- Plate: 1/2 in. clear from hole to plate edge, washers fit and do not overlap, projection past the column for the weld, a recommended N ≥ d + 6 in. and B ≥ bf + 6 in., tp ≥ 3/4 in. (1/2 in. for posts).
- Rods: spacing ≥ 4 da (6 da torqued), edge distance ≥ cover + da/2 (and 6 da torqued), with a 6 da recommendation. Also clearance to the column for washer, nut and weld, embedment within the pier depth, and hook length 3 da ≤ eh ≤ 4.5 da.
- Grout: depth for the leveling nut and washer, a recommended 1 in. minimum, and notes on grout and leveling.

**OSHA** (OSHA 29 CFR 1926.755(a) (column anchorage))

- At least four anchor rods, unless the column is a post.
- Erection stability: 300 lb at 18 in. from the column face, × 1.6 in LRFD. It is checked about each axis on the rod couple and, for fillet-welded W columns, on the flange weld couple.

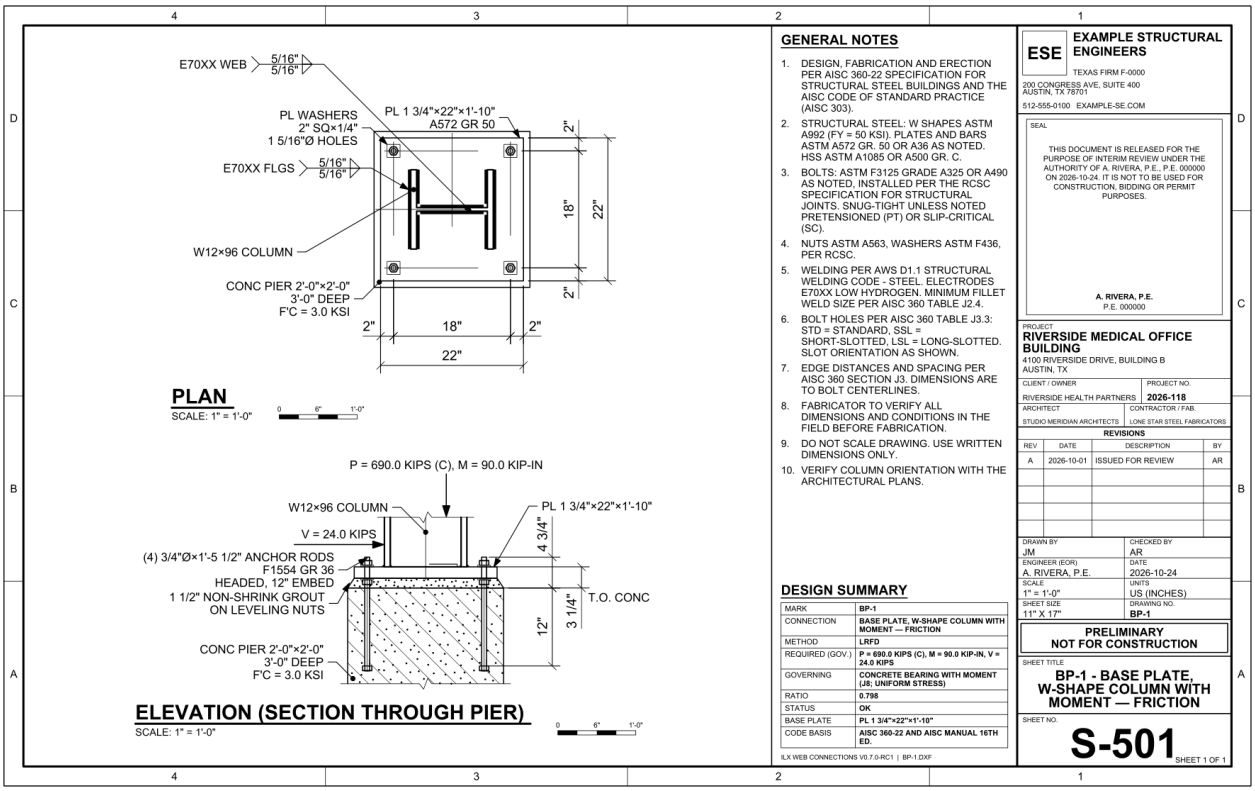
**Fatigue (Appendix 3)** (the **Cyclic loading (fatigue)** section; see Fatigue (Appendix 3)). The cyclic range is a fraction of the live load (ML when a moment acts, else PL) or the range of M (or P) entered. The details: the anchor rods in tension (Table A-3.1 Case 8.5, G, on the tensile stress area: the grip holds grout, so the rods carry the whole range, Section 3.4(b)); for a W column with flange tension Tf = M/(d − tf) − P/2, the flange at its fillets to the plate (Case 5.7, C and C′) and those fillets (Case 8.2, F), or the CJP weld (Case 5.4). A base in compression without rod tension has no tensile stress range; the result says so (Commentary 3.2).

## Drawings, report and sheet

The editor draws two views:

- **PLAN**: the pier outline, base plate, column section with its welds and weld symbols, anchor rods with plate washers and nuts, the shear lug (hidden) and the edge-distance and rod-spacing dimensions.
- **ELEVATION (SECTION THROUGH PIER)**: column stub, base plate, grout pad, the pier in section, the anchor rods with nuts, washers, leveling nuts and the embedded end, the shear lug, and the rod projection and embedment dimensions. Load arrows are shown when there is room.

Select an element in a drawing to open its inputs (see Drawings).




The drawing sheet places both views beside the general notes and the design summary: mark, connection, method, required strength (P, M, V of the governing combination), governing check, ratio, status and base plate. **Drawing (DXF)** gives the same sheet as a DXF file.

The calculation report is titled **Column Base Plate**. Its input summary covers the column, loads, plate, concrete support, bearing, grout, anchor rods, holes and washers, rod projection and length, shear transfer and welds, and the governing code for anchorage. See Reports.

## Auto-design and batch for base plates

**Auto-design** searches:

- **Plate thickness** from 1/2 in. to 4 in. (the thinnest plate that passes);
- **Rod diameter**, 3/4 in. and larger;

- **Embedment hef**: 9, 12, 18 or 24 in.

It keeps the column, plate size N × B, rod pattern, pier, materials and loads. Base-plate fields cannot be pinned. The objectives are those of the other types (see Auto-design, compare and batch):

- **Lightest (steel weight)**: plate plus rods;
- **Fewest bolts**: the rod count is fixed, so this ranks like the weight;
- **Lowest cost (fabrication estimate)**: steel plus a per-rod allowance.

If no candidate passes, enlarge the plate or the pier yourself, then run it again.

**Compare** works as for other types. **Batch** runs a schedule of BP marks. The default columns are `mark`, `column`, `PD` and `PL`; add any other input field name, such as `N`, `B`, `MD` or `rod_d`, with **+ Column**. **Auto-design each mark** sizes the plate thickness, rod diameter and embedment of each row for its own loads.

## Limitations and documented deviations

- **Biaxial bending** follows DG1 3rd Ed. §4.3.11 (Eq. 4-69). DG1 states that interaction for axial compression with the shear on a lug or friction. Under uplift or with the rods in shear ILX still reports it, with a note, and checks every elastic rod force.
- **Symmetric pattern.** Two rod lines, symmetric about the column; moment and shear are designed by magnitude. Perimeter rod patterns and the rows along the web of DG1 Ex. 4.7-7 cannot be entered.
- **Not covered**: embedded column bases, stiffened base plates, shear by plate-edge bearing (DG1 Ex. 4.8).
- **Seismic design** covers uniaxial bending, as DG1 Chapter 6 does. The AISC 341-22 D2.6b shear is generated for the strong axis only when you give the story height; otherwise it is the shear you enter. Anchor chairs and rod sleeves for the stretch length are not modelled. The ASD weak base is an ILX conversion of the LRFD-only DG1 example (see above).
- **Plate bending.** DG1 3rd Ed. App. B.4 two-way bending is an option for the n strip at the bearing interface. The tension interface uses the full B; the 45° width is advisory only.
- **Triangular method.** It is the DG1 2nd Ed. App. B.5 method. Whether the 3rd Ed. keeps it is not confirmed.
- **Shear lug bearing area.** Aef,sl uses the full lug depth below the grout. Some programs limit it to 2 tsl.
- **Anchor reinforcement.** An entered As stays an advisory strength, φ As fy. Designed bars (or pier bars) are checked for placement and development; shear hairpins are not designed, and pier ties are taken as developed by their 135° hooks.
- **App. C stiffness** covers exposed bases only (no blockout or embedded bases). **Double-nut joints**: fatigue (App. A.1.2) and punching below the heads are not checked.
- **Grout strength** is not used in bearing; the concrete f′c governs.
- **Pier and footing** are calculated alike, with the support concentric with the plate.
- **Rods not in Table 14-1.** Table 14-1 lists 3/4 to 2-1/2 in. rods (no 1-1/8, 1-3/8 or 2-1/4 in.). For other rods the hole and washer are yours; the prefilled values (holes d + 1-1/4 in. above 2-1/2 in., following Commentary C-J9.1; washers extrapolated) are not Manual values and the report says so until you enter them.
- **F844 washers** are drawn round in the plan (outside diameter as entered); plate washers are drawn square. The callout names them.
- **Not checked**: rod bearing on the plate hole and the column end finish (M2.6).
- **Verification.** Against AISC Design Examples J.6 and K.9, Design Guide 1 2nd Ed. Examples 4.1 to 4.11 and B.5, the SteelTube Institute HSS examples and published ACI anchorage examples. Three documented differences:
  - DG1 Ex. 4.7, ASD rod tension: DG1 rounds Y before computing T.

- DG1 Ex. 4.9, lug bearing: DG1 uses ACI 349; ILX uses ACI 318-19 17.11.
- DG1 Ex. B.5.2, rod tension: DG1 uses a bearing stress different from the one it computes.

See Verification.

CHAPTER 26

# Designing an embedded column base

The **Embedded column base** type (Wizard group *Column bases*, mark prefix `EB`) checks a W-shape column set into a concrete footing, pier or grade beam, so that the foundation itself holds the column's moment. It follows AISC Design Guide 1, *Base Connection Design for Steel Structures*, 3rd Edition (2024), Chapter 5, and for Type II connections Hassan and Kanvinde (2023), *AISC Engineering Journal*, 4th quarter. Bending is about the column's strong axis.

Choose the **connection type**:

- **Type I** (the default). The concrete below the column end is there to set the column, not to carry vertical load. The moment and shear are resisted by the column flanges bearing sideways on the concrete, plus any horizontal bars attached to the flanges (DG1 §5.2.1).
- **Type II**. The concrete below the embedded base plate is designed for vertical stress. The base plate adds a second mechanism, vertical bearing, to the flange bearing: M = MHB + MVB (Hassan and Kanvinde 2023). Type II needs the embedded base plate.

| Configuration | What is checked | Published check |
|---|---|---|
| Plain embedment with face bearing plates | Moment strength MHB at the shear (Eq. 5-6), required depth, face plate bearing and thickness | DG1 Example 5.3-1 |
| Bars welded to the flanges | As above, with the bars in tension on one flange and compression on the other; development length | Hassan and Kanvinde Test 1 |
| U-bar hairpins | As above, with the bars in tension only | Hassan and Kanvinde Tests 2, 3 |
| Embedded base plate | Wider bearing width bj; uplift through the plate (plate bending, bearing, concrete breakout) | — |
| Anchor rods below the plate | Uplift by ACI 318-19 Chapter 17 on the rod group | — |
| Type II | MHB + MVB; bearing, plate bending and punching of the concrete below | Grilli et al. tests 1G to 5G |

## Embedded base inputs

- **Loads**: axial force P (compression positive, tension negative), shear V and strong-axis moment M at the top of the foundation. Enter service dead and live loads, required strengths, ASCE 7 combinations or load cases. Both LRFD and ASD are available.
- **Column**: the connection type (I or II), the W-shape and its material. **Seismic design: strong base** replaces the moment with 1.1 Ry Fy Z of the column (DG1 §6.4.1, AISC 341-22 D2.6c(b)(1); ÷ 1.5 in ASD); enter P and V from

the overstrength combinations. You enter Ry; the line under the input shows the AISC 341-22 Table A3.2 value for the column grade, for reference. With the strong base you can also choose:

- **Strong-base moment**: 1.1 Ry Fy Z / αs (default) or the lesser of that and the overstrength moment you enter (AISC 341-22 D2.6c(b)(2), only when a ductile limit state of the base or the foundation controls; the column flexure in the embedment is then checked);
- **Story height H**: ILX generates the D2.6b shear, the lesser of 2 Ry Fy Z / (αs H) and your shear, at least 0.7 Fy Z / (αs H) (turn the minimum off for Exceptions (a) and (b); limit it by analysis for Exception (c));
- **Special cantilever column system** (E6): the column axial force at most 0.15 Fy Ag, the protected zone 2 d above the foundation (hatched on the elevation) and demand-critical welds.

- **Foundation and embedment**:
  - the concrete strength f'c;
  - the foundation width bw, perpendicular to the plane of bending;
  - the foundation length in the plane of bending and its depth. They are optional, but the depth is needed for the checks of the concrete below the column (punching), and the length for one-way shear;
  - the embedment depth (0 gives the required depth, rounded up to 1 in.).

  **Confined concrete** takes the bearing limit under the plates as 1.7 f'c, as DG1 does. **Limit damage** uses Mn = 0.8 MHB (0.8 (MHB + MVB) for Type II), the load at which the concrete starts to crack and crush in tests.
- **Face bearing plates**: plates between the flanges at the top of the foundation that carry axial compression. Enter their thickness and material.
- **Embedded base plate**: a plate on the column end, with width B, length N and thickness. It widens the concrete panel that bears on the flanges, bj = (bf + B)/2, carries axial tension by bearing up on the concrete above it, and in a Type II connection bears down on the concrete below.
- **Anchor rods below the plate**: none, 2 (on the axis along N) or 4 (a rectangle), with:
  - the rod diameter and material;
  - the embedment of the heavy hex nut below the underside of the plate;
  - the spacing along N and along B;
  - cracked or uncracked concrete.

- **Attached horizontal bars**:
  - welded to the flanges (they work in tension and compression) or U-bar hairpins around the flanges (tension only);
  - the bar size and yield strength;
  - the number of bars per flange in a top row and a bottom row, and the depth of each row below the top of the foundation;
  - the clear cover at the foundation ends.

## Embedded base checks

- **Moment strength of the embedment.** ILX checks Mu ≤ φMn with φ = 0.75 (DG1 §5.3; ASD Ω = 2.00 = 1.5/0.75):
  - the bearing stress is fb = 1.54 √f'c (bw/bf)^0.66, at most 1.7 f'c (Eq. 5-1);
  - fb acts over a top block on one flange and a bottom block on the other (Eqs. 5-2 to 5-4);
  - MHB is the closed form of Eq. 5-6 at the actual shear, with the bar forces Asr Fysr. A row of bars welded to both flanges counts twice, 2 n Asr Fysr: tension on one flange and compression on the other. The report says so.

The required embedment depth is reported as a note. With bars, ILX also checks that the neutral axis lies between the two rows, which Eq. 5-6 assumes.

- **Type II: vertical bearing.** ILX adds MVB to MHB (Hassan and Kanvinde Eq. 20):
  - The stress under the plate is idealized as three steps. The two outer steps are Y = 0.3 N wide (Eqs. 8, 9).
  - Without attached bars (Scenario 1), the uplifting end is held by a concrete breakout, Ft = (40/9) √f'c A35 / √dembed with A35 = (B + 3 dembed)(0.3 N + 1.5 dembed) − 0.3 N B (Eqs. 12, 13).
  - Then MVB = (Ft + P Y/N)(N − Y) (Eq. 11). Compression P adds to MVB and tension reduces it.
  - With attached bars (Scenario 2), the tension field behind the column removes this restraint, so MVB = 0.
  - Scenario 3, bars with vertical stirrups, is not modeled. ILX uses MVB = 0, which is conservative.

  The paper gives no resistance factor. ILX uses the DG1 φ = 0.75 on MHB + MVB. The demand is shared between the two mechanisms in proportion to their strengths. The share on the plate gives the outer-step stress on the concrete below. The concrete below is then checked for bearing, plate bending and punching, as for compression below. A warning shows when the embedment depth to column depth ratio or f'c is outside the tested range.
- **Attached bars.** Development length by ACI 318-19 Table 25.4.2.3 ("other cases"), with ψt = 1.3 for a row with more than 12 in. of concrete below it. ILX compares it with the length available beyond the flange when you give the foundation length. Welding of the bars (ASTM A706, AWS D1.4) is noted, not designed.
- **Axial compression with face bearing plates.** The face bearing plates are checked for:
  - concrete bearing, φ fp(max) A1 with A1 = (bf − tw)(d − 2tf) and φ = 0.65;
  - thickness, tmin = l √(2Pu / (0.9 Fy A1)) with l = √(d bf)/4 (Eqs. 5-7, 5-8);
  - the AISC 341 F3.5b.4 minimum, 0.75 tw and 3/8 in. This is a recommendation, and a requirement for strong-base seismic design.

  DG1 calls for CJP groove welds.
- **Axial compression through the bottom.** This applies to Type I without face bearing plates, and always to Type II. ILX calculates the load path below the column:
  - bearing of the embedded base plate (or of the bare column end) on the concrete below. This uses ACI 318-19 22.8 and AISC J8: 0.85 f'c √(A2/A1) ≤ 1.7 f'c, with A2 within the depth below the plate and the foundation plan, and φ = 0.65;
  - bending of the base plate on that bearing (DG1 §4.3.1 cantilevers m, n and λn');
  - two-way (punching) shear of the concrete below, ACI 318-19 22.6.5.2, with the size factor λs. The effective depth is the depth below the plate less the cover and half a bar;
  - one-way shear of the foundation at d from the plate, when its length is given.

  A check fails only when the calculated strength is too small.
- **Axial tension.** Tension needs an embedded base plate, which bears up on the concrete above it. ILX checks:
  - the plate bending between and outside the flanges (§4.3.1 cantilevers);
  - concrete bearing on top of the plate;
  - the breakout of the concrete above the plate. This applies DG1 Eq. 5-9, a 35° cone, to the plate's uplift. It is an ILX extension.

  With **anchor rods** below the plate, ILX also computes the rod group by ACI 318-19 Chapter 17, using the same engine as the base plate:
  - steel strength in tension (17.6.1);
  - concrete breakout of the group (17.6.2). The embedment hef runs from the top of the foundation to the nuts, and the foundation edges limit the projected area;

  - pullout of the heavy hex nut (17.6.3);
  - side-face blowout where hef > 2.5 ca1 (17.6.4).

  The uplift resistance is the larger of the plate path and the anchor path. Each path's parts are listed in the report. Without an embedded base plate, the calculated uplift resistance is zero.
- **Column within the embedment.** ILX checks the column's own strength:
  - flexure for the moment Mu + Vu²/(2 fb bj) where the column shear is zero (F2 or F3);
  - axial force and flexure together (H1-1, no buckling in the concrete).

  The shear in the column web between the two bearing blocks is **not** checked. The bearing load spreads through the flanges and plates into the concrete over a large area, and DG1 has no such check. The report shows a line explaining this.
- **Design basis.** The report states the factors used:
  - LRFD: φ = 0.75 on the embedment moment, 0.65 on concrete bearing and 0.90 on the steel;
  - ASD: Ω = 2.00, 2.31 and 1.67;
  - ACI 318 shear and anchor items: φRn, or φRn/1.6 with ASD loads.

## Embedded base limits

- Type II follows a paper validated on ten tests (W14X370 and W18X311 columns, 20 and 30 in. embedments, about 4 ksi concrete). Use judgement outside that range; ILX warns.
- Scenario 3 of Type II (vertical stirrups with attached bars) is not modeled; MVB = 0 is used.
- Headed studs on the flanges are not designed.
- The foundation's flexure and reinforcement are outside ILX. Punching and one-way shear below the column are checked when compression goes through the bottom.
- Weak-axis bending is not covered.
- The rotational stiffness of the base is not computed; see DG1 Appendix C.2.3.
- Erection (DG1 §5.4): the setting nuts, shims or setting plate must resist the forces of placing the concrete around the column.

## Embedded base drawings

- The **elevation** shows the column embedded in the foundation, with:
  - the face bearing plates;
  - the embedded base plate and the anchor rods below it, with their nuts;
  - the two DG1 bearing zones, on opposite flanges;
  - for Type II, the outer vertical bearing steps above and below the base plate;
  - the attached bars on both sides;
  - the embedment and bar depths.
- The **plan at the top of the foundation** shows the column section with the face bearing plates, the embedded plate and anchor rods (hidden), the bars, and the foundation width.

## Embedded base verification

- **DG1 Example 5.3-1** is reproduced row by row: W14X176, 22 in. embedment, bw = 60 in., f'c = 4 ksi, Pu = 250 kips, Vu = 96 kips, Mu = 700 kip-ft. The rows include fb, the three terms of Eq. 5-6, MHB = 958 kip-ft, φMHB = 719 kip-ft, A1, φPp = 827 kips, l and tmin = 0.941 in. DG1 prints the resistance factor as "(0.7)". The product, 719 kip-ft, is 0.75 × 958, the φ = 0.75 of §5.3.
- **Hassan and Kanvinde (2023), Table 1.** ILX reproduces the model strength of the ten tests within 3 %:
  - five Type II tests by Grilli et al. without bars, 1G to 5G (Scenario 1, including tension and zero axial load);
  - three tests with attached bars, Tests 1 to 3 (Scenario 2).

  Tests 4 and 5 (Scenario 3, stirrups) are listed as a documented deviation: ILX takes MVB = 0 and is conservative.

See the Verification page.

CHAPTER 27

# Designing a braced-frame column base

The **Braced-frame base** type (Wizard group *Column bases*, mark prefix `BF`) designs a column base plate at the foot of a braced frame, where a vertical brace lands on a gusset welded to the base plate and to the column. It combines two guides:

- AISC Design Guide 29, *Vertical Bracing Connections*, §4.3 and Example 5.12, for the brace, the gusset, the welds and the column;
- AISC Design Guide 1, 3rd Edition, Examples 4.7-7 and 4.7-8, for the base plate, the anchor rods, the shear transfer and the concrete (ACI 318-19 Chapter 17). These checks are the same as the **Base plate** type's.

| Gusset on the column | What is checked | Example |
|---|---|---|
| Flange (strong axis) | Gusset-to-flange welds with the 1.25 ductility factor, column web local yielding and crippling, web and flange welds to the plate | DG29 Ex. 5.12.1 |
| Web (weak axis) | Gusset-to-web welds without the 1.25 factor, the web yield line (DG29 Eq. 4-20), the ⅝ tg weld to the plate, flange welds for uplift and shear | DG29 Ex. 5.12.2, DG1 Ex. 4.7-7 |

## How the brace force is split

The work point sits on the column centre line, a height **e** above the top of the base plate. The brace force P (positive in tension) has a horizontal part H and a vertical part V. Following DG29 Figures 4-22 and 4-23:

- the gusset edge on the column carries all of **V** and a horizontal force **Hc = (H e + V ec) / β**, where β is the height of the middle of that edge above the plate and ec is half the column depth (0 for a gusset on the web);
- the gusset edge on the base plate carries **Hb = H − Hc**.

Putting the work point on the top of the plate (e = 0) and the gusset on the web gives the smallest forces in the column. The base plate is designed for the column's own forces plus the brace reaction: axial P − V (uplift for a tension brace), shear H and moment H e, in the plane of the brace (along N for a flange gusset, along B for a web gusset).

## Braced-frame base inputs

The editor is the base plate editor with three more sections after the loads.

- **Loads**: the column's own axial force, moment and shear, as for a base plate. Enter the brace force in the *Brace force and geometry* section: one value for required strengths, the service components D, L, Lr, S, W, E for load combinations, or one value per load case. With **Brace force reverses** ticked, every case is also checked with the brace in compression.
- **Brace force and geometry**: the column face the gusset is welded to (flange or web), the brace bevel (vertical and horizontal legs) and the height e of the work point above the plate.

- **Seismic brace forces** (AISC 341-22 D2.6a, D2.6b(a), D2.6c(a)): the base forces take the brace components from the required strengths of the brace connection. **SCBF** replaces the brace force by Ry Fy Ag, min(Ry Fy Ag, (1/0.877) Fne Ag) and 0.3 of that (F2.6c), from the brace Ry (0 = Table A3.2), length (end to end) and K; **OCBF** keeps your overstrength force but limits it to the expected brace strength (F1.6a Exception); **BRBF** uses ω Ry Pysc and β ω Ry Pysc (F4.6c.1) with Pysc, β and ω from the brace tests. All are divided by αs = 1.5 in ASD. The column forces you enter are added as before. With the **story height H** of the seismic design section, the column's own shear is limited to 2 Ry Fy Z / (αs H) (D2.6b(b)) and the total shear is at least 0.7 Fy Z / (αs H) (D2.6b(c)).
- **Brace connection**: a W brace whose web is spliced to the gusset with four angles (DG29 Ex. 5.12), or a double angle, single angle, WT or slotted HSS bolted or welded to the gusset. Bolts, lines, pitch, gage and edge distances; the distance from the work point to the first gusset bolt (0 places the connection just clear of the column and the plate).
- **Gusset**: thickness and material, the edge lengths on the column and on the plate, the corner clip, the two fillet sizes, and the buckling factor K (0.6 for this extended gusset, DG29 Table C-1) with the buckling length (0 = measured from the Whitmore section to the column face or the plate along the brace).

## Braced-frame base checks

- **Brace and its connection**: brace yielding and rupture, bolt bearing and tearout, block shear, the splice angles (yielding, rupture, block shear, bolt shear) and bearing and block shear on the gusset.
- **Gusset**: Whitmore yielding in tension, Whitmore buckling in compression, shear and tension yielding along the column, the Neal interaction, shear yielding along the plate.
- **Welds**: the gusset-to-column and gusset-to-plate fillets (Table J2.4 minimum sizes; ⅝ tg at a web), the column web and flange fillets to the base plate for the brace forces, with the base metal check of Manual Eq. 9-3.
- **Column**: web local yielding and crippling (flange gusset), the web yield line and web shear (web gusset).
- **Base plate, anchor rods and concrete**: every check of the base plate type for the combined reactions.

## Braced-frame base drawings

The sheet shows the base plate **plan** with the gusset, and an **elevation in the plane of the brace**: the column, base plate, grout, concrete, anchor rods, the gusset with its clip, the brace and its splice angles cut beyond the connection, the bolts, the gusset welds with their symbols and the work point.

## Braced-frame base limits

- The gusset is designed with the DG29 force field: no frame-distortion (seismic) forces. The seismic brace forces above give the D2.6 base forces; the other AISC 341 requirements of the brace connection (hinge zone, gusset welds) are checked in the bracing gusset type.
- Two-direction loading (a brace in each direction at the same column) is not combined; design each direction.
- The column-to-plate flange welds take the uplift less any column compression entered in the same case.

See Verification for the published examples reproduced by this type.

CHAPTER 28

# Components and calculators

## What the calculators are

Six calculators design a single component or joint, rather than a whole beam-end connection. They use the same editor, checks, drawings, report and drawing sheet as the connection types.

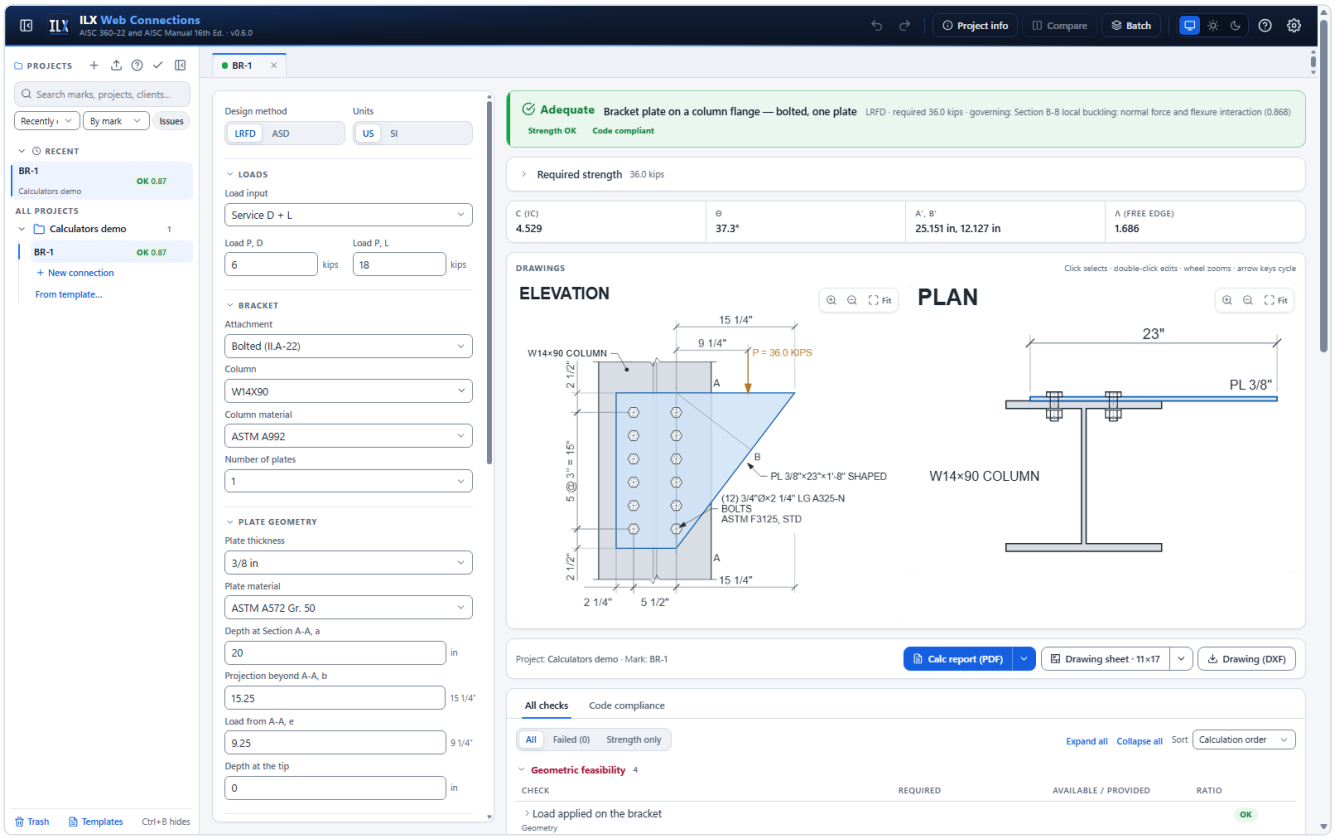
| Calculator | What it designs | Basis | Design Examples reproduced |
|---|---|---|---|
| **Bracket plate** | A shaped plate on a column flange, bolted or welded, with an eccentric load | Manual Part 15, Parts 7 and 8 | II.A-22, II.A-23 |
| **Bolt group** | Any bolt pattern under an in-plane eccentric load at any angle | Manual Part 7 (IC and elastic methods) | II.A-24, II.A-25 |
| **Weld group** | A line, two lines, C, L or box fillet weld group (Manual Tables 8-4 to 8-11a) under an eccentric load | Manual Part 8 (IC and elastic methods) | II.A-26, II.A-27 |
| **Bolted joint** | A lap, splice, tee/flange or double-angle joint in shear, tension or both | AISC 360-22 J3, J4, J5 | J.3, J.4A, J.4B, J.5, II.D-3 |
| **Welded joint** | Fillet or PJP groove welds loaded at an angle, in a lap or tee joint | AISC 360-22 J2 | J.1, J.2A, J.2B |
| **Pin-connected member / eyebar** | A plate with a pin hole, or an eyebar, in tension, with one or more plies on the pin | AISC 360-22 D5, D6, J7 | D.7, D.8 |

To start one, choose **New connection**. In the wizard, pick a card under **Components & calculators**, give it a mark and choose **Create**. The calculator opens with the inputs of its first Design Example, so its first result can be checked against the published one.

Every calculator has:

- **LRFD or ASD**, and **US or SI** units. Changing the units converts every value.
- **Loads** as service D + L, a required strength, or every ASCE 7-22 combination. With combinations, each check reports its governing combination.
- A status of **Adequate**, **Not adequate** or **Not feasible**, using the same rules as the connections.
- Checks with their AISC clause and a fix hint, and the code-compliance list (see Checks and compliance).
- Two drawings with click-to-edit elements, plus the report and the drawing sheet in PDF and DXF (see Drawings and Reports).

# Bracket plate calculator




The plate is lapped on the column flange. Its depth at the attachment line is **a**. It projects a distance **b** beyond that line, to a tapered free edge. The load acts downward at **e** from the attachment line. With two plates, enter the total load. The plates share it, and every strength is for both plates.

**Section A-A** is the outer bolt line or, for a welded bracket, the end of the weld returns. ILX checks:

- flexural yielding, with Mn = Fy t a²/4 (Manual Eq. 15-2);
- flexural rupture through the holes (Eq. 15-3);
- shear yielding and shear rupture.

**Net section** chooses how the flexural-rupture section is calculated. Choose the holes as drawn, or Manual Table 15-2, which conservatively assumes a 1½ in. edge distance. Design Example II.A-22 uses Table 15-2.

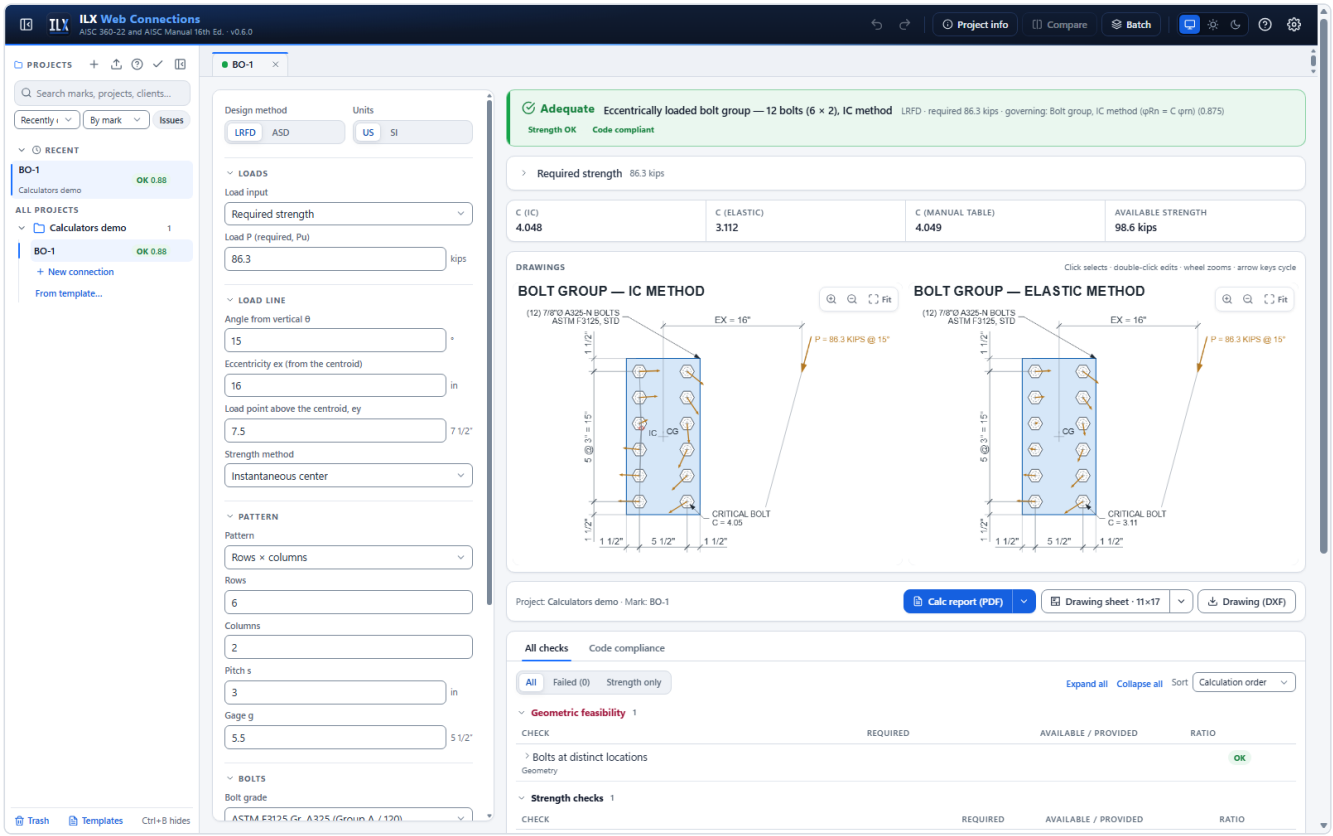
**Section B-B** runs perpendicular to the free edge. ILX checks its shear yielding. It also checks local yielding or buckling of the free edge, with λ, Q and Fcr from Manual Eqs. 15-13 to 15-18, and the interaction Nr/Nc + Mr/Mc ≤ 1.0 (Eq. 15-10).

The attachment depends on how the plate is connected:

- **Bolted.** The bolt group is solved by the instantaneous-center method. Each bolt's strength is the least of its shear strength, plate bearing, plate tearout toward the top edge and column-flange bearing. ILX also checks block shear of the plate, the spacing and edge distances, the edge distance to the flange tip and the wrench clearance to the column web.
- **Welded.** The weld is a C-shaped group: the vertical weld plus returns along the top and bottom edges. ILX gives C, the required size Dmin (Manual Eq. 8-30), the minimum and maximum fillet sizes, and the base-metal thickness of the plate and the flange.

The column itself is assumed adequate, as in the Design Examples.

## Bolt group calculator




Enter the bolt pattern as rows × columns at pitch **s** and gage **g**, or as custom coordinates (one "x, y" pair per line).

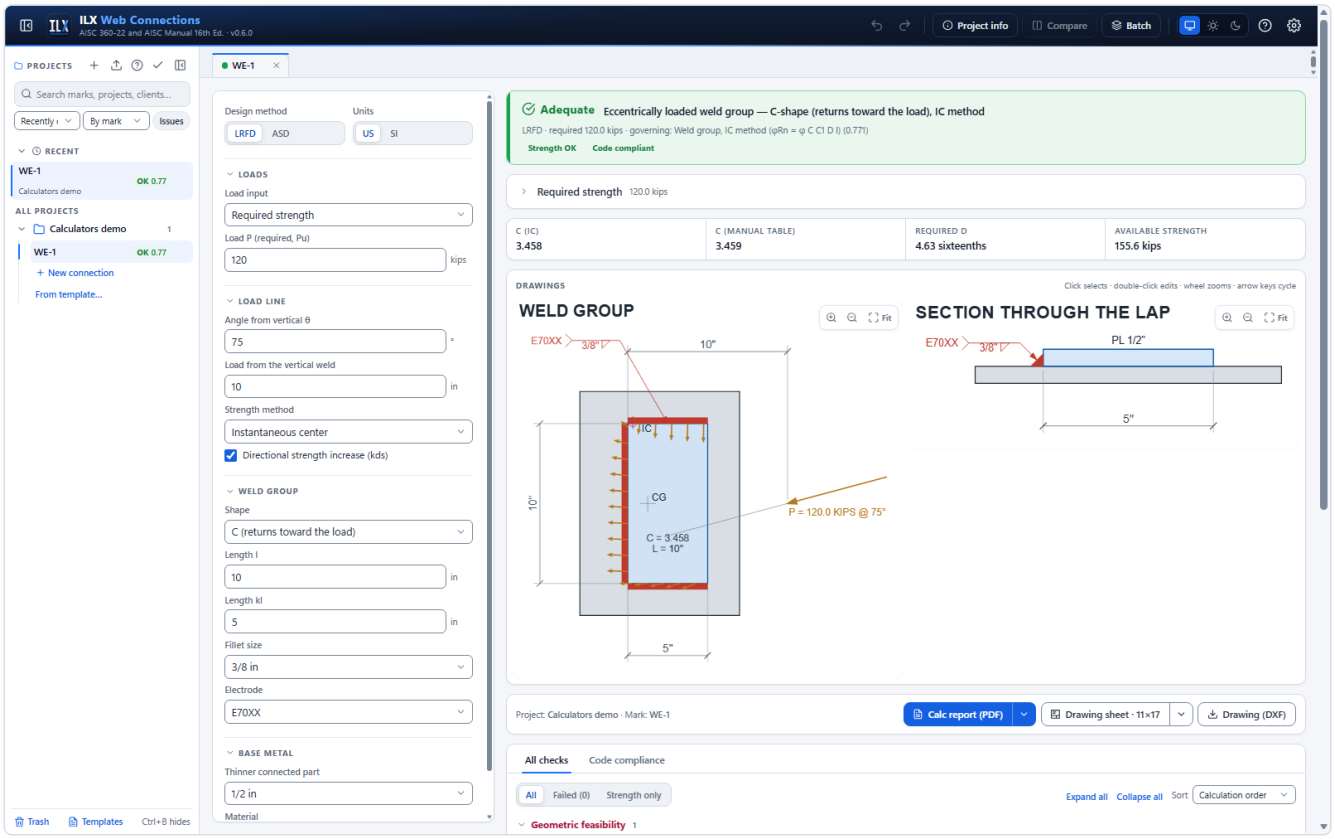
The load acts at **θ** from the vertical, measured clockwise from straight down. Its line passes a distance **ex** to the right of the centroid, at a height **ey** above it. A load at 15° applied at the top bolt row (II.A-24, Solution B) is ex = 16 in., ey = 7.5 in. and θ = 15°.

| Result | Meaning |
|---|---|
| **C (IC)** | Instantaneous-center coefficient for the actual pattern and load line. The drawing shows the IC and every bolt force. |
| **C (elastic)** | 1/r of the critical bolt by the elastic method (Manual Eqs. 7-2 to 7-8). It is usually more conservative. |
| **C (Manual table)** | The value you would read from Manual Tables 7-6 to 7-13, interpolated between the tabulated eccentricities and angles. It is shown for comparison. Table 7-6 (one vertical row) starts at ex = 1 in., the other tables at 2 in. A test compares the IC method with every printed C of Tables 7-6, 7-7 and 7-8 (7,692 values): all agree within 1 % or 0.01, except a few printed values at 75°, s = 6 in., ex = 2 in. (up to 2.3 %). |

**Strength method** chooses which method sets the status. The other method is listed as a comparison. Bolt shear is calculated for any grade, threads N or X, and single or double shear.

To include bearing and tearout, enter the thinnest ply and its edge distance in the load direction. Tearout then uses that edge distance for every bolt, conservatively, as II.A-22 does. Slip-critical bolts add a slip-resistance check, C × φrn.

# Weld group calculator




Choose a shape and enter its lengths **l** and **kl**, as the Manual table for that shape names them. The load line is **x** from the vertical weld at the centroid level, at θ from the vertical (positive θ turns the downward load clockwise, so it leans toward the vertical weld).

| Shape | l and kl | Manual table |
|---|---|---|
| Single line | l vertical | 8-4 (k = 0, one line = C/2) |
| Two vertical lines | l vertical, kl apart | 8-4 |
| Two horizontal lines | l = the spacing, kl = the line length (Table 8-5 names them the other way; ILX converts) | 8-5 |
| Box, l vertical | l tall, kl wide | 8-6 |
| Box, l horizontal | l wide, kl tall | 8-7 |
| C, returns toward / away from the load | l vertical, returns kl | 8-8 / 8-9 |
| L, return toward the load | l vertical, bottom return kl | 8-10 (θ ≤ 0), 8-10a (θ > 0) |
| L, return away from the load | l vertical, bottom return kl | 8-11 (θ ≥ 0), 8-11a (θ < 0) |

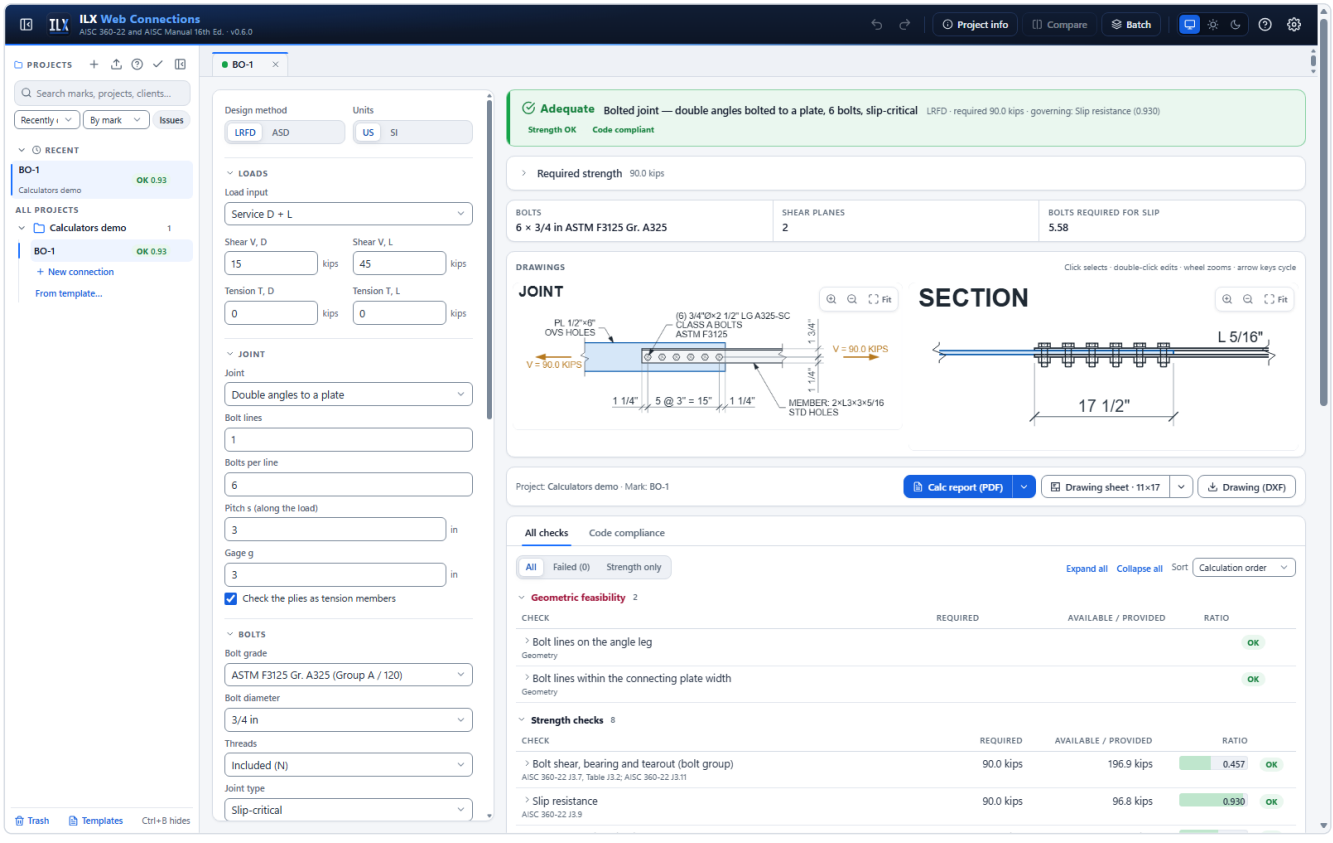
The L is the only unsymmetric shape, so the Manual tabulates both inclinations of the load: the "a" tables are the load leaning the other way. ILX picks the table from the sign of θ. For an L with the return at the top, change the sign of θ.

ILX solves the IC method with the directional strength increase (kds; it can be turned off) and gives:

- **C**, with Rn = C C1 D l (Manual Eq. 8-30);
- the **required size** Dmin;
- the **Manual-table value**, interpolated in k, a and the angle;
- the **elastic-method** strength (Manual Eqs. 8-11 to 8-17, with kds = 1.0).

It also checks the minimum and maximum fillet sizes, the minimum length (4w) and the base-metal thickness, tmin = 3.09 D / Fu.

## Bolted joint calculator




Pick the joint:

- a **lap** (single shear);
- a **splice** with two splice plates (double shear);
- a **tee or flange** joint, where the bolts carry tension and shear;
- **double angles** bolted to a plate (double shear).

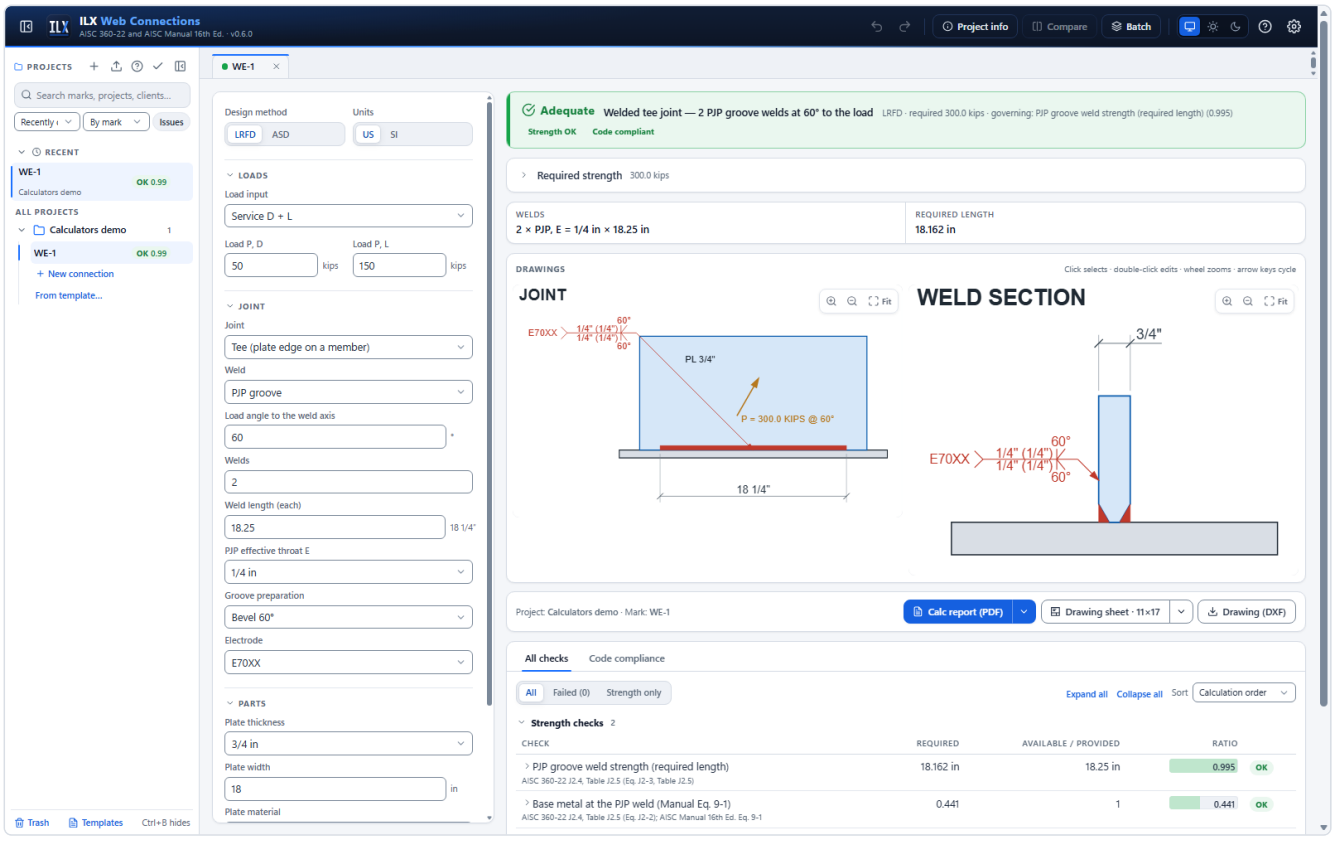
The bolts lie in **lines** parallel to the load, with **bolts per line** at pitch s. Ply A is the member, which ends beyond the first bolt. Ply B is the connecting plate, which ends beyond the last bolt. Each ply tears out toward its own end.

| Check | Clause |
|---|---|
| Bolt shear, bearing and tearout: Σ of each bolt's least value | J3.7, J3.11 |
| Bolt tension, and combined tension and shear in bearing-type joints (F'nt; the 30 % exemption is applied) | J3.7, J3.8 |
| Slip resistance, with φ / Ω from the hole type relative to the load, the J3.10 factor ksc with tension, and the bolts required | J3.9, J3.10 |
| Fillers: the shear reduction for bearing-type joints, and hf for slip-critical joints | J5.2, J3.9 |
| Member and plate as tension members: yielding (Whitmore section for the plate), rupture with the shear-lag factor U, block shear (L and U blocks) | J4.1, D3, J4.3 |
| Spacing, edge distances with the Table J3.5 increments, maximum spacing and edge, hole use, pretension, washers | J3.3 to J3.6, RCSC 6 |

Slot directions are relative to the drawing, where the load is horizontal. **Short slot, vertical** is a slot perpendicular to the load (J.4A). **Long slot, horizontal** is parallel to it (J.4B).

**Cyclic loading (fatigue)** designs the joint for fatigue (Fatigue (Appendix 3)): both plies at the bolt holes under the shear (Table A-3.1 Case 2.1 or 2.2, or Case 1.5 for snug-tight joints) and the bolts in tension (Case 8.5, G).

## Welded joint calculator



A **lap** joint has fillet welds along the edges of the lapped plate. A **tee** joint has fillet or PJP groove welds along the edge of a plate on a member. Enter the angle between the load and the weld axis: 0° is longitudinal and 90° is transverse.

- **Fillet welds.** The strength is 0.60 FEXX × 0.707w × kds (Eqs. J2-4, J2-5). Long end-loaded welds in a lap get the factor β (Eq. J2-1). ILX gives the strength per inch, the required length and the required size. The base metal is checked by shear rupture of the lapped plate (Eq. J2-2) or by the Manual Part 9 minimum thickness. The fillet size limits and the lap length (≥ 5t) are checked.
- **PJP groove welds.** Enter the effective throat **E** and the preparation. ILX checks the shear and tension strengths per inch (Table J2.5), combined for the load angle, and the base-metal interaction (Manual Eq. 9-1). It also checks the minimum throat (Table J2.3).

The drawings show the true weld cross-section and its AWS symbol, both from one weld definition.

**Cyclic loading (fatigue)** designs fillet-welded joints for fatigue (Fatigue (Appendix 3)): the fillets (Table A-3.1 Case 8.2, F) and the loaded plate at the weld ends (Case 4.1 for longitudinal welds, load angle under 45°; Case 5.7, C and C″, for transverse welds). PJP joints are not covered.

## Pin-connected member / eyebar calculator

Choose the member: a **pin-connected plate** (AISC 360-22 D5, Example D.7) or an **eyebar** (D6, Example D.8). Enter the thickness t, the width w (the plate width at the hole, or the eyebar body width), the pin diameter d and the hole diameter dh, the materials and the number of **plies** on the pin. Identical plies share the load equally.

- **Pin-connected plate.** Also enter the end distance **a** from the hole edge to the end of the plate and, when the corners are cut at 45°, the distance **c** from the hole edge to the cut. ILX checks tensile rupture on 2 t be with be = 2t + 0.63 in. (2t + 16 mm) and not more than b = (w − dh)/2 (Eq. D5-1), shear rupture on Asf = 2t (a + d/2) (Eq. D5-2), bearing on the projected area t d (Eq. J7-1) and yielding on w t (Eq. D2-1). The D5.2 limits are checked: w ≥ 2 be + d, a ≥ 1.33 be, c ≥ a and, when the pin moves under full load, dh ≤ d + 1/32 in.
- **Eyebar.** Also enter the head diameter **dhead** and the transition radius **R**. The head is concentric with the hole. ILX checks yielding of the body (w ≤ 8t) and the D6.2 limits: t ≥ 1/2 in. (unless external nuts snug the plies), d ≥ 7/8 w, dh ≤ d + 1/32 in., R ≥ dhead, b = (dhead − dh)/2 more than 2/3 w (and not counted beyond 3/4 w).
- **Bearing** uses the smaller yield stress of the plate and the pin. For an eyebar it is checked only when the pin is weaker, as the note to Example D.8 says.
- **Not checked.** The pin itself (shear and flexure), the clevis or gusset it bears on, and fatigue. Design Examples D.7 and D.8 do not check the pin either. ILX evaluates Eq. D5-2 with Cr = 1.0 only, so a hole more than 1/32 in. (1 mm) larger than the pin fails a geometry check.

The drawings show the plate or eyebar head with the pin, the widths b | dh | b, a or R, the corner cuts and the effective width, and a section through the pin with the plies, the connected parts and the shear planes.

## Verification of the calculators

Every listed Design Example is reproduced end to end, in LRFD and ASD, as an automated test. The results appear on the **Verification** page (see Verification).

One documented deviation remains, R1, in II.A-22. AISC rounds b' and λ before calculating the buckling stress, so ILX's Fcr and Nc come out about 1 % lower. The interaction ratio agrees within 0.5 %.

## Calculator limits

- The column under a bracket plate is not checked (flange bending, web local yielding), as in the Design Examples.
- Prying in tee and flange joints is not included: the bolt tension is T / n.
- Auto-design, batch design and Compare are not available for the calculators yet.
- Load cases (the **Load cases** load input) are entered through the API. The editor uses the required values when no cases are given.

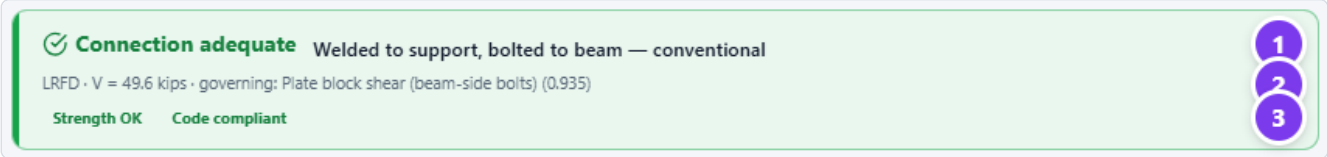
CHAPTER 29

# Checks and compliance

## Reading the results

The results column reads from top to bottom: is the connection adequate, why, and what to change.

### The status bar




1. **Connection adequate** (green), **Connection not adequate** (red) or **Connection not feasible** (rose, see NOT FEASIBLE), followed by the configuration, for example *Welded to support, bolted to beam — conventional*.
2. The design method, the required shear V (and axial P), and the **governing** check with its ratio. With several load combinations, the bar adds **envelope of combinations**.
3. The tallies:
   - **Strength OK**, or the number of strength failures;
   - **Code compliant**, or the number of code-compliance violations;
   - the number of advisories, when there are any.

   Select the compliance or advisory pill to open the **Code compliance** list. When the geometry is not feasible, the tallies read **NOT FEASIBLE · n geometry issues** and **Strength not evaluated** instead, and **Strength OK** and **Code compliant** are not shown.

A connection is **adequate** only when every strength check and every detailing (code-compliance) check passes. Recommendations never make a connection inadequate.

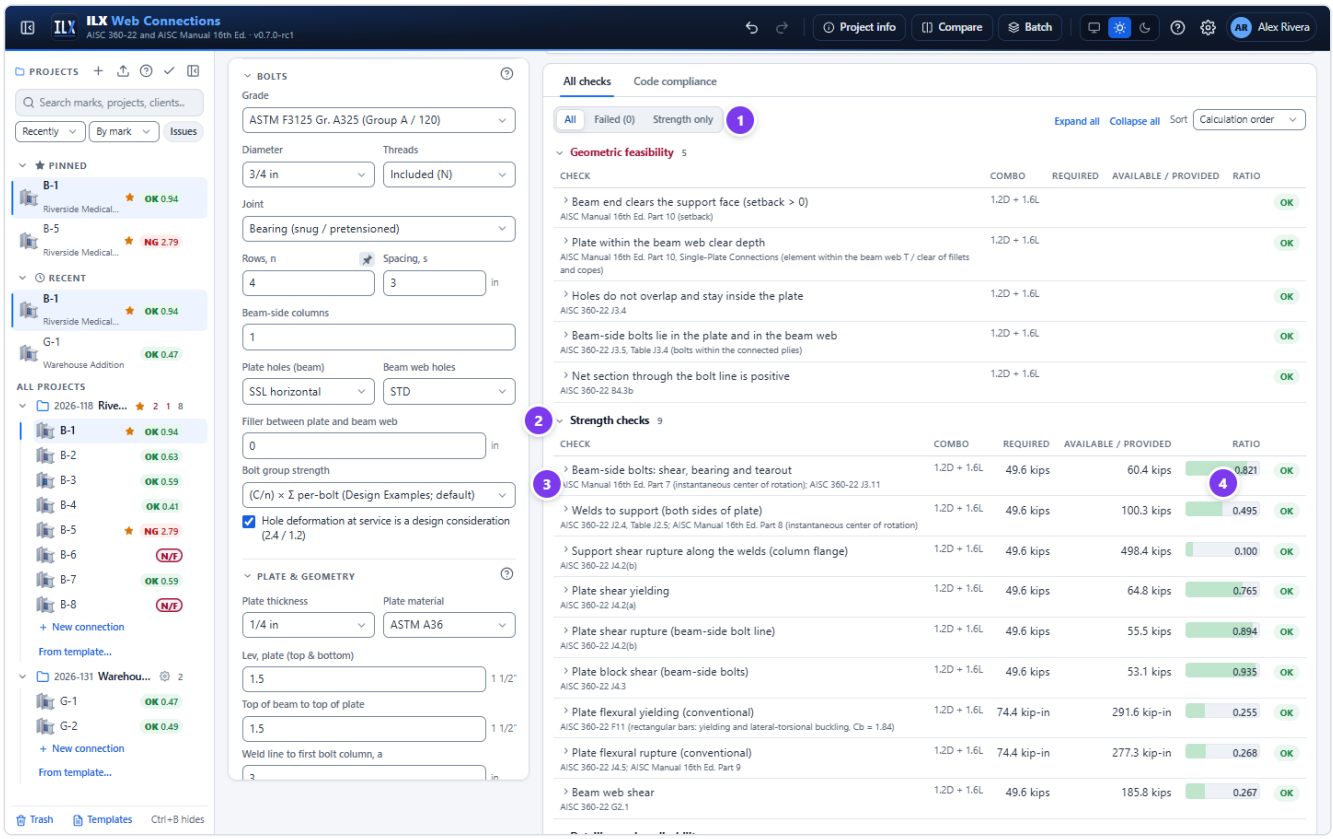
Below the status bar:

- **Required strength**: open the card to see how V and P were obtained from the loads, typeset step by step.
- **Key geometry**: the plate or angle size, the eccentricity used for the bolt group, the bolt coefficient C and the setback.

### The governing check

The **governing** check is the strength check with the highest ratio. Its ratio is the one shown in the status bar, the file tree and the report. A connection can be NG with a governing ratio below 1.0: then a detailing rule fails, not a strength check. The compliance tally tells you which.

# All checks




1. **Filter**:
   - **All**;
   - **Failed (n)**: only the failing checks;
   - **Strength only**.

   On the right, **Expand all** and **Collapse all** open or close every row, and **Sort** orders by **Calculation order** or **Ratio (highest first)**.
2. **Groups**. Select a group title to collapse it.
   - **Geometric feasibility**: can it be built at all?
   - **Strength checks**: limit states with a demand, a capacity and a ratio.
   - **Fatigue (Appendix 3)**: with **Cyclic loading** ticked, the stress range at each detail against its Table A-3.1 allowable range FSR; see Fatigue (Appendix 3). The report lists them in a table of their own with the stress category.
   - **Detailing and applicability**: spacing, edge distances, hole permissions, Table 10-9 limits, weld size rules and similar.
   - **Recommendations**: good practice that does not affect adequacy (marked **Advisory**).
3. **Clause reference**. Under each title is the governing clause or Manual reference, for example *AISC 360-22 J4.2(b)* or *AISC Manual 16th Ed. Part 7*.
4. **Ratio bar**: required ÷ available. A green bar is OK; red is NG.

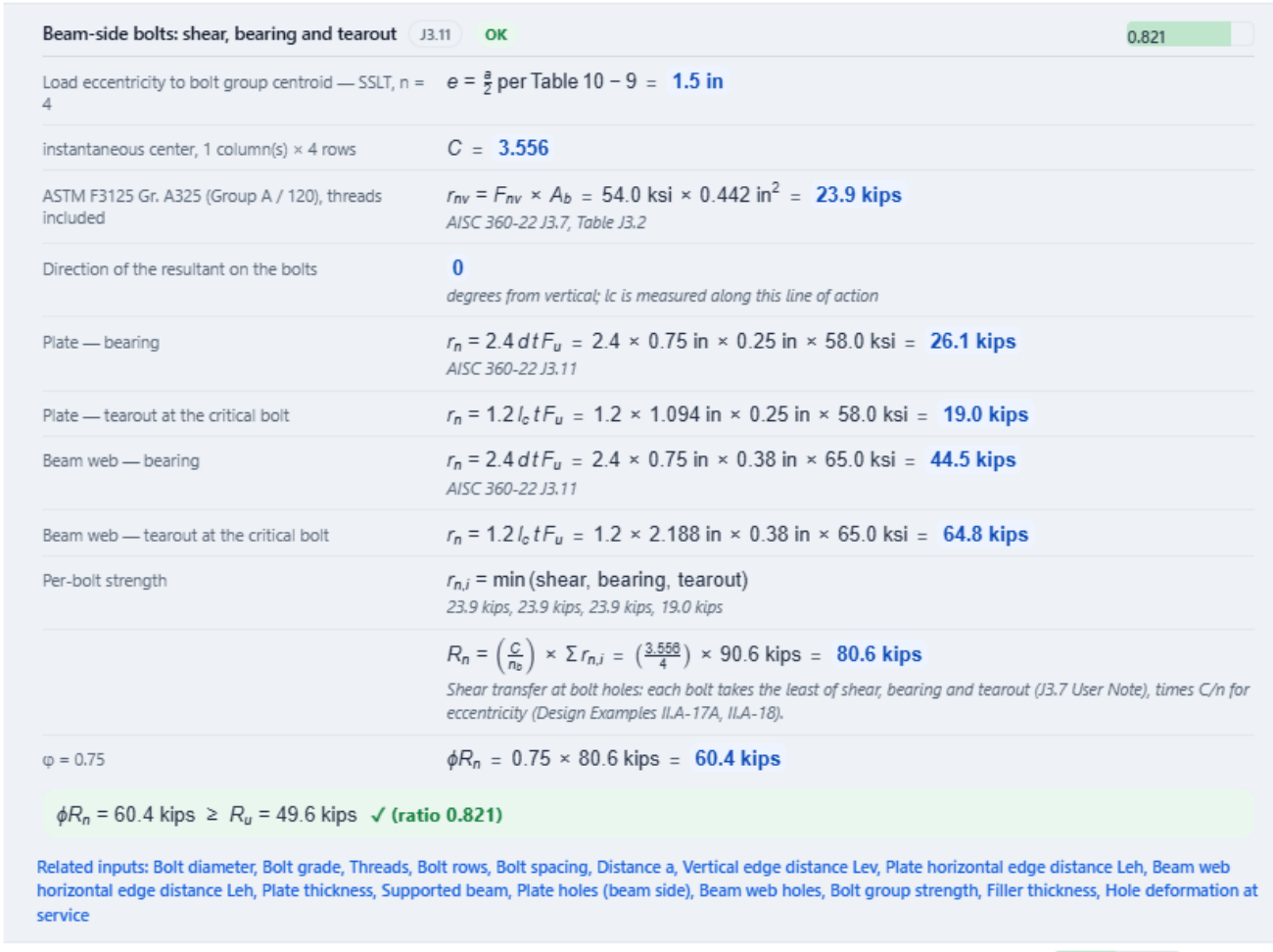
The **Combo** column gives the load combination that governs each check.

## Strength, geometry and recommendation

| Category | Pass / fail means | Affects adequacy |
|---|---|---|
| **Strength** | Demand ≤ available strength (φRn or Rn/Ω) | Yes |
| **Geometry** (detailing and applicability) | A code or Manual rule is met: spacing, edge distance, hole type, tmax, weld size, procedure limits | Yes |
| **Recommendation** | A practice rule is met; shown as **Advisory** when not, for example Fy Z ≤ Fu Znet for an extended plate | No |
| **Feasibility** | The geometry can physically be built | Yes: see NOT FEASIBLE |

## Calculation cards, clause references and fix hints

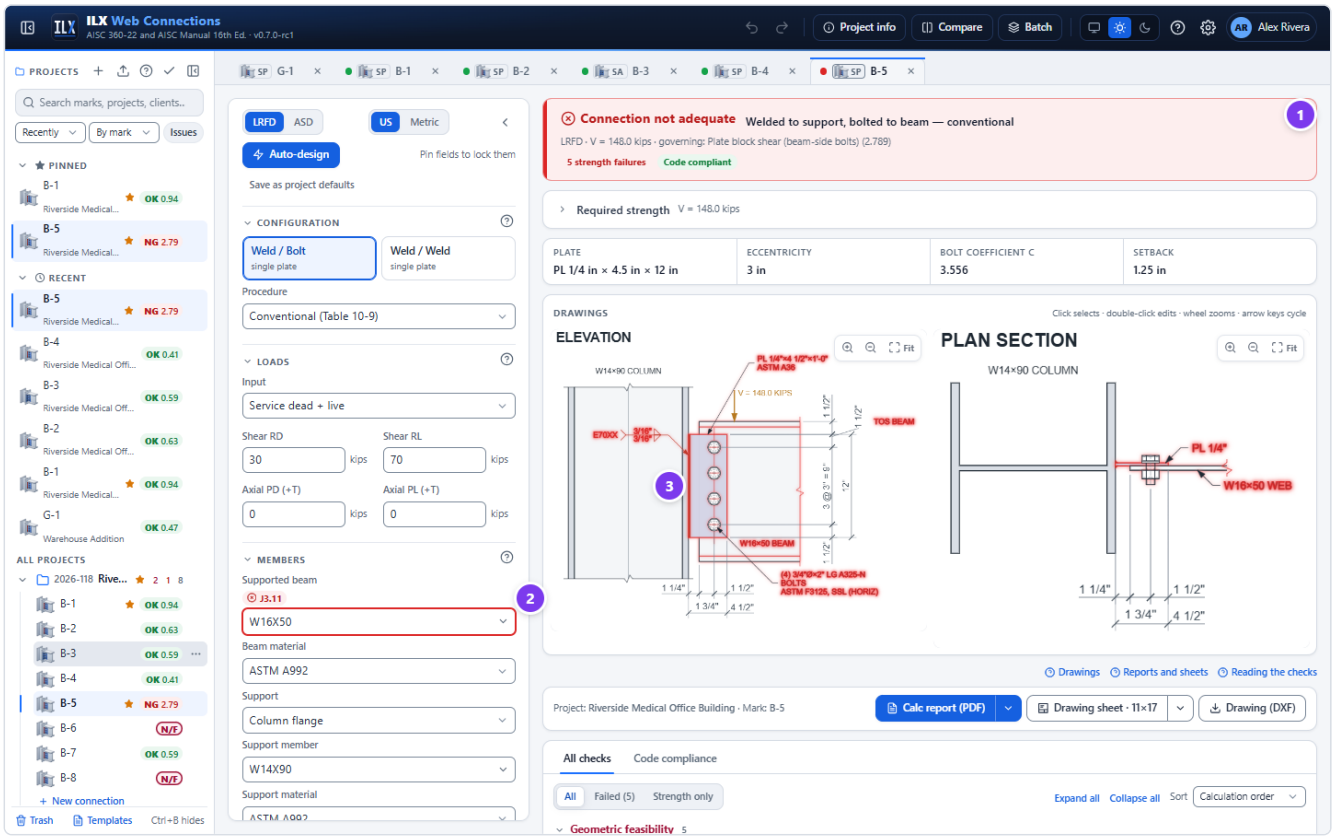
Select a check row to open its calculation card.




- **The working.** Each step gives the equation, the substituted values and the result, with the clause or table that supplies it.
- **The comparison.** The card ends with the code comparison, for example φRn = 60.4 kips ≥ Ru = 49.6 kips ✓, and the ratio.
- **Fix hint.** A failing check adds **How to fix:** with the change that resolves it, for example *Add bolts, increase edge distances, thicken the thinner ply or use a larger bolt.*

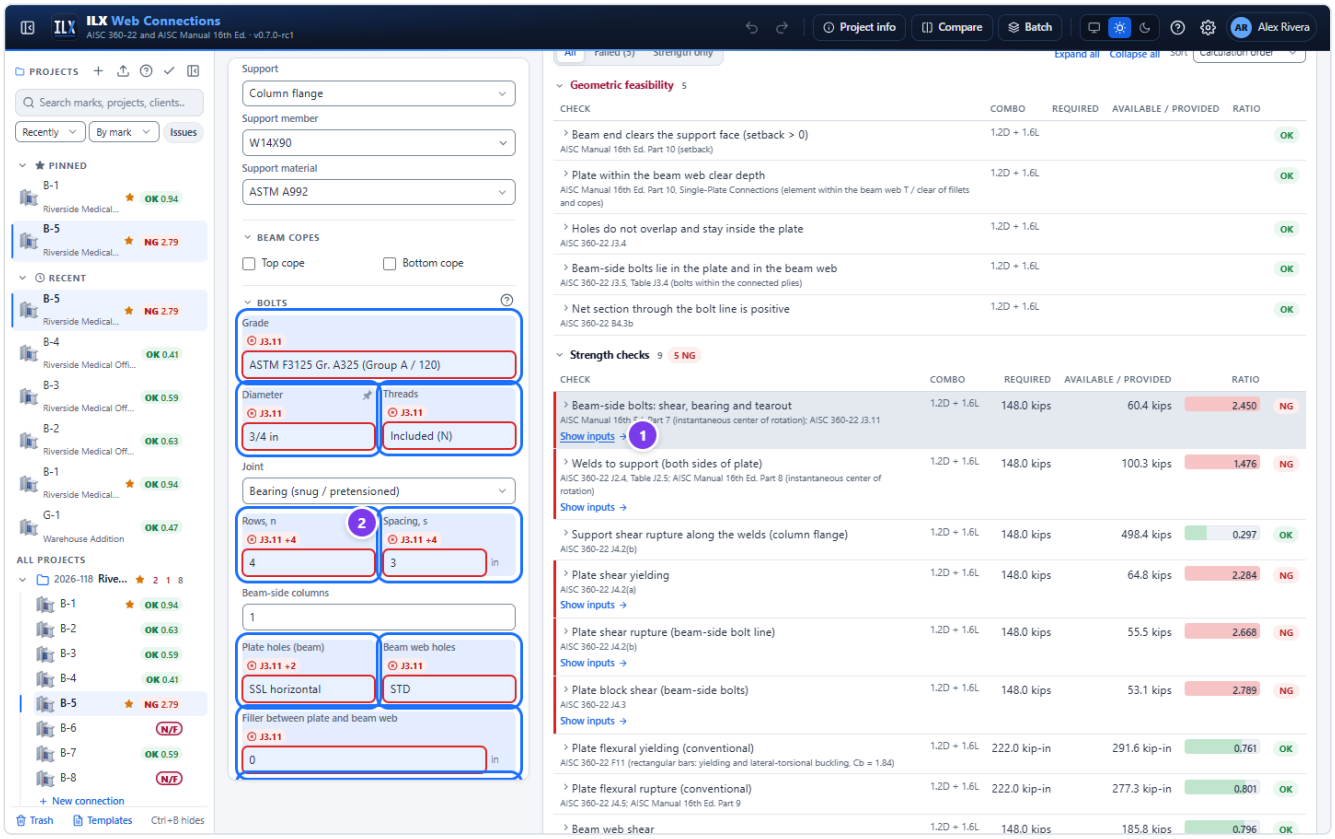
- **Related inputs.** Lists, by their labels in the input panel, the fields the check depends on. Select it to jump to them.

## Failing checks: from the result to the input



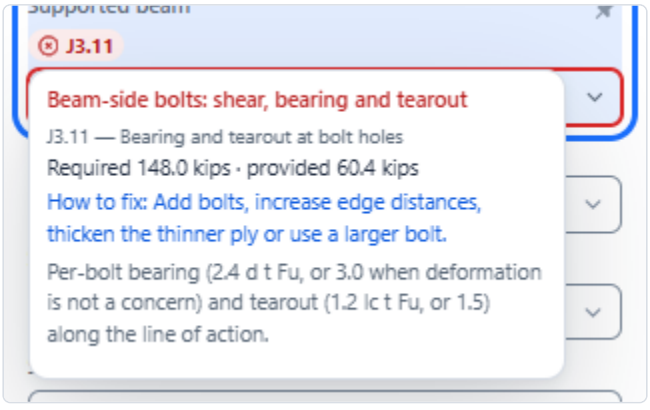

1. The status bar turns red and counts the failures.
2. Every input that a failing check depends on carries an **issue badge** with the clause, for example **J3.11 +4** (this clause and four more issues).
3. In the drawing, the elements of failing checks are outlined in red.

To go from a failing check to the inputs that cause it:



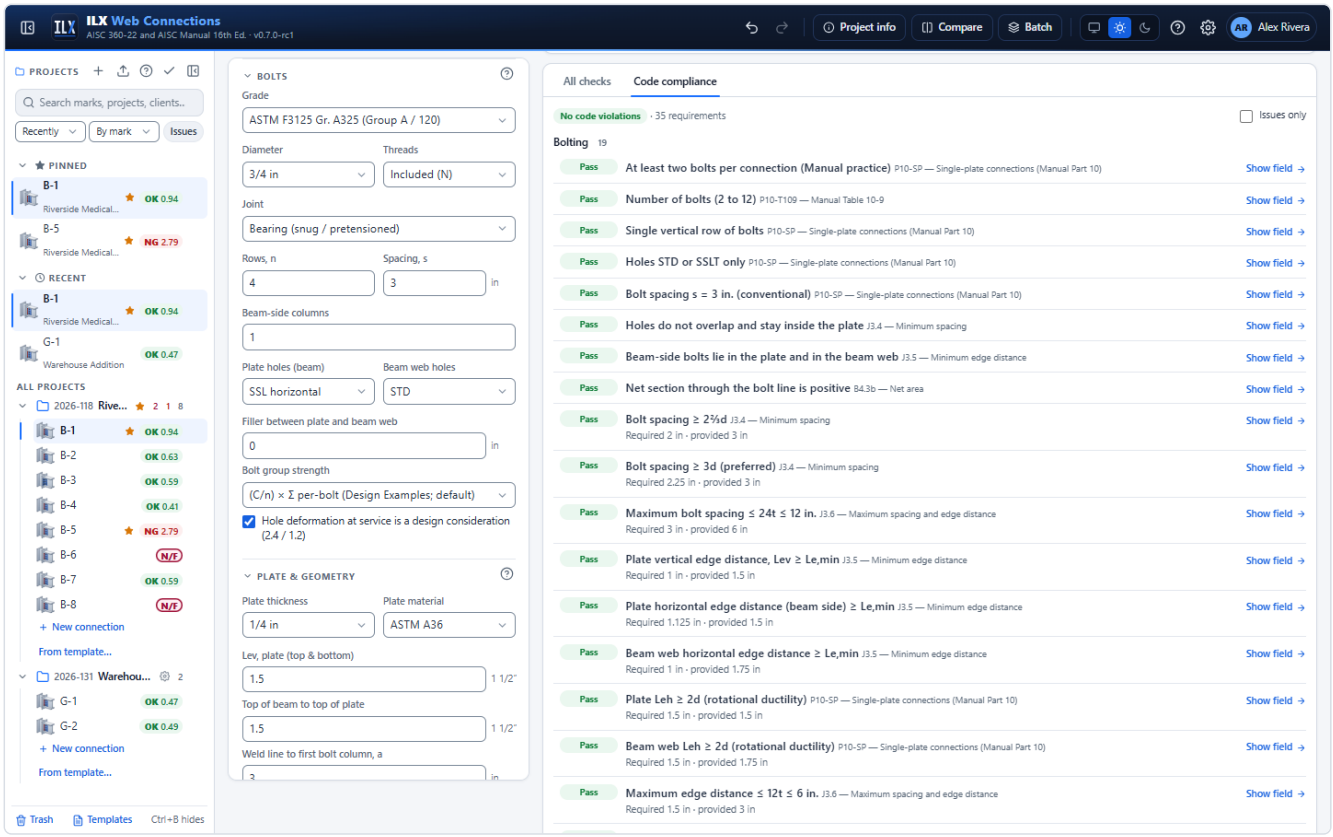

1. Select **Show inputs** under the failing check.
2. The input panel scrolls to the related fields and highlights them for a few seconds with a blue outline set outside the field and a blue tint, so the highlight also shows on fields already outlined in red. The element is also selected in the drawing.

Hover over an issue badge, or select it, to read the check without leaving the input. The popup gives the check, the clause and its title, the required and provided values, the fix hint and a one-line summary of the clause.




## Code compliance

**Code compliance** lists every detailing and applicability requirement, with recommendations and feasibility, grouped by topic: **Bolting**, **Welding**, **Copes**, **Plate / angle** and **Geometry fit**.




- **Status.** Each row is **Pass**, **Fail** or **Advisory**, with the clause, its title, and the required and provided values.
- **Failing rows.** These add the message and the fix hint.
- **Show field.** Jumps to the input.
- **Issues only.** Tick it to hide the passing requirements.
- **Tab badge.** The **Code compliance** tab shows the number of violations and advisories.

## Bolt rules for every type

Every connection type with bolts applies the same bolt rules, from AISC 360-22 J3 and the RCSC Specification (2020 by default, or 2025 as a design rule). The check, the report note, the bolt callout and the drawn bolt all come from one source, so they always agree.

- **Pretension required** (`pretension_req`). A snug-tight joint fails in three cases: Group 150 (A490, F2280) or Group 200 (F3043) bolts carry tension or combined shear and tension, bolts in tension see cyclic load, or a joint is subject to fatigue load. AISC 360-22 J3.2(a)(2) allows snug-tight bolts in tension for Group 120 only (RCSC 4.2). The fix is **Pretensioned** or **Slip-critical**. Every type whose bolts can carry tension offers the **Pretensioned** joint.
- **Project rule** (`pretension_project`). When the project's design rules require pretension, every bolted joint that is not pretensioned or slip-critical fails (RCSC 4.2(1)).
- **Joint type on the drawing.** The general notes make snug-tight the default. Pretensioned bolts are called out **-PT**, for example `A490-N-PT BOLTS`, and slip-critical bolts **SC CLASS A** or **SC CLASS B**.
- **Washers** (RCSC Section 6). These cover slotted outer plies of snug-tight joints (an F436 or a 5/16-in. plate washer) and oversized or slotted outer plies of pretensioned and SC joints. For those plies, Group 150 bolts over 1 in. need the 5/16-in. extra-thick F436. Long slots need a 5/16-in. plate washer or bar, or for Group 150 bolts over 1 in. a 3/8-in. plate washer or bar with an F436. Pretensioned Group 150 bolts in steel with Fy < 40 ksi get

F436 washers under the head and the nut. Sloped faces (S, C and MC flanges) get beveled washers. Group 200 assemblies take the washers of their ASTM standard. The drawing shows a head washer wherever the list needs one.

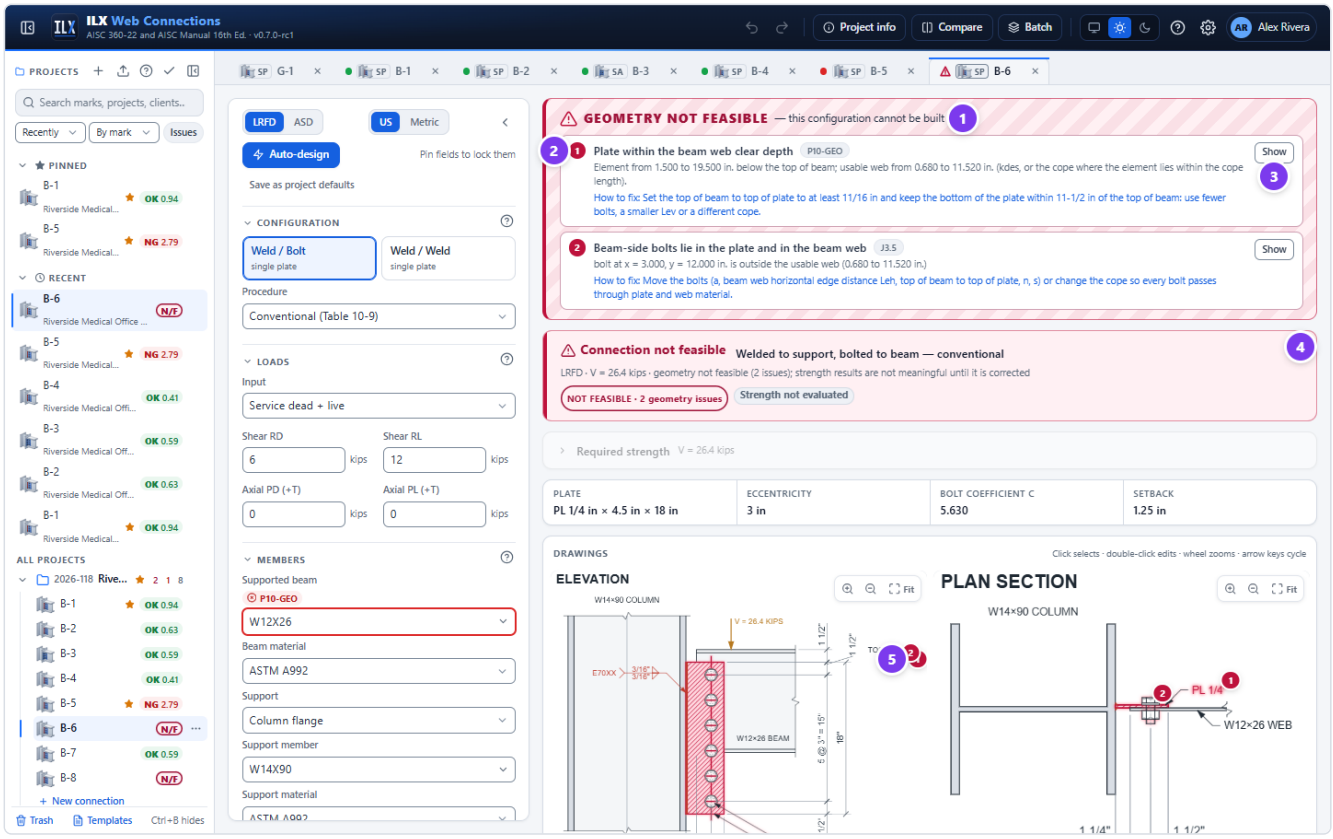
- **Group 200 sizes** (`group_c_size`). F3043 / F3111 bolts come only in 1, 1-1/8 and 1-1/4 in. (Table J3.1). The diameter lists offer only those sizes, and auto-design never proposes others.
- **Galvanized** (`bolt_galvanized`). Galvanized Group 150 and Group 200 bolts fail (RCSC 2.8.1; RCSC covers Groups 120, 144 and 150, and Group 200 follows its ASTM standard). Galvanized Group 120 bolts get the assembly note: a matched, lubricated and rotation-capacity-tested assembly with overtapped DH nuts, never reused.
- **Threads excluded** (`threads_x…`). With **Excluded (X)**, ILX takes the bolt length the drawing uses, the thread length (ASME B18.2.6) plus the run-out, and the ply stack. The check fails when the threads reach a shear plane. The hint offers threads included (N, 26 % less shear strength), a reversed bolt, or a longer bolt with extra washers.
- **Fillers** (J5.2). Fillers thicker than 1/4 in. reduce bolt shear by $1 - 0.4(t - 0.25)$, not below 0.85, where t is the total filler thickness. Slip-critical joints get the reduction in their bearing-type check too (J3.9), unless the fillers are developed.
- **Long bolts.** For a pretensioned bolt longer than 12d, a note says the turn-of-nut rotation must be found by test (RCSC Table 8.1).

Clause labels such as **P10-SP** (single-plate practice, Manual Part 10) or **P10-T109** (Table 10-9) refer to the Manual. Labels such as **J3.4** or **B4.3b** refer to AISC 360-22. The glossary lists the short labels.

## NOT FEASIBLE

ILX tests the geometry before it trusts any strength result. If any part cannot physically be built, the connection is **NOT FEASIBLE**, whatever its strength. Examples:

- a bolt outside a ply, or beyond the beam end;
- a plate deeper than the beam web's clear depth;
- a plate beyond the support face;
- the beam clashing with the support.

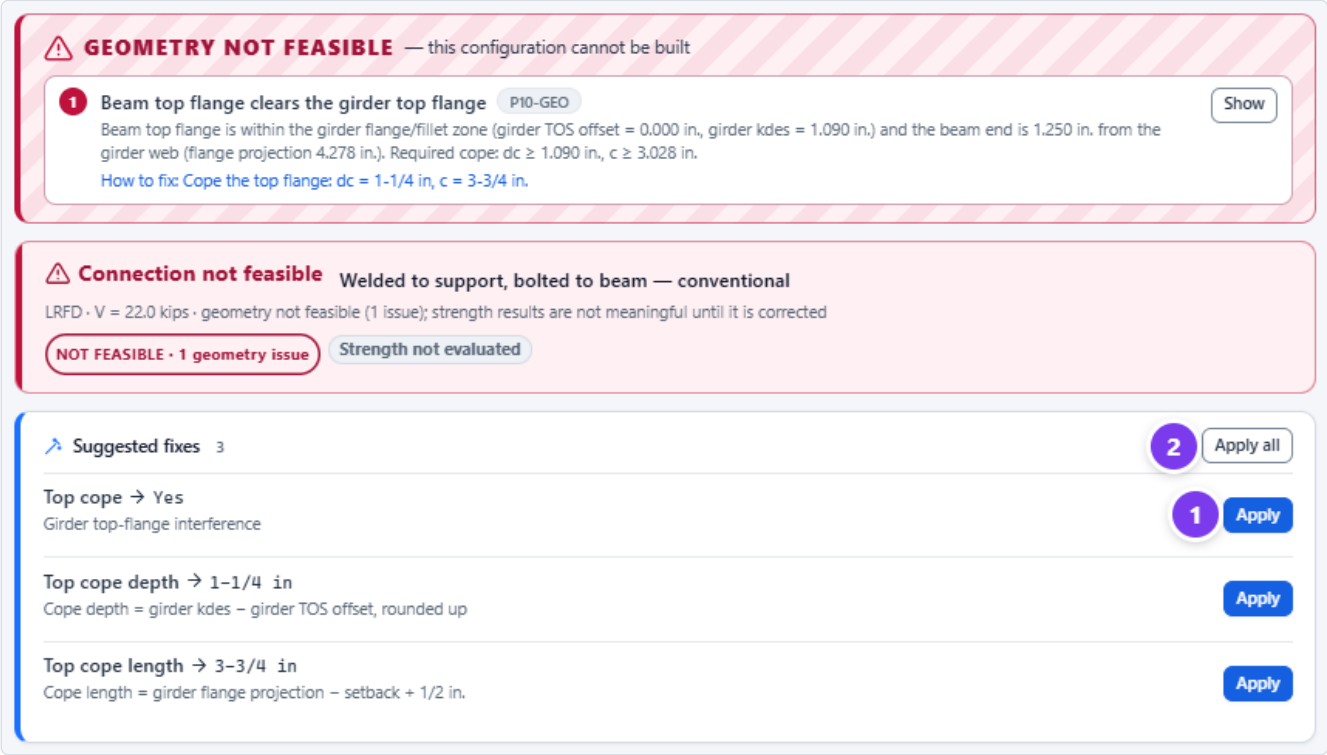



1. **The banner.** A red banner, **GEOMETRY NOT FEASIBLE — this configuration cannot be built**, replaces the usual first line.
2. **Numbered items.** Each problem is numbered and gives the title, a clause label, the measured values and **How to fix:**.
3. **Show.** Selects the element in the drawing and jumps to its inputs.
4. **Status bar.** It reads **Connection not feasible**, with the number of geometry issues and *strength results are not meaningful until it is corrected*. The pills show **NOT FEASIBLE** and **Strength not evaluated**; the strength results below are greyed out.
5. **Markers.** The same numbers mark the offending elements on the drawings, which are hatched in red.

In the project panel and the tabs the connection shows **N/F** and a warning triangle. Auto-design, batch and Compare show **NOT FEASIBLE**, never NG or *Not adequate*. The same status rule is used everywhere, including the report.

## Suggested fixes

For some problems ILX can compute the fix itself. Examples are the cope a beam needs to clear a girder flange, or the weld size the 5/8 tp rule requires. These appear as **Suggested fixes** under the status bar.




1. Select **Apply** on one suggestion. The value is entered, and the field is highlighted in the input panel.
2. Select **Apply all** to apply every suggestion at once.

Each suggestion names the input by its label and shows the value with its unit, for example **Top cope → Yes** and **Top cope depth → 1-1/4 in**. It gives its reason, for example *Cope depth = girder kdes − girder TOS offset, rounded up*. **Undo** (Ctrl+Z) reverses an applied suggestion.

## Load combinations in the results

With **Service components → ASCE 7 combinations** or a **Factored load cases** table, ILX designs the connection for each combination and reports the envelope.

Load combinations 12

| COMBINATION | V (KIPS) | P (KIPS) | STRENGTH RATIO | DETAILING | STATUS |
|---|---|---|---|---|---|
| 1.4D | 12.6 kips | 0.0 kips | 0.190 | ✓ | OK |
| 1.2D + 1.6L | 36.4 kips | 0.0 kips | 0.550 | ✓ | OK |
| 1.2D + 1.6L + 0.5S GOVERNS | 39.4 kips | 0.0 kips | 0.595 | ✓ | OK |
| 1.2D + 1.6S + L | 36.4 kips | 0.0 kips | 0.550 | ✓ | OK |
| 1.2D + 1.6S + 0.5W | 16.4 kips | 6.0 kips | 0.261 | ✓ | OK |
| 1.2D + 1.6S − 0.5W | 24.4 kips | -6.0 kips | 0.378 | ✓ | OK |
| 1.2D + 1.0W + L | 18.8 kips | 12.0 kips | 0.325 | ✓ | OK |
| 1.2D − 1.0W + L | 34.8 kips | -12.0 kips | 0.552 | ✓ | OK |
| 1.2D + 1.0W + L + 0.5S | 21.8 kips | 12.0 kips | 0.367 | ✓ | OK |
| 1.2D − 1.0W + L + 0.5S | 37.8 kips | -12.0 kips | 0.595 | ✓ | OK |
| 0.9D + 1.0W | 0.1 kips | 12.0 kips | 0.173 | ✓ | OK |
| 0.9D − 1.0W | 16.1 kips | -12.0 kips | 0.292 | ✓ | OK |

- **The table.** The **Load combinations** table lists each combination with its V, P and **Strength ratio**, the governing strength ratio of that combination. The combination with the highest ratio is tagged **governs**.
- **The Detailing column.** A check mark when every detailing (geometry) check passes in that combination, or a cross with the number of failing detailing checks. Hover over it for their names. Detailing checks can depend on the load, for example the slip-critical requirement once an axial load is present.
- **The Status column. OK**, **NG** when a strength check fails, or **NG (detailing)** when the strength is adequate but a detailing check fails. A row reading NG (detailing) can therefore have a strength ratio below 1.0.
- **The Combo column.** In **All checks**, it gives the combination that governs each check, so different checks can be governed by different combinations.

**Strength checks** 22

| CHECK | COMBO | REQUIRED | AVAILABLE / PROVIDED | RATIO | |
|---|---|---|---|---|---|
| Beam-side bolts: shear, bearing and tearout<br>AISC Manual 16th Ed. Part 7 (instantaneous center of rotation); AISC 360-22 J3.11 | 1.2D − 1.0W + L + 0.5S | 39.7 kips | 66.6 kips | 0.595 | OK |
| Welds to support (both sides of plate)<br>AISC 360-22 J2.4, Table J2.5; AISC Manual 16th Ed. Part 8 (instantaneous center of rotation) | 1.2D − 1.0W + L + 0.5S | 39.7 kips | 116.3 kips | 0.341 | OK |
| Support shear rupture along the welds (column flange)<br>AISC 360-22 J4.2(b) | 1.2D − 1.0W + L + 0.5S | 39.7 kips | 623.0 kips | 0.064 | OK |
| Column web local yielding (axial force through the flange)<br>AISC 360-22 J10.2 | 1.2D + 1.0W + L | 12.0 kips | 474.1 kips | 0.025 | OK |
| Column web crippling (axial compression)<br>AISC 360-22 J10.3 | 1.2D − 1.0W + L | 12.0 kips | 456.3 kips | 0.026 | OK |
| Plate shear yielding<br>AISC 360-22 J4.2(a) | 1.2D + 1.6L + 0.5S | 39.4 kips | 81.0 kips | 0.486 | OK |
| Plate shear rupture (beam-side bolt line)<br>AISC 360-22 J4.2(b) | 1.2D + 1.6L + 0.5S | 39.4 kips | 69.3 kips | 0.568 | OK |
| Plate block shear (beam-side bolts)<br>AISC 360-22 J4.3 | 1.2D + 1.6L + 0.5S | 39.4 kips | 66.2 kips | 0.595 | OK |
| Plate compression<br>AISC 360-22 J4.4 | 1.2D − 1.0W + L | 12.0 kips | 106.6 kips | 0.113 | OK |
| Plate flexural yielding (conventional) | 1.2D − 1.0W + L + 0.5S | 113.4 kip-in | 455.6 kip-in | 0.249 | OK |

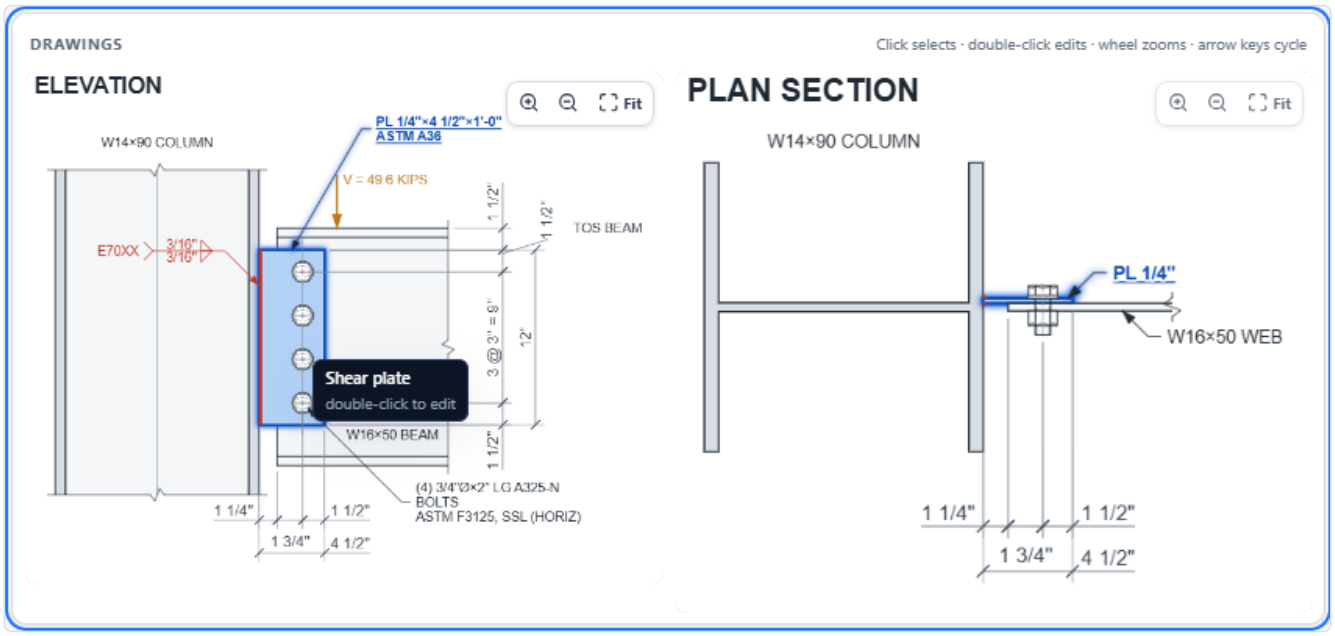
CHAPTER 30

# Drawings

## On-screen drawings

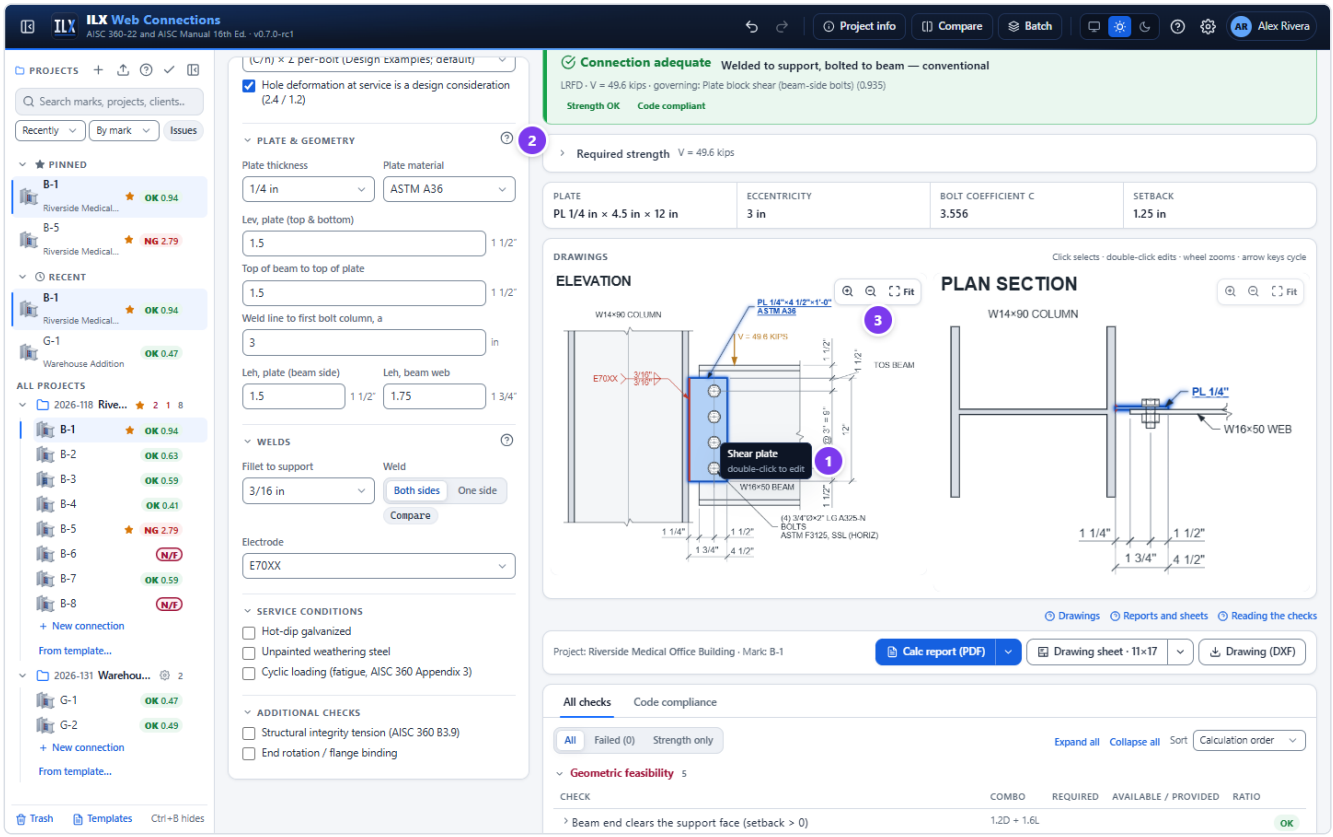
Every connection is drawn live, to scale, as an **elevation** and a **plan section**. Some types draw other views: the shear end plate adds an **end plate detail** and the double angle a **section A-A**; the seated connection shows a **section A-A** in place of the plan; the FR moment plan is a **plan section at the top flange**; and the base plate shows an **elevation (section through the pier)** and a **plan**. The drawings show:

- the real support profile: a column flange with the I-section in plan, a column web with the flanges hidden, a girder cross-section offset by the top-of-steel difference, or the HSS walls;
- the plate or angle, with an angle's support leg edge-on (each connection type draws its own elements: angles, end plate, seat, tee, flange plates, base plate and anchor rods);
- bolts with heads, washers, nuts and stick-out;
- radiused copes;
- dimensions, callouts and AWS weld symbols;
- the load arrow.



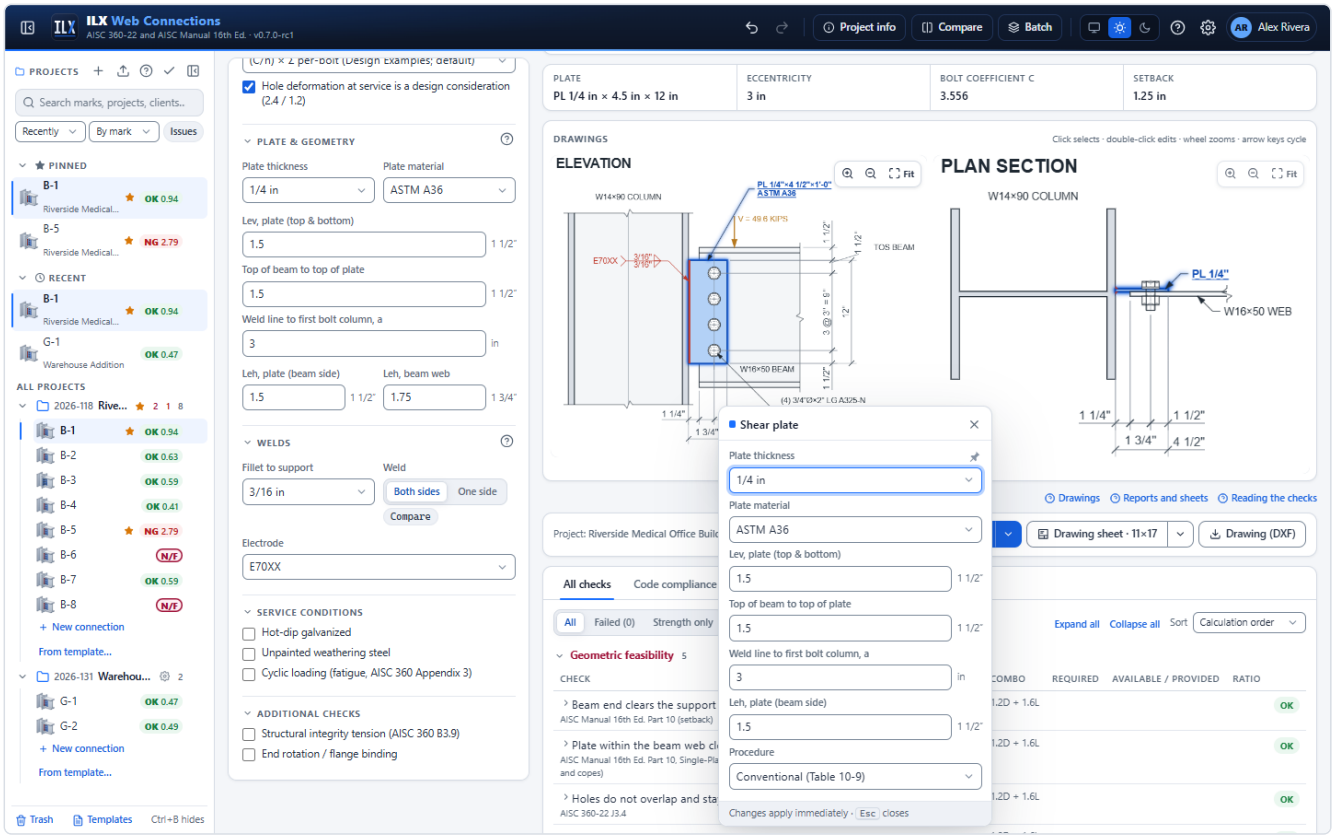

The on-screen drawing, the DXF and the PDF sheet are generated from the same drawing description, so they always agree.

## Selecting and editing elements




1. **Hover and select.** Hovering over an element shows its name and what a click does. Click an element (plate, angle, bolts, welds, beam, support, cope, load) to select it. It is outlined in blue in both views.
2. **The matching inputs.** The input panel scrolls to the element's section.
3. **Zoom controls.** Each view has its own (see Zoom and pan).

**Double-click an element to edit it.** Its fields open in a small editor beside it. Changes apply immediately; press **Esc** or select **×** to close.




The element editor shows only the fields that apply to the element in the current configuration. For example, the plate in a Weld / Bolt connection shows the thickness, material, Lev, top of plate, a, Leh and procedure.

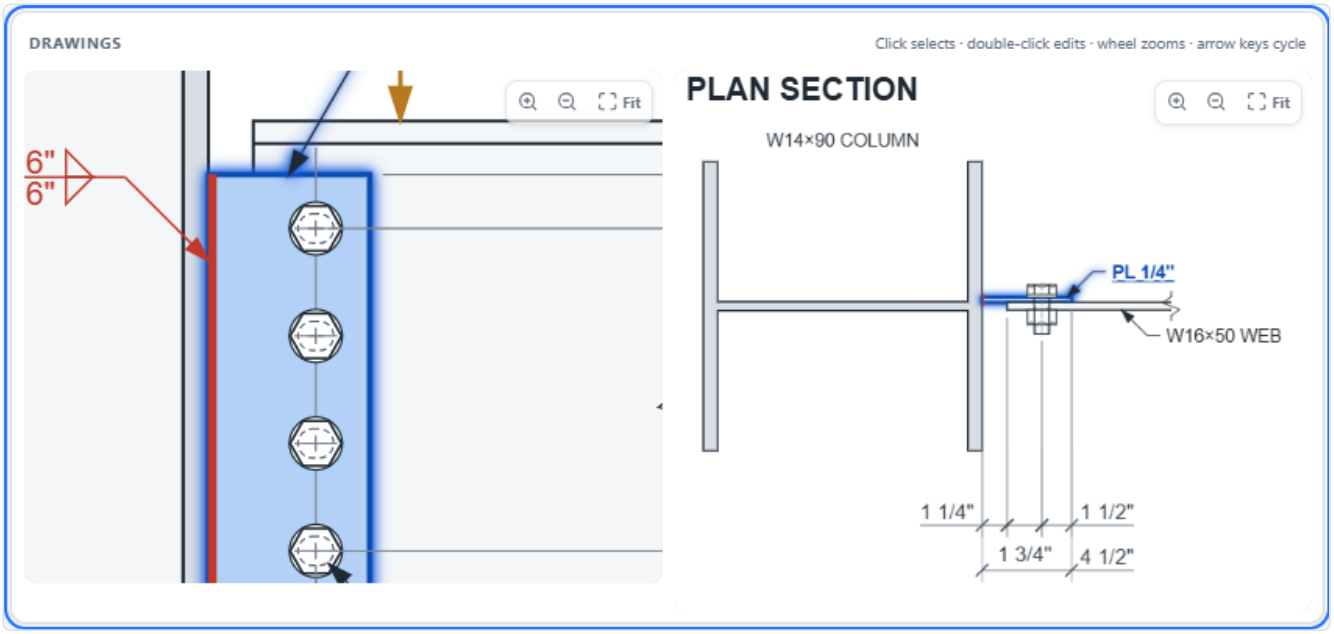
**From the keyboard:** select the drawings panel, then use the arrow keys (or Tab) to step through the elements. Press **Enter** to edit the selected element and **Esc** to clear the selection.

**Failing elements** are outlined in red. **Not feasible** elements are hatched red and carry the numbered markers of the NOT FEASIBLE banner.

## Zoom and pan

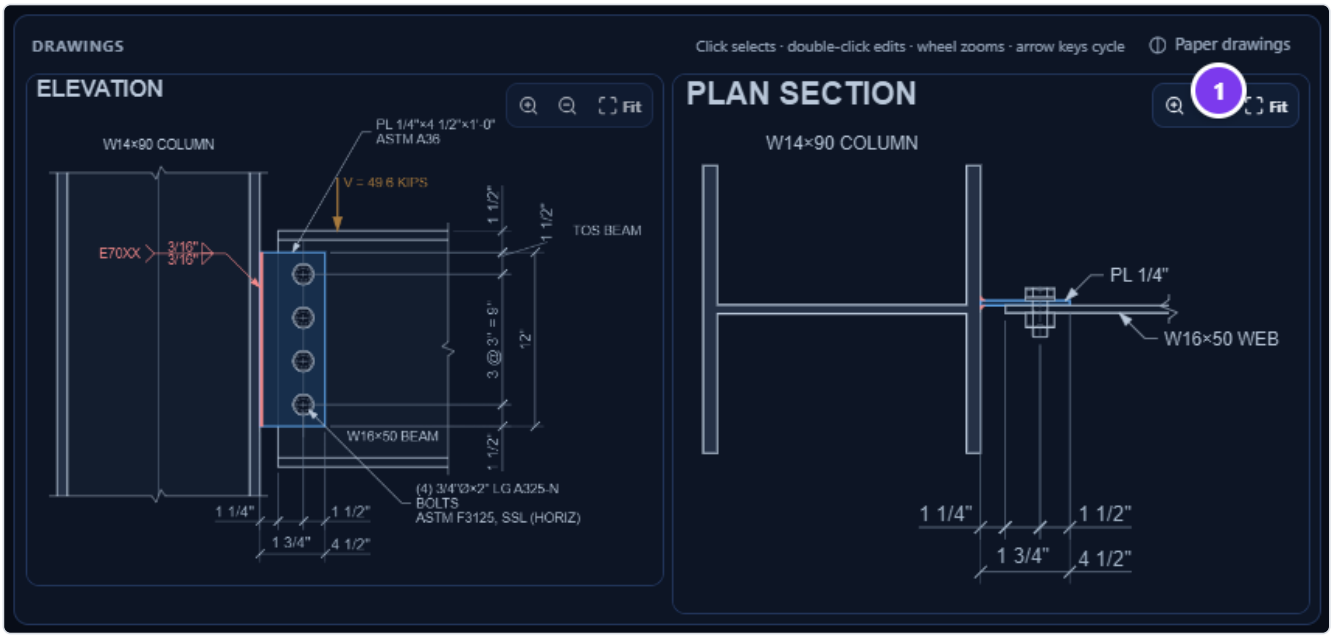
| To… | Do this |
|---|---|
| Zoom about the cursor | Turn the mouse wheel over a view |
| Zoom in steps | Use **Zoom in** and **Zoom out** in the view's corner |
| Pan | Drag |
| Fit | Select **Fit**, or double-click empty space |

Each view zooms on its own, from half size to eight times.




## Dark drawings

In the dark theme the drawings stay on white paper by default. To draw them in light ink on navy instead, select **Dark drawings** in the drawings bar. The red, blue and amber highlights keep their meaning. Select **Paper drawings** to go back. The button appears only in the dark theme.

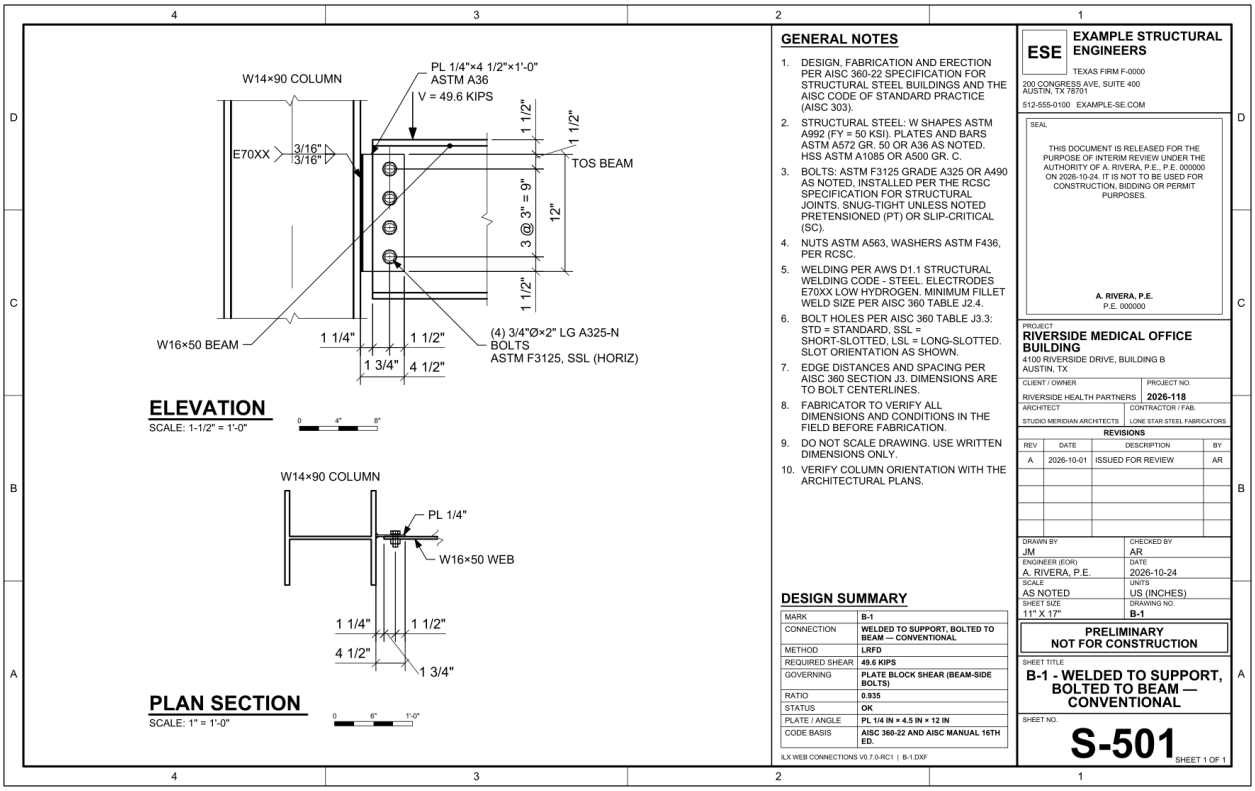



1. **Paper drawings** / **Dark drawings** toggle.

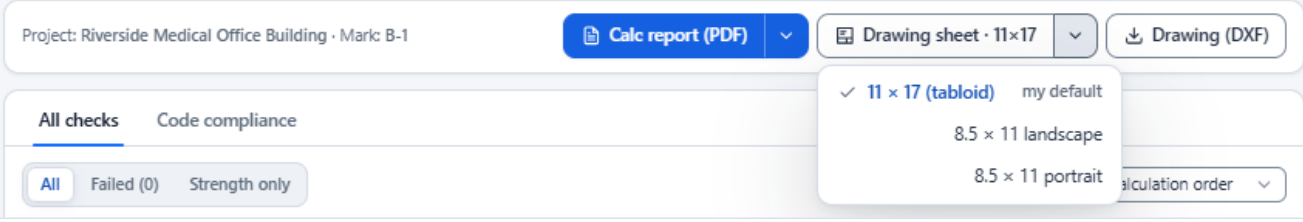
# Drawing sheets

A drawing sheet is the connection detail on a title-blocked sheet, ready to issue or to insert in a drawing set. It contains:

- the elevation and plan views at standard scales, each with its title, scale and bar scale;
- general notes;
- a design summary table: mark, connection, method, required shear, governing check, ratio, status, plate or angle, code basis;
- the title block, with your firm, the seal block, project information, revisions, people, dates, status, sheet title and sheet number.




## Producing a sheet




1. **Drawing sheet** in the output bar produces the PDF at the current size (shown on the button, for example **11×17**).
2. To use another size, open the arrow beside it and pick:
   - **11 × 17 (tabloid)** landscape;

- **8.5 × 11 landscape**;
- **8.5 × 11 portrait**.

The PDF opens at once, and this browser remembers your choice.

3. **Use my default** (or **Use the project default**) returns to the default sheet size of the title block settings. The menu labels that size **project default** when the project overrides the sheet size, and **my default** otherwise (see How settings are inherited).
4. **Drawing (DXF)** produces the same sheet as a DXF file.

In metric connections the sizes are labelled in millimetres and the sheet is drawn in millimetres.

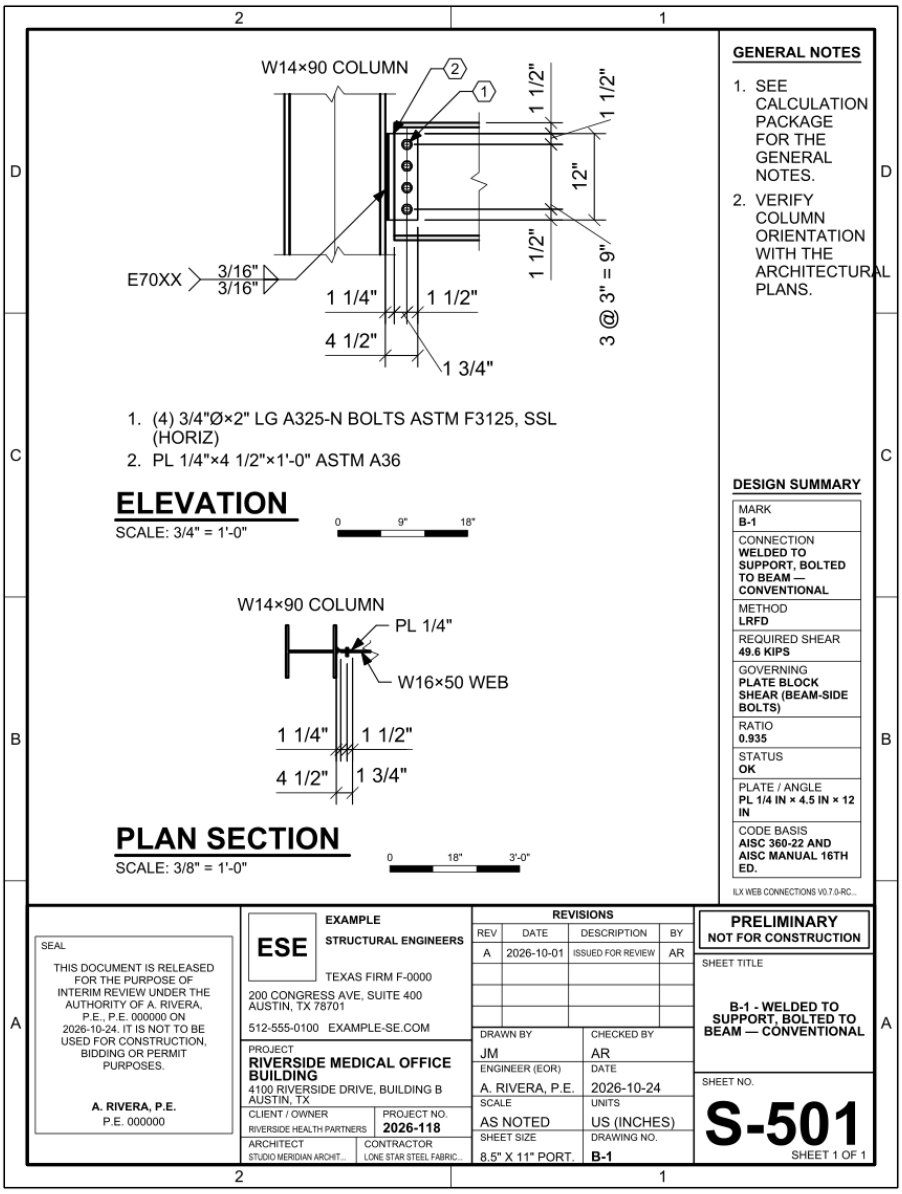
**Scales.** ILX picks the largest standard scale at which each view fits its space, for example 1-1/2" = 1'-0" for the elevation on 11 × 17 and 3/4" = 1'-0" on letter. The scale and a bar scale print under each view title.

### Letter sheets and keynotes

On 8.5 × 11 sheets space is tight. ILX:

- wraps long callouts, and replaces them with numbered **keynotes** (hexagon tags) listed under the view;
- compacts the general notes. When they do not fit, it prints *See calculation package for the general notes.*

On portrait sheets the title block becomes a strip along the bottom.




## What the title block shows

| Title block area | Source |
|---|---|
| Firm name, address, phone, web, license, logo text | Settings → Title block (your defaults or the project's overrides) |
| Seal block | Engineer of record and license, plus the sheet **Status** (for example *Preliminary — not for construction*) |
| Project name, address, client, number, architect, contractor | Project information |
| Revisions | Project information → Revisions (the latest rows) |
| Drawn by, Checked by, Engineer, Date | Project information, falling back to the title block settings |

| Title block area | Source |
|---|---|
| Sheet number | The sheet prefix (for example S-5) and the sheet's place in the set: S-501, S-502… |
| Sheet title | The mark and the connection description |

See Title block settings.

## DXF files

The DXF (AutoCAD R2013 format) is the drawing sheet as an editable CAD drawing:

- **Model space** holds the views at full size, in inches (US) or millimetres (SI).
- **Paper space** holds the sheet, with one viewport per view at its scale, plus the notes, the design summary and the title block.
- **Dimensions** are true DIMENSION entities on the ILX_IN or ILX_MM dimension style, so they update if you stretch the geometry. Text uses the ILX and ILX_BOLD styles.

### Layers

The layers follow the US National CAD Standard (discipline S, Structural).

| Layer | Contents |
|---|---|
| S-DETL-SUPT | Supporting member (column, girder, HSS) |
| S-DETL-MBND | Supported beam |
| S-DETL-PLAT | Shear plate |
| S-DETL-ANGL | Angle |
| S-DETL-BOLT | Bolts (heads, nuts, washers, shanks) |
| S-DETL-HOLE | Bolt holes and slots |
| S-DETL-WELD | Fillet welds |
| S-DETL-HIDN | Hidden lines |
| S-DETL-CNTR | Centre lines |
| S-ANNO-DIMS | Dimensions |
| S-ANNO-TEXT | Callouts and notes |
| S-ANNO-SYMB | Weld symbols |
| S-ANNO-IDEN | View titles |
| S-ANNO-LOAD | Load arrow and value |
| S-ANNO-TTLB | Title block and border |
| S-ANNO-REVS | Revision table and seal box |
| S-ANNO-NPLT | Non-plotting construction geometry |

### Blocks

| Block | Contents |
|---|---|
| `ILX_TITLEBLOCK_<size>` | The title block, with attributes for every field. Edit the attributes (ATTEDIT) to change a value without regenerating the sheet. |
| `ILX_BOLT_<diameter>_<view>_<washers>…` | One block per bolt assembly: size, view, washers and grip. Every bolt is an insert of its block, with the attributes DIA, LENGTH and GRIP. |

## Realistic bolts

Bolts are drawn to their real proportions in every output. Each bolt shows:

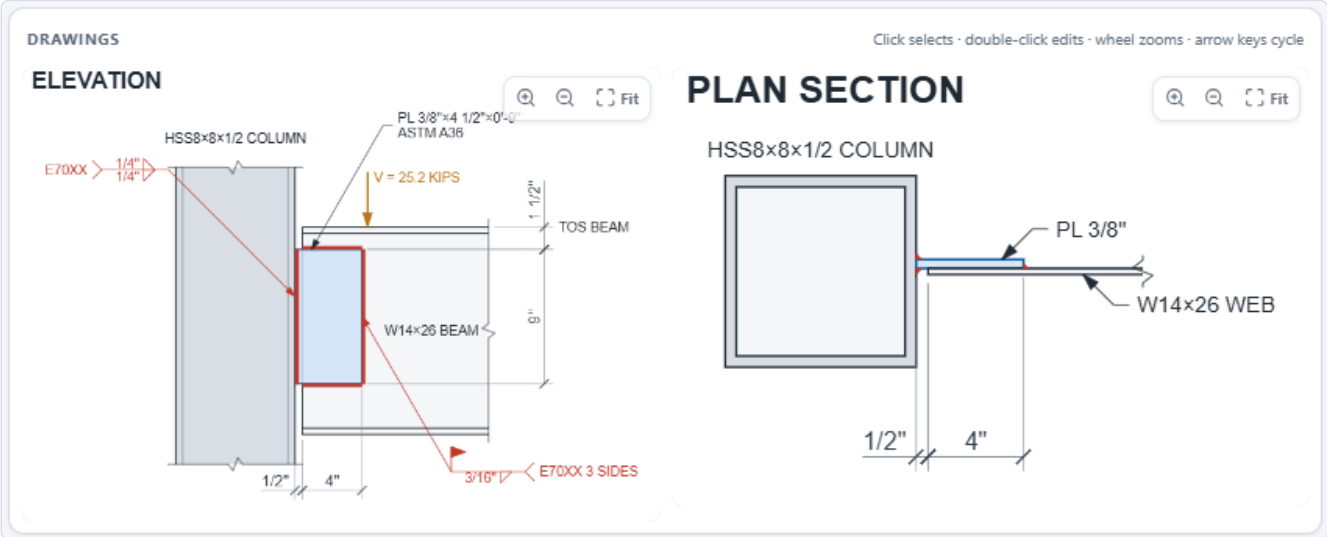
- a heavy hex head and nut, and F436 washers where the joint needs them;
- the threads, and the stick-out beyond the nut;
- the holes, including slots and oversized holes, drawn to size.

The bolt callout gives the quantity, diameter and length, for example **(4) 3/4"Ø×2" LG A325-N BOLTS, ASTM F3125, SSL (HORIZ)**. The bolt length follows AISC Manual 16th Ed. Part 7 (Proper Selection of Bolt Length): the grip, plus the Table 7-14 addition for the nut and stick-out, plus 5/32 in. per F436 washer. It is rounded up to the next 1/4 in. up to a 5 in. length and to the next 1/2 in. over 5 in. Metric lengths are rounded up to 5 mm.

## Weld symbols

Welds are drawn true to size in their corners. Each weld has an AWS weld symbol:

- **Support weld.** A fillet symbol on both sides of the arrow line, with the size on each side (for example 3/16 and 3/16) and the electrode (**E70XX**) in the tail. It means the plate is welded on both faces. A one-sided weld (**Weld**: **One side**) is drawn on the arrow side only.
- **Beam-side weld (single plate Weld / Weld, single angle Bolt / Weld).** The size, the electrode and the extent, for example **E70XX 3 SIDES** for a C-shaped weld.
- **Groove welds** (FR moment beam flanges, base-plate CJP welds): the AWS bevel-groove symbol, with **CJP** in the tail.
- **Colour.** Weld symbols are drawn in red on screen, on the S-ANNO-SYMB layer.

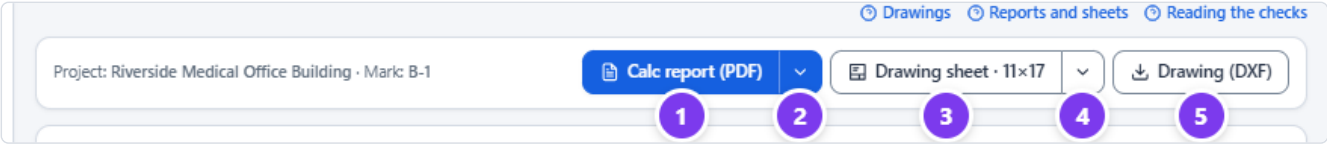


CHAPTER 31

# Reports

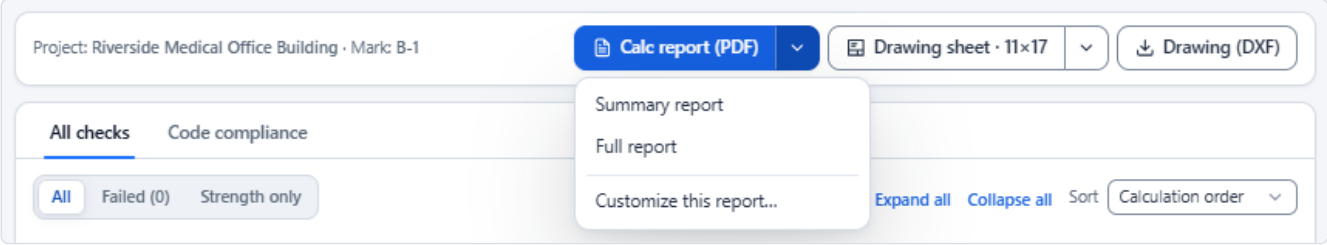
## The calculation report

The calc report is a PDF on US letter portrait pages. It documents one connection, or every connection of a project when you produce a calculation package.




1. **Calc report (PDF)** produces the report with your saved report settings: the project's overrides if it has any, otherwise your defaults.
2. **The arrow beside it** offers:
   - **Summary report**: tables only.
   - **Full report**: every step.
   - **Customize this report…**: change any setting for this one PDF.
3. **Drawing sheet** produces the drawing sheet PDF (see Drawings).
4. **The sheet-size arrow** chooses the sheet size.
5. **Drawing (DXF)** produces the sheet as a DXF file.

The PDF opens in a new browser tab. A report takes a few seconds to produce.

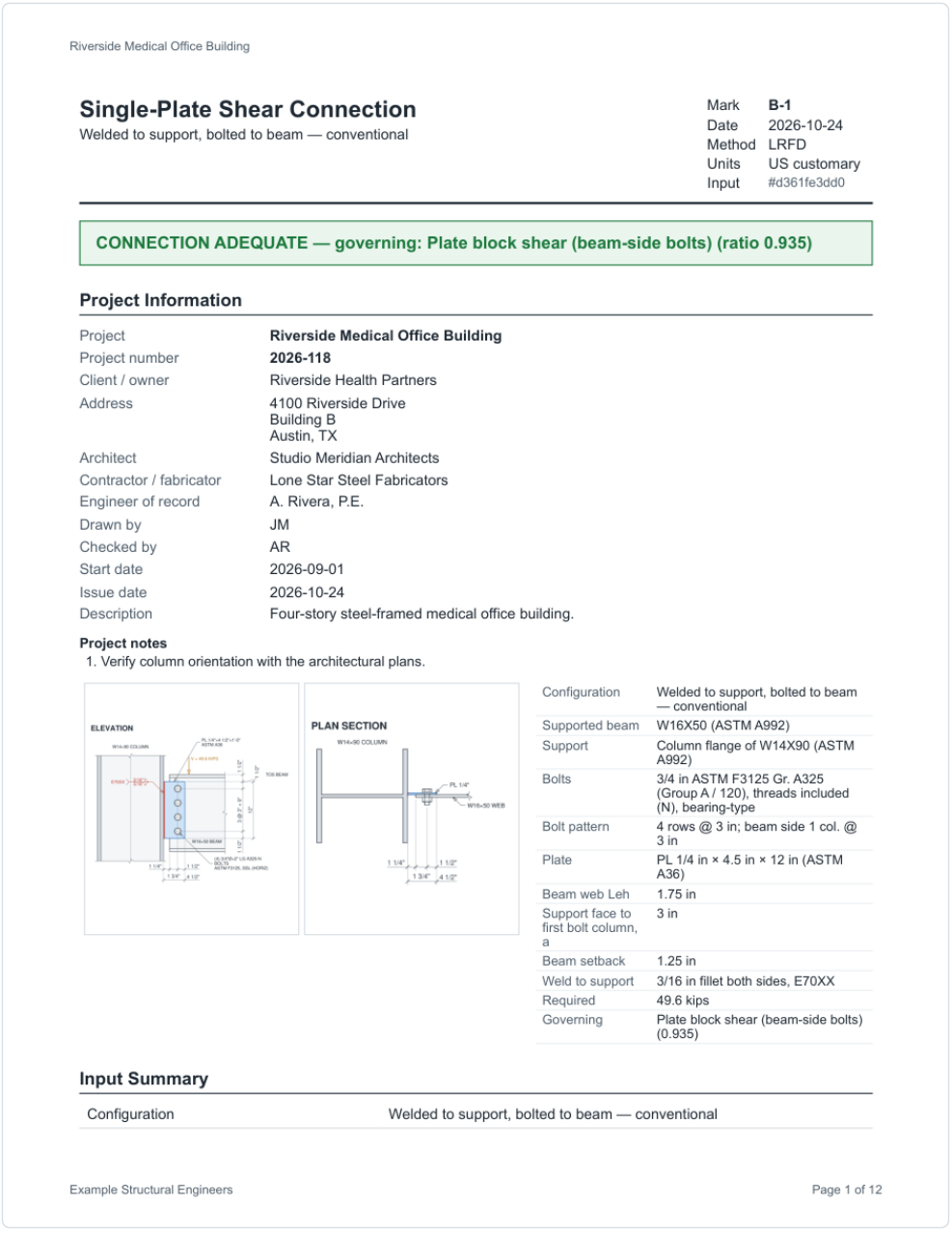



### What the report contains

With the **Standard** preset, each connection's part of the report holds, in order:

1. **Header block**: the connection title and configuration, the mark, the date, the method, the units and the **input hash**. The hash is a short fingerprint of the inputs, so a printed report can be matched to its inputs.
2. **Status banner**: **CONNECTION ADEQUATE** or **NOT ADEQUATE**, with the governing check and ratio. An infeasible connection gets a NOT FEASIBLE banner listing the geometric problems.
3. **Project information**: project number, client, address, architect, contractor, engineer of record, drawn and checked by, dates, description and project notes.
4. **Connection graphic and key data**: small elevation and plan views next to a table of the key sizes and the governing check.

5. **Input summary**: every input value.
6. **Loads and combinations**: the required strength, worked out from the loads.
7. **Summary of checks**: the strength checks grouped by element, with demand, capacity and ratio.
8. **Code compliance**: each detailing requirement, provided against limit, with its clause.
9. **Detailed calculations**: each check step by step, with typeset equations and clause references.
10. **Connection drawings**: full-width elevation and plan.
11. **Appendices**: a summary of every code clause cited, and the revision history when the project has revisions.
12. **Seal / signature block**: prepared by, checked by, reviewer and a seal box.
13. **Disclaimer**.
14. **Drawing sheet**: the title-blocked sheet is appended at the end and counted in the page numbers ("Page x of y").



Riverside Medical Office Building

## Single-Plate Shear Connection

Welded to support, bolted to beam — conventional

| | |
|---|---|
| Mark | **B-1** |
| Date | 2026-10-24 |
| Method | LRFD |
| Units | US customary |
| Input | #d361fe3dd0 |

**CONNECTION ADEQUATE — governing: Plate block shear (beam-side bolts) (ratio 0.935)**

### Project Information

| | |
|---|---|
| Project | **Riverside Medical Office Building** |
| Project number | **2026-118** |
| Client / owner | Riverside Health Partners |
| Address | 4100 Riverside Drive<br>Building B<br>Austin, TX |
| Architect | Studio Meridian Architects |
| Contractor / fabricator | Lone Star Steel Fabricators |
| Engineer of record | A. Rivera, P.E. |
| Drawn by | JM |
| Checked by | AR |
| Start date | 2026-09-01 |
| Issue date | 2026-10-24 |
| Description | Four-story steel-framed medical office building. |

**Project notes**

1. Verify column orientation with the architectural plans.







| | |
|---|---|
| Configuration | Welded to support, bolted to beam — conventional |
| Supported beam | W16X50 (ASTM A992) |
| Support | Column flange of W14X90 (ASTM A992) |
| Bolts | 3/4 in ASTM F3125 Gr. A325 (Group A / 120), threads included (N), bearing-type |
| Bolt pattern | 4 rows @ 3 in; beam side 1 col. @ 3 in |
| Plate | PL 1/4 in × 4.5 in × 12 in (ASTM A36) |
| Beam web Leh | 1.75 in |
| Support face to first bolt column, a | 3 in |
| Beam setback | 1.25 in |
| Weld to support | 3/16 in fillet both sides, E70XX |
| Required | 49.6 kips |
| Governing | Plate block shear (beam-side bolts) (0.935) |

### Input Summary

| | |
|---|---|
| Configuration | Welded to support, bolted to beam — conventional |

Example Structural Engineers Page 1 of 12

*Page 1 of a sample report. The image was printed from the report's HTML by Chromium; the server renders the PDF with WeasyPrint, which may differ slightly in fonts and line breaks.*

The page header and footer print the project, the mark, the firm, the date and "Page x of y". Every page footer carries the input hash.

## Project calculation package and sheet set

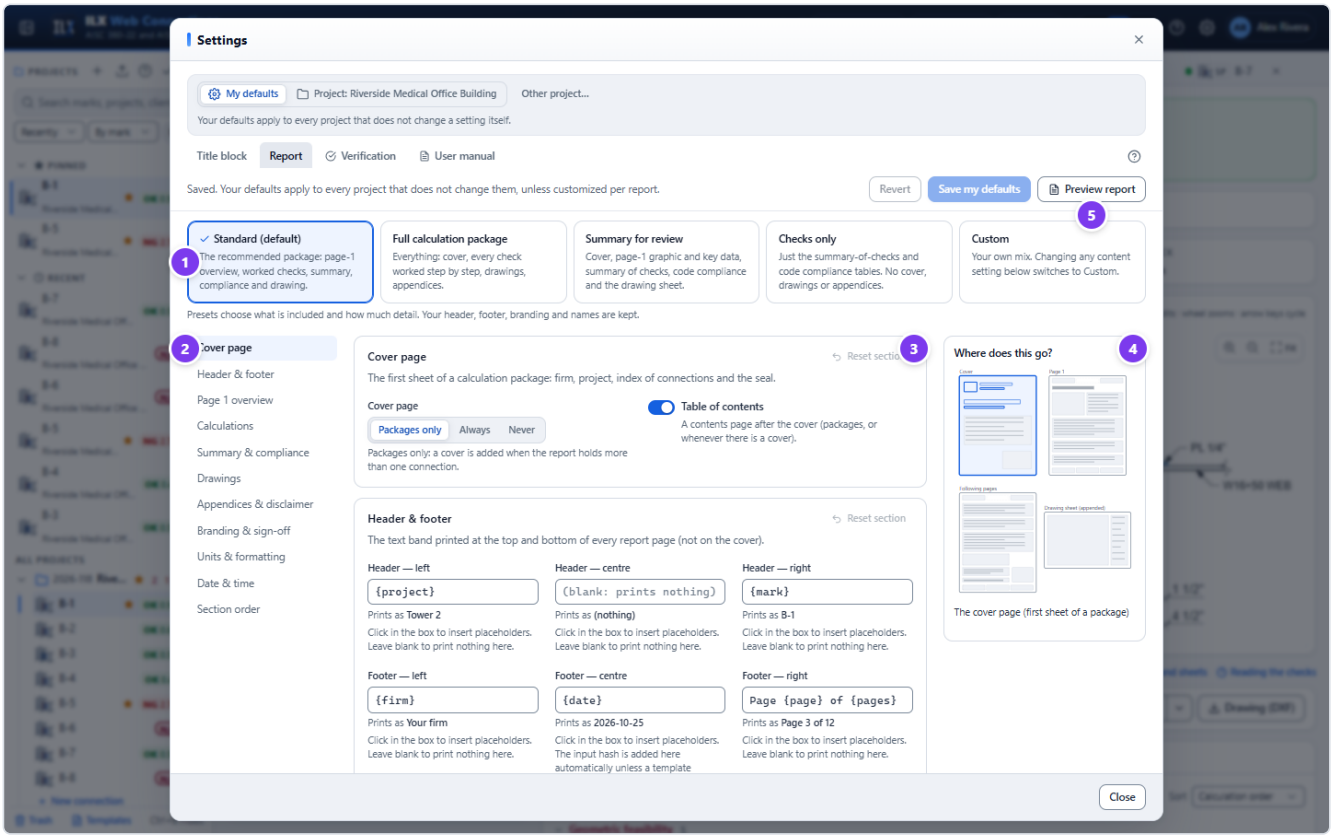
From a project's menu in the project panel:

| Menu item | Produces |
|---|---|
| **Calc package (PDF)** | One report for every connection in the project. By default it has a cover page (firm, project, index of connections and seal) and a table of contents, followed by each connection's report and, at the end, the drawing sheets S-501, S-502 and so on. |
| **All sheets (PDF)** | Every connection's drawing sheet in one PDF, numbered S-501, S-502… with "1 OF n" sheet counts. |

The package uses the project's report and title block settings. Connections appear in the same natural order as the project panel: numbers inside marks sort by value, so B-2 comes before B-10. The drawing sheets S-501, S-502… follow the same order.

## Report settings

Report settings decide what the report contains and how it looks. Open them from **Settings** (the gear in the header) → **Report**, either for **My defaults** or for a project (see Settings).

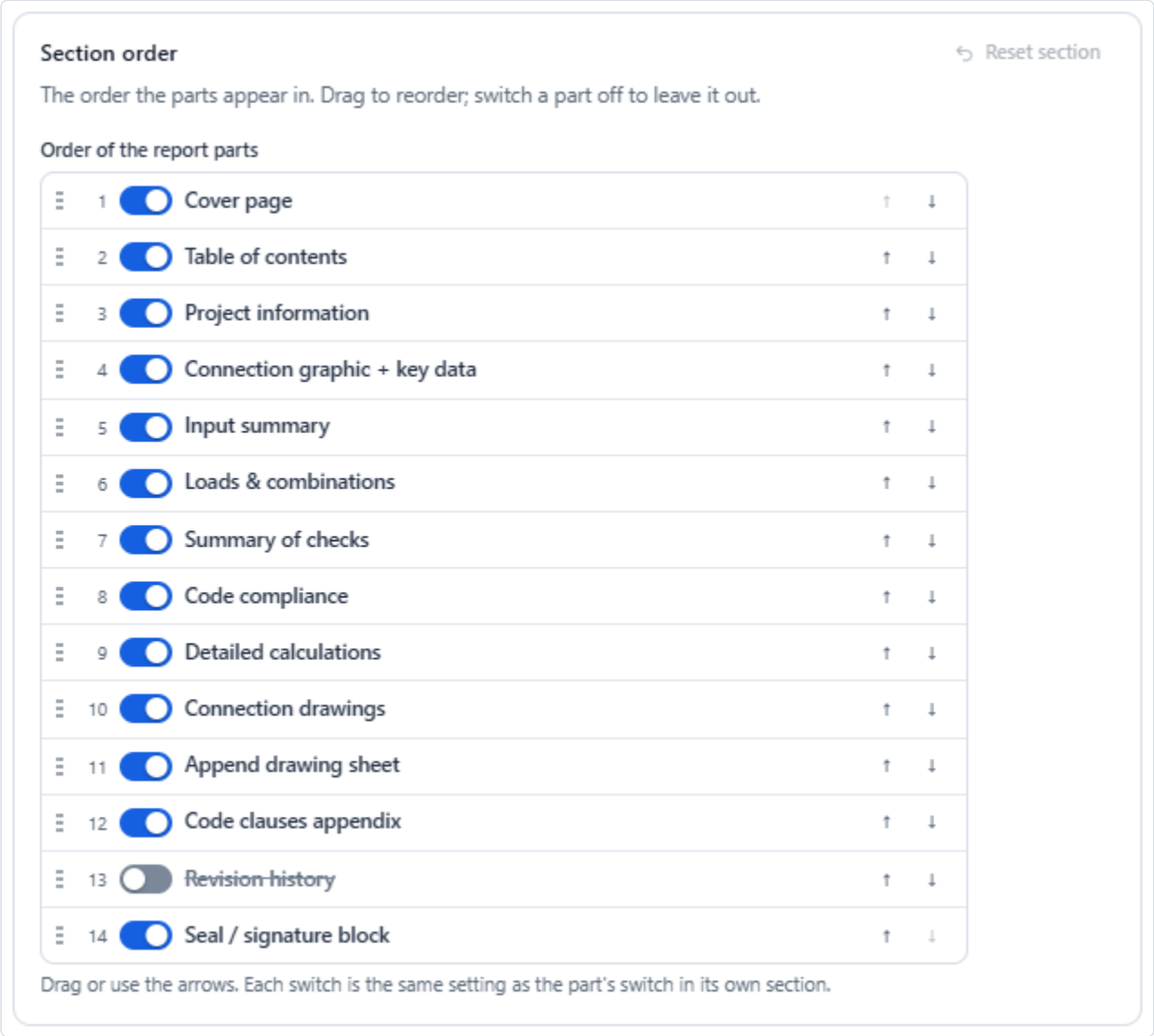



1. **Presets** choose what is included and how much detail. Your header, footer, branding and names are kept when you switch.
   - **Standard (default)**: page-1 overview, worked checks, summary, compliance and the drawing.
   - **Full calculation package**: everything, including the cover, every check worked step by step, the drawings and the appendices.
   - **Summary for review**: the cover, the page-1 graphic and key data, the summary of checks, code compliance and the drawing sheet.
   - **Checks only**: the summary-of-checks and code-compliance tables; no cover, drawings or appendices.
   - **Custom**: selected automatically as soon as you change any content setting.
2. **Section list** in document order. Select a group to jump to it. A dot marks a group with unsaved changes; a red number marks fields to fix.
3. **Settings groups**. Each group has a short explanation and **Reset section** (to the factory defaults, or to your defaults in project scope).
4. **Where does this go?** A thumbnail of the report pages highlights the region the setting you point at affects.
5. **Preview report** renders a real PDF with the current, unsaved settings, using the open connection (or a sample connection), beside the settings. Select **Update preview** after further changes.

## The settings groups

| Group | Settings |
|---|---|
| **Cover page** | **Cover page**: **Packages only** (a cover when the report holds more than one connection), **Always** or **Never**. **Table of contents**. |

| Group | Settings |
|---|---|
| **Header & footer** | Six text boxes: left, centre and right, top and bottom. Click a box to insert placeholders: `{project}`, `{mark}`, `{date}`, `{page}`, `{pages}`, `{firm}`, `{engineer}`, `{hash}`. The line under each box shows how it prints. A blank box prints nothing. |
| **Page 1 overview** | **Project information**, **Connection graphic + key data**, **Input summary** |
| **Calculations** | **Loads & combinations**, **Detailed calculations**, **Detail level** (**Summary**: tables only; **Standard**: hides the working of passing checks below **Show working above ratio**; **Full**: every step), **Only show working for failing checks**, **Code references**, **Clause summaries inline**, **Fix hints** |
| **Summary & compliance** | **Summary of checks**, **Code compliance** |
| **Drawings** | **Connection drawings** (inside the report), **Append drawing sheet** |
| **Appendices & disclaimer** | **Code clauses appendix**, **Revision history**, **Disclaimer** text |
| **Branding & sign-off** | **Logo text** (up to 12 characters; blank uses the title block's), **Accent colour** for headings and rules, **Seal / signature block**, **Prepared by** and **Checked by** (blank uses the project's people), **Reviewer** |
| **Units & formatting** | **Units**: **As entered**, **US**, **SI** or **Dual**, for example 12.0 kips (53.4 kN). **Decimal places** for forces and lengths. **Text size** (8 to 11 pt). |
| **Date & time** | **Time zone** for `{date}`, for example America/Chicago. Blank uses the server's time zone. |
| **Section order** | The order of the report parts. Drag a part, or use the arrows. Each switch is the same setting as the part's switch in its own group. |



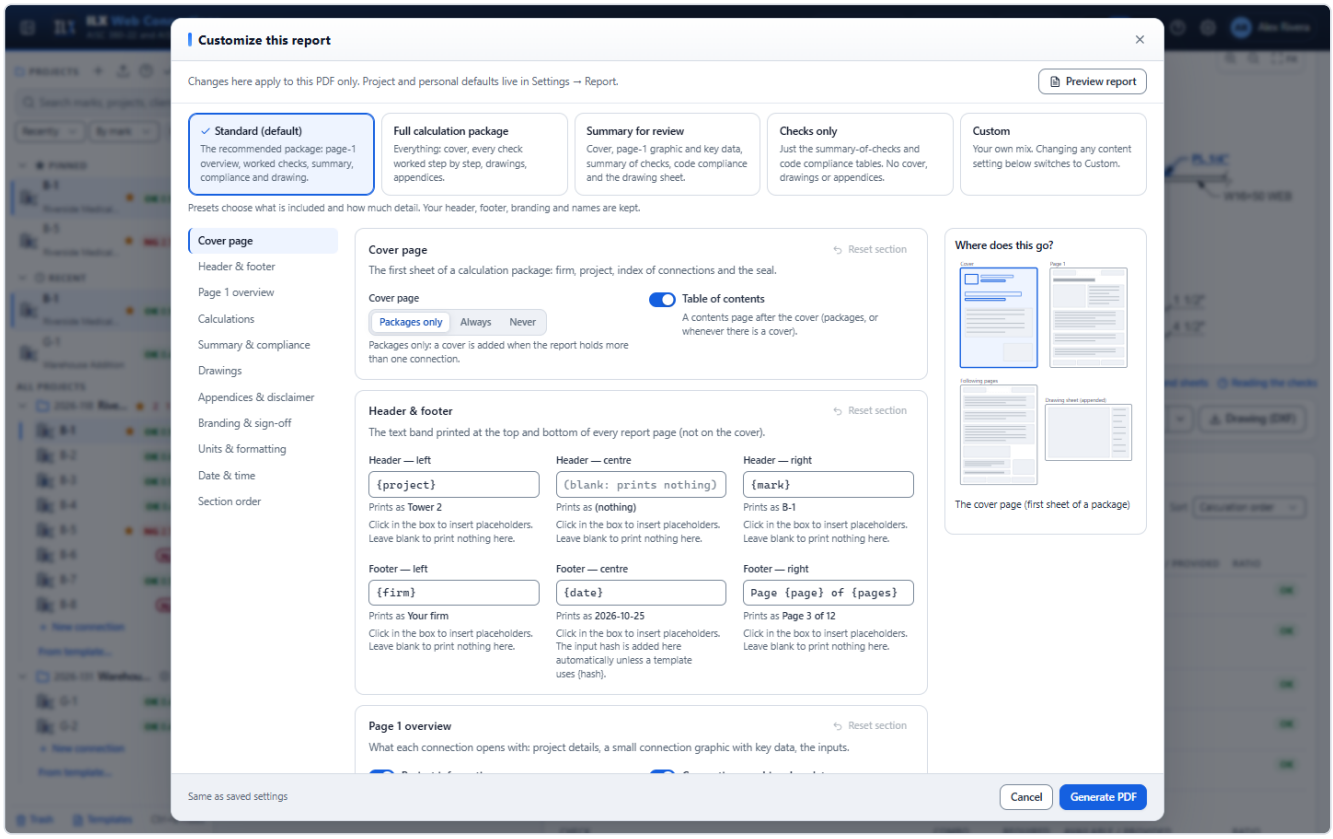

**To save:** select **Save my defaults**, or **Save for this project** in project scope. **Revert** discards your unsaved changes. If you close Settings with unsaved changes, ILX asks before discarding them.

## Customize one report

**Customize this report…** opens the same editor, starting from the saved settings that apply to this connection.

1. Change what you need. The footer shows how many overrides apply to this PDF.
2. Select **Generate PDF**.

Nothing is saved. The overrides apply to this PDF only.


Customize this report
Changes here apply to this PDF only. Project and personal defaults live in Settings → Report.
Preview report
Standard (default)
The recommended package: page-1 overview, worked checks, summary, compliance and drawing.
Full calculation package
Everything: cover, every check worked step by step, drawings, appendices.
Summary for review
Cover, page-1 graphic and key data, summary of checks, code compliance and the drawing sheet.
Checks only
Just the summary-of-checks and code compliance tables. No cover, drawings or appendices.
Custom
Your own mix. Changing any content setting below switches to Custom.
Presets choose what is included and how much detail. Your header, footer, branding and names are kept.
Cover page
Header & footer
Page 1 overview
Calculations
Summary & compliance
Drawings
Appendices & disclaimer
Branding & sign-off
Units & formatting
Date & time
Section order
Cover page
Reset section
The first sheet of a calculation package: firm, project, index of connections and the seal.
Cover page
Packages only
Always
Never
Packages only: a cover is added when the report holds more than one connection.
Table of contents
A contents page after the cover (packages, or whenever there is a cover).
Header & footer
Reset section
The text band printed at the top and bottom of every report page (not on the cover).
Header — left
{project}
Prints as Tower 2
Click in the box to insert placeholders. Leave blank to print nothing here.
Header — centre
(blank: prints nothing)
Prints as (nothing)
Click in the box to insert placeholders. Leave blank to print nothing here.
Header — right
{mark}
Prints as B-1
Click in the box to insert placeholders. Leave blank to print nothing here.
Footer — left
{firm}
Prints as Your firm
Click in the box to insert placeholders. Leave blank to print nothing here.
Footer — centre
{date}
Prints as 2026-10-25
Click in the box to insert placeholders. The input hash is added here automatically unless a template uses {hash}.
Footer — right
Page {page} of {pages}
Prints as Page 3 of 12
Click in the box to insert placeholders. Leave blank to print nothing here.
Page 1 overview
Reset section
What each connection opens with: project details, a small connection graphic with key data, the inputs.
Where does this go?
The cover page (first sheet of a package)
Same as saved settings
Cancel
Generate PDF

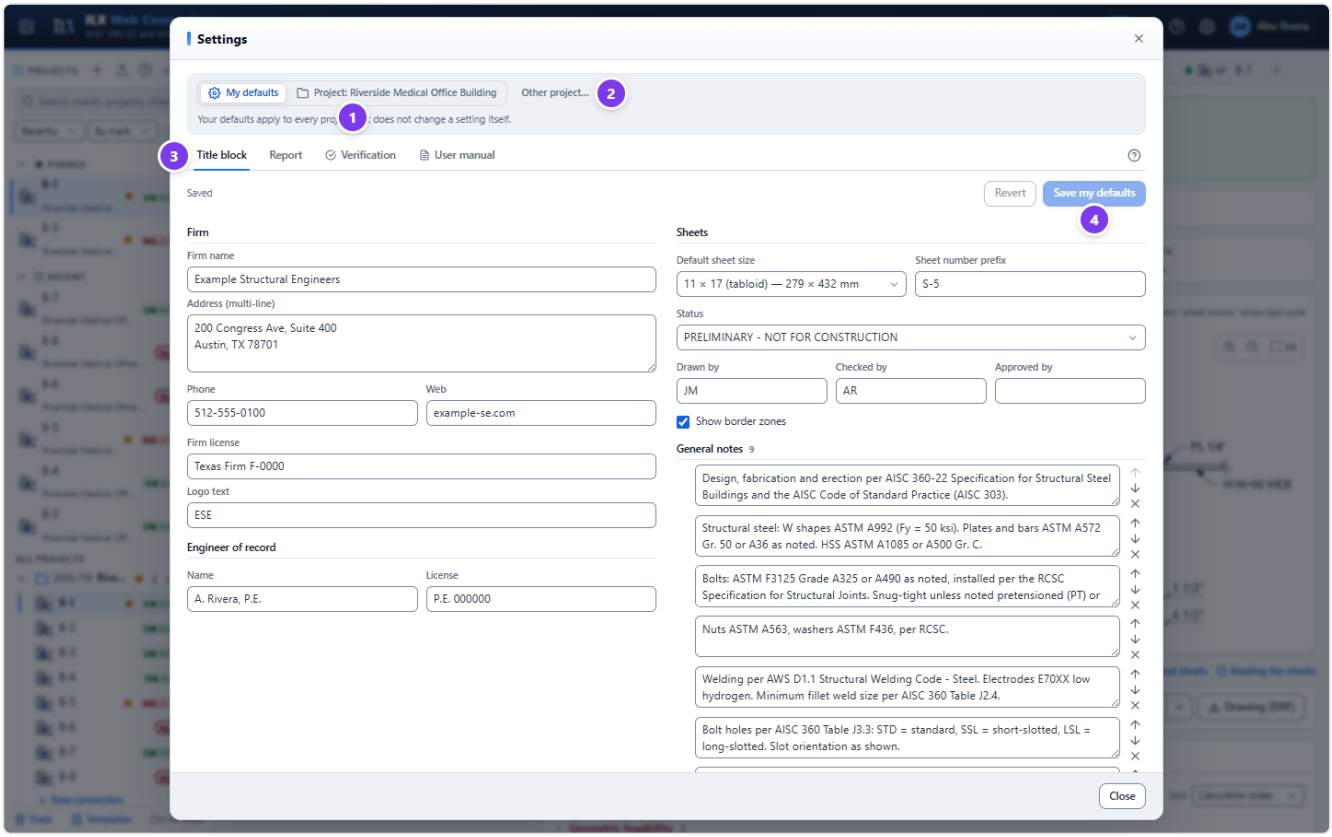
CHAPTER 32

# Settings

Settings hold your **title block** (firm, engineer of record, sheets and general notes) and your **report** configuration. Both belong to you, not to the whole firm, and any project can override them. An engineer who works for several clients can keep one set of defaults and change only what a client's project needs.

## Opening Settings

- **For your defaults:** select the gear in the header. Settings opens on **My defaults**.
- **For a project:** open the project's menu in the project panel and choose **Project settings…**. Or open Settings from the gear and select **Project: <name>** (the active connection's project) or **Other project…** in the scope bar.




1. **Scope bar**: **My defaults** or **Project: <name>**.
2. **Other project…** picks another project.
3. The **Title block** and **Report** tabs. In project scope each tab shows its number of overrides.
4. **Save my defaults** (or **Save for this project**) and **Revert**. Nothing is saved until you select **Save**. Closing with unsaved changes asks first.

## How settings are inherited

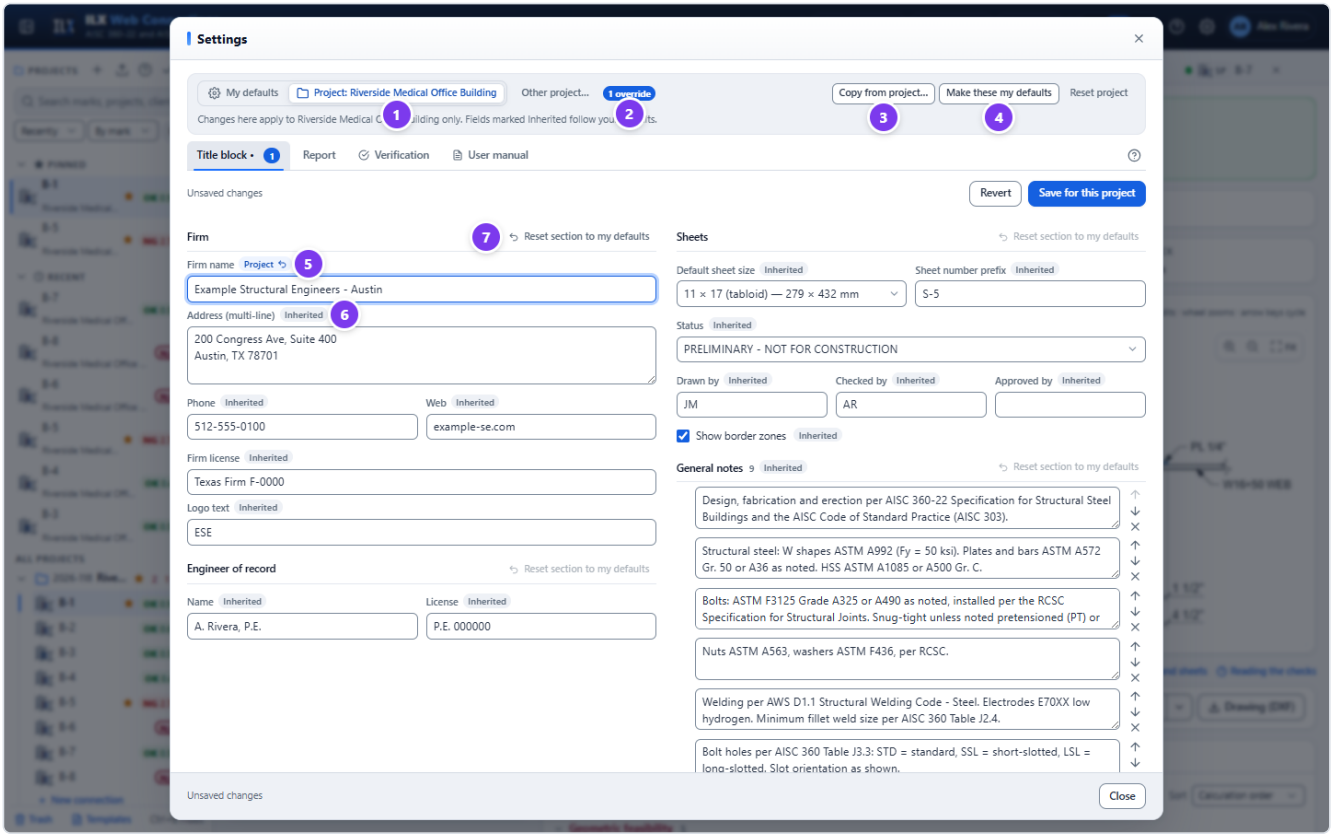
Each setting comes from the first of these layers that sets it:

1. **This report**: the overrides in **Customize this report…**, for one PDF only.
2. **Project overrides**: the fields a project changes.
3. **My defaults**: your own settings.
4. **Built-in defaults**.

A project stores only the fields it changes. A field it does not change follows your defaults, including later changes to them. Changing your firm phone number in **My defaults** therefore updates every project that did not set its own.

Lists (such as the general notes or the report section order) are replaced as a whole when a project overrides them; they are not merged item by item.

## Project overrides




1. **Project scope. Project: <name>** is selected. The help line says changes apply to this project only.
2. **Override count.** The number of fields this project changes, for example **1 override**.
3. **Copy from project…** replaces this project's overrides with another project's, after you confirm.
4. **Make these my defaults** makes this project's settings your defaults for every project that does not override them. This project then simply inherits them.
5. **Project** mark. A field this project changes is marked **Project**. Select its reset arrow to go back to your default.
6. **Inherited** mark. A field that follows your defaults is marked **Inherited**.
7. **Reset section to my defaults** resets every field of one section.

**Reset project** (at the right of the scope bar) removes every title block and report override of the project, which then follows your defaults again.

A project with its own settings shows a gear badge in the project panel.

## Design rules

The **Design rules** tab holds the rules that change the design itself, set for you (**My defaults**) or overridden for one project, like the other settings.

- **Pretension required for every bolted joint.** Tick it when the governing code requires pretensioned or slip-critical bolts throughout the building (RCSC Section 4.2(1)). AISC 360-22 no longer lists these cases, so this is your project decision. The former AISC 360-16 J1.10 cases were column splices and column bracing in tall buildings, buildings with cranes over 5 tons, and supports of running machinery or other sources of impact or stress reversal. With the rule on, each bolted connection gets the **Project rule: bolts pretensioned or slip-critical** check.
- **Reason.** A short text, such as "cranes > 5 tons", printed with that check.
- **RCSC Specification edition.** The default is 2020, the edition AISC 360-22 adopts (Section A2). Choose 2025 (ANSI/AISC 348-25) if your project specifies it. The two editions agree on hole sizes, minimum pretensions, slip resistance and the joints that must be pretensioned, so no bolt strength or pass/fail result changes. The setting changes the cited edition and clause numbers and these notes. With 2025, the calibrated wrench method is gone from the washer list, F3394 reaction and backup washers are listed as an option, and plate washers and bars must have Fy of at least 40 ksi. The coating hint names ASTM F3393 zinc-flake. With 2020, bolts in tension under fatigue get the 30 % prying limit of Section 5.5; RCSC 2025 drops that limit and checks the stress range instead, as AISC 360-22 Appendix 3 does. The drawing general notes name the edition as you type them, so edit them to match.
- **Tightening clearance C1 (AISC Manual Tables 7-15 / 7-16).** For twist-off bolts (F1852, F2280), Table 7-16 (p. 7-81) gives C1 by installation tool and socket. For each diameter (5/8 to 1-1/4 in.) pick the tool and socket your erector uses. The default is the socket the table's staggered-bolt part (p. 7-82) is built on: S-61EZ 2-7/8 in. for 5/8 in., GS-111EZ 3-5/8 in. for 3/4 to 1-1/8 in., GX-361ZA-UC 4-5/8 in. for 1-1/4 in. **Other tool: enter C1** takes the manufacturer's value (Table 7-16 note a). Heavy hex bolts always use Table 7-15.
- **C1 for 1/2 in. bolts.** Tables 7-15 and 7-16 start at 5/8 in., so this value is yours (prefilled 7/8 in.).
- **C1 for bolts the Manual does not tabulate.** Tables 7-15 and 7-16 cover ASTM F3125 bolts only. For Group 200 bolts (ASTM F3043 / F3111; 1, 1-1/8 and 1-1/4 in.) the Manual refers you to the manufacturer (p. 7-4), and no manufacturer publishes tool or socket dimensions. For ASTM A307 bolts and threaded rods (F1554, A36; 5/8 to 1-1/2 in.) there is no table either. Enter C1 per diameter from your erector's tool. A blank field uses the Table 7-15 heavy hex value, shown greyed in the field. Every check states the value as an engineer setting, for example "engineer setting C1 = 1-3/8 in., default the Manual Table 7-15 heavy hex value: the Manual does not tabulate F3043 / F3111 clearances". For F3043 the entering clearance C2 is the round head diameter / 2 + 1/8 in. (the Table 7-16 note, with the ASTM F3043 head: 1-3/16, 1-5/16 and 1-3/8 in.).

Every clearance check states the C1 it used and where it comes from, for example "C1 = 1.188 in. (Manual Table 7-16, S-61EZ, 2-1/8 in. dia. socket (project setting))" or "engineer-entered C1 = 7/8 in. (project setting): 1/2 in. bolts are not tabulated in Manual Tables 7-15 / 7-16".

**Fatigue details (AISC 360-22 Appendix 3).** Connections with **Cyclic loading** ticked are designed for fatigue (see Fatigue (Appendix 3)). Two fabrication choices set the Table A-3.1 category of details ILX does not model:

- **Reentrant corners of copes and weld access holes. As cut** (default, conservative): drilled, punched or thermally cut, Category E′ for a radius of at least 3/8 in. (Cases 1.3 and 1.4). **Predrilled / reamed, or thermally cut and ground to bright metal**: Category C for a radius of at least 1 in. A radius under 3/8 in. fails (Section 3.5).
- **CJP groove welds under cyclic load are RT or UT tested.** Off by default. Eq. A-3-1 applies to CJP groove welds only when they are radiographically or ultrasonically tested and meet AWS D1.1 clause 8.12.2 or 8.13.2 (Section 3.6). Until it is on, each connection with a CJP fatigue detail (directly welded FR flanges, CJP flange plates, CJP column-to-base-plate welds) shows the requirement **CJP groove welds under cyclic load: RT or UT tested** as not met. Specify the testing on the drawings and tick it.

The rules apply to the connections of the project when you design them, in their saved summaries, in the calculation reports and on the drawing sheets. It is saved as soon as you change it.

## Title block settings

| Section | Fields | Used on |
|---|---|---|
| **Firm** | Firm name, address (multi-line), phone, web, firm license (for example *Texas Firm F-1234*), logo text (the short text in the logo box) | Title block; report cover and footer (`{firm}`) |
| **Engineer of record** | Name, license | Seal block; report (`{engineer}`) |
| **Sheets** | **Default sheet size** (11 × 17, 8.5 × 11 landscape or portrait), **Sheet number prefix** (S-5 gives S-501, S-502…), **Status** (for example *PRELIMINARY - NOT FOR CONSTRUCTION* or *ISSUED FOR CONSTRUCTION*), **Drawn by**, **Checked by**, **Approved by**, **Show border zones** (the 1-2-3-4 / A-B-C-D zone marks) | Drawing sheets |
| **General notes** | A numbered list of notes. Edit, reorder with the arrows, remove, or **Add note**. | Every drawing sheet, followed by the project notes |

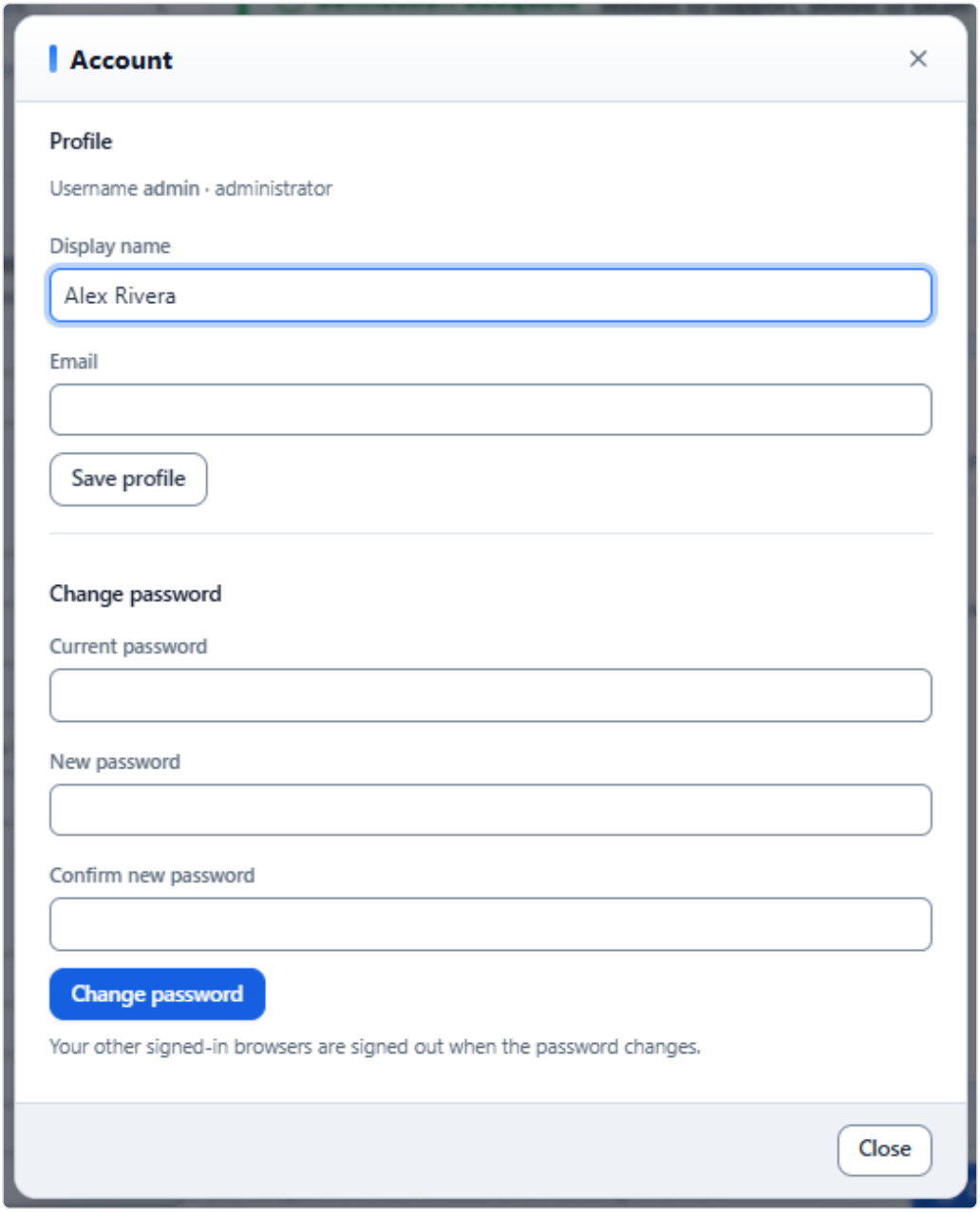
The project information (Project information) supplies the project-specific fields: the project, client, address, revisions, and the drawn, checked and engineer names. A name entered in the project information takes precedence over the title block default.

## The Report tab

The **Report** tab holds the calculation report configuration: presets, the parts included, the detail level, header and footer, branding, units and the section order. It works the same way in both scopes. See Report settings.

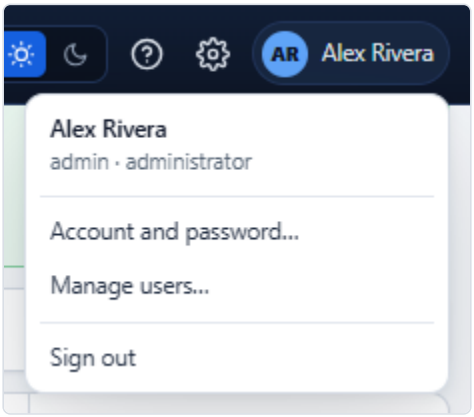
# Users and administration

## Your account

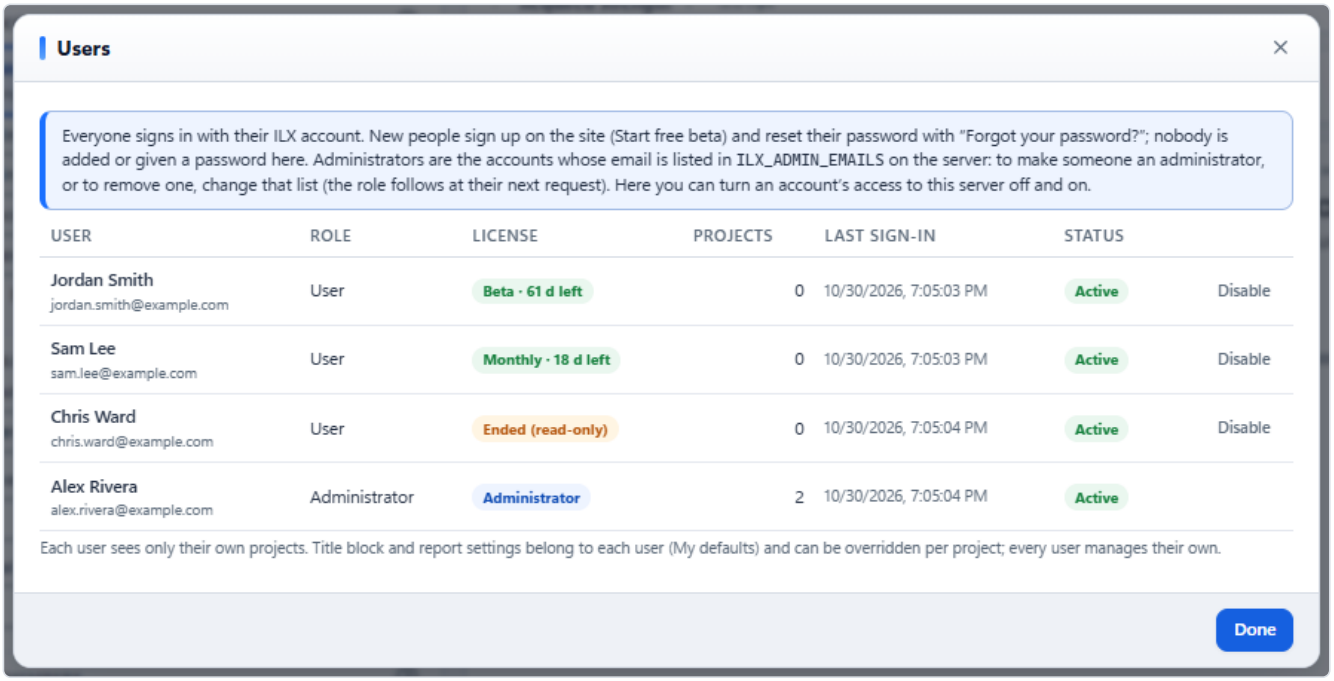



Open the account menu (your name in the header) and choose **Account and password…**:

- **Profile.** Change your **Display name**, then select **Save profile**. Your email address is your ILX account and cannot be changed here; contact support to change it.
- **Password.** Your password belongs to your ILX account. To change it, sign out and use **Forgot your password?** on the sign-in page.




## Managing users (administrators)




Administrators see **Manage users…** in the account menu. It lists every account on this server with its role, its license (plan and days left, or *Ended (read-only)*), its number of projects and its last sign-in.

| To… | Do this |
|---|---|
| Add a user | Nobody is added here. The person selects **Start your free beta** on the sign-in page (or the website) and creates their ILX account; they appear in the list after their first sign-in. |
| Make someone an administrator, or remove one | Roles come only from the server setting `ILX_ADMIN_EMAILS` (the administrators' emails). Ask whoever runs the server to add or remove the email; the role changes at that person's next page load. The dialog says so. |
| Reset a password | Not here: the person uses **Forgot your password?** on the sign-in page. |
| Disable or enable | Select **Disable**. The person is signed out and cannot use this server until enabled again; their projects are kept. Their ILX account itself is not affected. You cannot disable yourself. |

An account marked **Local (cannot sign in)** is left over from before ILX accounts. The server administrator moves its projects to an ILX account and deletes it ( `INSTALL.md` , section 7).

Each user sees only their own projects, templates and settings. Title block and report settings belong to each user (**My defaults**) and can be overridden per project; every user manages their own, and the dialog says so. The table shows each user's number of projects and last sign-in.

**Running without accounts.** A server started with `ILX_AUTH=0` has no sign-in, and everyone shares the same projects. This is intended for a single-user installation (development only).

## The Verification page

**Settings → Verification** shows how ILX was verified against the AISC Design Examples. See Verification. **Settings → User manual** shows this manual. The scope bar is hidden on these two tabs, because they do not depend on a project.

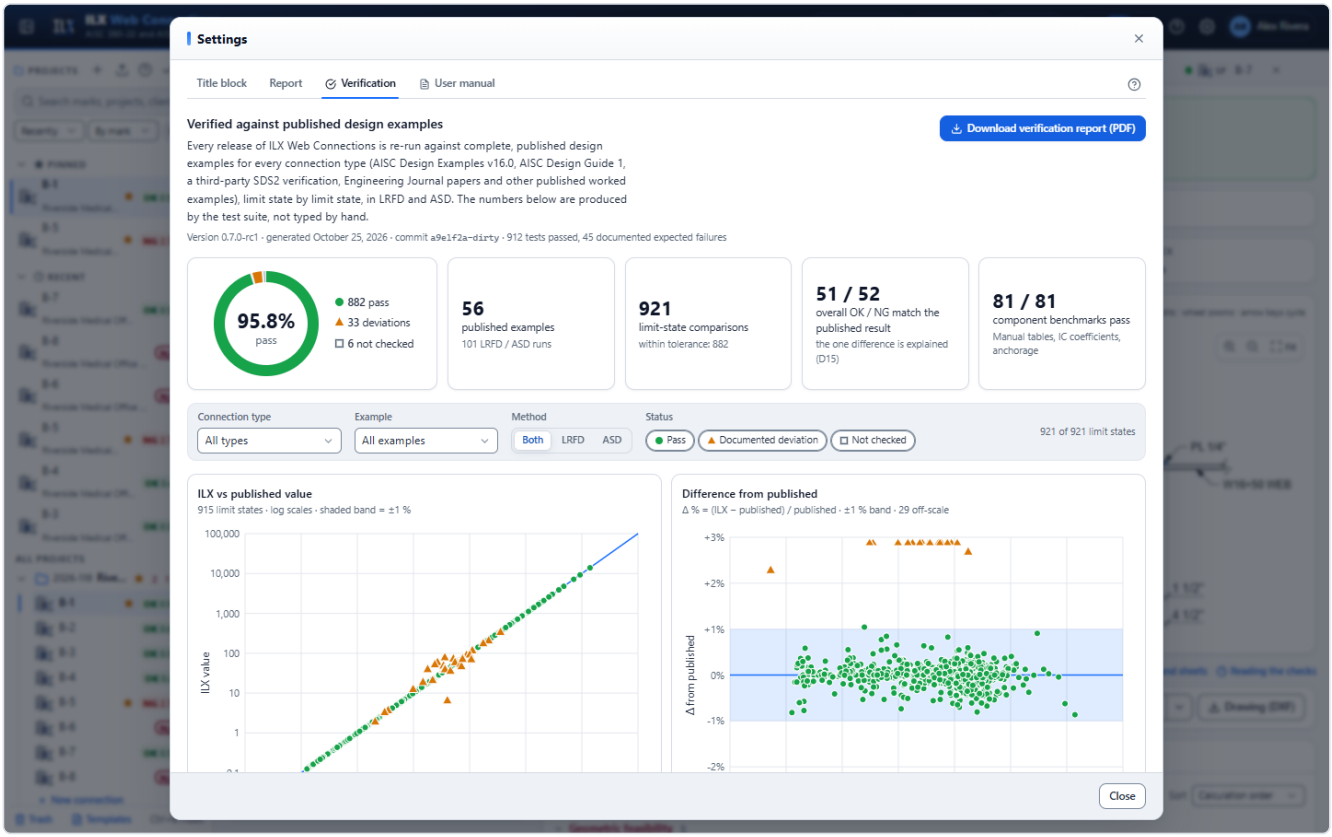
CHAPTER 33

# Verification

Every ILX connection type and calculator is verified against complete published design examples, limit state by limit state, for LRFD and ASD. The full record for the single plate and single angle is the *Verification Manual* (`docs/VERIFICATION_MANUAL.md`, issue 0.4-B); the other types are summarised in its §6C and shown in full on the Verification page. This page summarises it.

## Opening the Verification page

1. Select the gear in the header to open **Settings**.
2. Select the **Verification** tab.




The page is generated from the verification tests at each release. It shows the version and date it was generated, and contains:

- **Summary tiles**: the pass ring (pass, deviations, not checked), the number of published examples and LRFD/ASD runs, the limit-state comparisons, how many overall OK/NG results match the published ones, and the component benchmarks that pass.
- **Filters**: **Connection type**, **Example**, **Method** (Both, LRFD or ASD) and **Status**.

- **Charts**: ILX against published values, and the difference from the published value against the tolerance band. Click a point to open its row.
- **Examples**: one card per example, with the source and pages. Expand a card for the limit-state table: published value, ILX value, Δ %, Δ against the tolerance and the status.
- **Documented deviations, in plain language**: what differs, why, and its effect, with a link to the affected examples.
- **Component benchmarks**: instantaneous-center coefficients, bearing and tearout, and slip resistance.
- **Download verification report (PDF)**: the same content as a typeset PDF to attach to a submittal.

## Method

1. **Benchmark comparison.** Each published example is entered into ILX field by field, using the drawn geometry. Every published quantity is compared with the value ILX computes: the check capacity, its demand, or a named calculation step. The numbers in the tables are generated by the test code, not typed.
2. **Component benchmarks.**
   - Instantaneous-center coefficients against Manual Tables 7-6, 7-7, 8-4, 8-8 and 8-10.
   - Bearing and tearout against Tables 7-4 and 7-5.
   - lc along the resultant against Design Example II.A-1B.
   - Slip resistance against Design Example J.4A and J3.9.
   - The rows of Manual Table 10-9.
3. **SI and US consistency.** Every example is run again in SI units, and every check must match its US result.
4. **Robustness.** Property-based fuzzing sends thousands of random and plausible inputs. None may produce a server error, and the strength ratios must be sensible.

**Acceptance.** Forces must agree within ±1 % or ±0.5 kip, whichever is larger. Moments and coefficients must agree within ±1 %, and interaction ratios within ±2 %. Published values are printed to three significant figures, so they carry up to 0.5 % rounding.

## Single plate and single angle

| Example | What it is | ILX result | Published |
|---|---|---|---|
| AISC II.A-17A | Conventional single plate to a column flange | OK | OK |
| AISC II.A-17B | Conventional single plate, axial + shear, to a column flange | OK | OK |
| AISC II.A-18 | Single plate to a girder web, top-coped beam | OK | OK |
| SDS2 II.A-18 | SDS2's published verification of II.A-18 (all 17 values) | OK | OK |
| AISC II.A-19A | Extended single plate to a column web | OK | OK |
| AISC II.A-19B | Extended single plate, axial + shear, to a column web | NG (column-web yield line) | NG (same reason) |
| AISC II.A-28A | All-bolted single angle to a girder web | OK | OK |
| AISC II.A-29 | Bolted/welded single angle (element checks only) | – | OK |
| Muir & Thornton (2011) | Conventional single plate | OK | OK |

| Example | What it is | ILX result | Published |
|---|---|---|---|
| Muir & Hewitt (2009) | Extended single plate | NG | OK (2009 code edition, see below) |
| Dowswell (2018) | Single-coped beam | – | OK |
| AISC K.6 | Single plate to a rectangular HSS column | OK | OK |
| AISC K.7 | Through plate to a rectangular HSS column | OK | OK |
| AISC K.8 | Longitudinal plate on a round HSS, load perpendicular to the HSS axis | OK | OK |

Across the 14 examples, 404 limit-state values were compared in LRFD and ASD:

- **376 pass** within tolerance. All 82 values from the HSS examples K.6, K.7 and K.8 pass.
- **24 are documented deviations.**
- **4 are limit states ILX does not compute** (coverage gaps).

All 24 component benchmarks pass, most within 0.5 %. In every AISC example it models, ILX reaches the published overall result for the published reason.

## Every connection type

The Verification page shows all of them; filter by **Connection type** to see one.

| Type | Examples | Limit states | Pass | Deviation | Not checked |
|---|---|---|---|---|---|
| Single plate | 12 | 352 | 336 | 16 | 0 |
| Single angle | 2 | 52 | 40 | 8 | 4 |
| Double angle | 11 | 137 | 135 | 2 | 0 |
| Shear end plate | 3 | 126 | 124 | 2 | 0 |
| Seated | 9 | 98 | 94 | 2 | 2 |
| Shear tee | 4 | 60 | 60 | 0 | 0 |
| Skewed bent plate | 1 | 34 | 32 | 2 | 0 |
| FR moment | 4 | 108 | 108 | 0 | 0 |
| Beam shear splice | 2 | 64 | 58 | 6 | 0 |
| Compression splice | 3 | 44 | 44 | 0 | 0 |
| Truss connection | 3 | 124 | 124 | 0 | 0 |
| Hanger | 1 | 60 | 60 | 0 | 0 |
| Beam bearing plate | 3 | 42 | 42 | 0 | 0 |
| HSS strut end plate | 1 | 28 | 28 | 0 | 0 |
| Base plate | 14 | 70 | 67 | 3 | 0 |
| Bracket plate | 2 | 90 | 86 | 4 | 0 |
| Bolt group | 3 | 26 | 26 | 0 | 0 |
| Weld group | 3 | 28 | 28 | 0 | 0 |

| Type | Examples | Limit states | Pass | Deviation | Not checked |
|---|---|---|---|---|---|
| Bolted joint | 5 | 66 | 66 | 0 | 0 |
| Welded joint | 4 | 54 | 54 | 0 | 0 |
| **Total** | **90** | **1,663** | **1,604** | **45** | **14** |

All 85 component and Manual-table benchmarks pass. Each deviation and coverage gap has its plain-language explanation on the Verification page.

## Known deviations (single plate and single angle)

These differences are method choices that AISC permits, or errors and edition differences in the sources. None changes an overall result.

| Topic | Difference | Why |
|---|---|---|
| Tearout (II.A-17B, II.A-19B) | ILX's per-bolt tearout is higher | ILX measures lc along the line of action of the resultant (the II.A-1B method). The examples take the smaller edge distance as a conservative worst case and note that "if a computer program is available, the true lc can be calculated". Tearout does not govern. |
| Bolt group at 38.7° (II.A-19B) | ILX's C is 7 % higher | ILX solves the instantaneous center at the true load angle; the example interpolates Table 7-7 at 30°. At 30°, ILX matches the table. |
| Single-angle eccentricity (II.A-28A) | Support leg +10 %, beam leg −20 % | ILX applies e = g + tw/2 on the support leg and e = a on the beam leg; the example neglects the beam-leg eccentricity (Manual Fig. 10-15). The governing strength agrees within 0.3 %. This is an open decision. |
| Beam-web edge bolt (II.A-28A) | Higher per-bolt value | The example conservatively uses le = 1¼ in. instead of the drawn 1½ in. |
| Muir & Hewitt (2009) | Bolt shear, tearout and net area differ | The paper used AISC 360-05, Fnv = 60 ksi, no tearout between holes and dh = 1⅛ in. ILX uses AISC 360-22 (Group 150 bolts, J3.11a, B4.3b). Under the current Manual the connection is NG, as ILX reports. |
| Table 10-10c (II.A-18) | +4 % web block shear at the stated leh | The tabulated value back-calculates to leh = 2.0 in.; at 2.0 in., ILX matches. |

## What is not yet verified

- **Double-coped beams** (Dowswell and Whyte, 2014): no complete published example reproduced yet.
- **Prying of angle support legs in tension**: no published example found.
- **Slip-critical design** at connection level: only the per-plane value is verified.
- **HSS supports** beyond K.6 to K.8: no published example with axial load on a rectangular HSS wall.
- **Structural integrity (B3.9)** and **rotation capacity**: no published numeric example.

## Items for engineering review

The Verification Manual also lists items for the engineer of record's attention:

- the Mmax expression used for the extended-procedure tmax;
- the double-cope method;

- the B3.9 wording as implemented;
- prying of angle support legs;
- the approximate yield-line mechanism used for girder webs under axial load.

Read them before you rely on ILX for those cases.

## Running the verification yourself

The verification is part of the automated test suite. From the `backend` folder:

```
.venv/Scripts/python -m pytest -q tests/test_verification_*.py
.venv/Scripts/python -m tests.test_verification_examples   > examples.md
.venv/Scripts/python -m tests.test_verification_components > components.md
```

The first command runs the comparisons of every type's golden examples. `tools/export_verification.py` (from the repository root) writes the data of the Verification page. The other two regenerate the comparison tables of the Verification Manual. When a deviation is fixed, its expected-failure test starts passing and the suite turns red, so the documentation cannot drift from the code.

CHAPTER 34

# FAQ and troubleshooting

## Frequently asked questions

**Why is my connection NG when the governing ratio is below 1.0?** A detailing (code-compliance) rule fails, for example an edge distance, the conventional spacing or the 5/8 tp weld rule. Strength is fine. Select the **code-compliance violation** pill in the status bar to see which rule fails and how to fix it.

**What is the difference between NG and NOT FEASIBLE?** NG means the connection can be built but fails a check. NOT FEASIBLE means it cannot be built as entered, for example a bolt outside the plate or a plate deeper than the web. Strength results of an infeasible connection are greyed out because they are meaningless. See NOT FEASIBLE.

**Which bolt-group method should I use?** The default, **(C/n) × Σ per-bolt**, is the method of the AISC Design Examples v16 (II.A-17A, II.A-18). **C × weakest bolt** (Muir and Thornton) is more conservative. **Split** reproduces the concentric-bearing approach of some commercial programs. See Bolts and holes.

**Does ILX use the Manual tables for C?** No. It solves the instantaneous center directly for your bolt pattern, eccentricity and load angle. It matches Tables 7-6 and 7-7 within about 0.5 % where they apply, and it also covers patterns and angles the tables do not.

**Why does the conventional procedure reject my metric spacing?** It does not, within rounding. The conventional procedure requires s = 3 in., which is 76.2 mm. For metric input, dimensional rules accept values within 0.5 mm of the inch limit, so 76 mm meets s = 3 in. (as 38 mm meets 1-1/2 in.). The check then notes *Met within the 0.5 mm metric rounding tolerance*. 75 mm is 1.2 mm short and is a different spacing, so it still fails. Strength checks, feasibility and thickness limits get no tolerance.

**Where do the firm name and the engineer on the sheet come from?** From **Settings → Title block**: your defaults, or the project's overrides. Project-specific names (engineer, drawn by, checked by) come from **Project information** and take precedence. See How settings are inherited.

**Can other users see my projects?** No. Projects, templates and settings belong to your account. To share a project, export the project file and send it; the colleague imports it as a new project.

**How do I get back a value I changed yesterday?** Open the connection menu → **Version history…** and restore an earlier version. The current state is saved first, so you can always undo the restore.

**Why did my saved connection show "Results changed since this connection was saved"?** The calculation engine was updated after the connection was saved, and the result is different now. The banner shows the saved and the current status and ratio. Review the connection before you reissue it.

**Can I edit the DXF?** Yes. Dimensions are true DIMENSION entities, bolts are blocks, and the title block is a block with attributes. See DXF files.

## Common problems

| Problem | What to do |
|---|---|
| **"Cannot reach the server"** | The browser cannot contact ILX. Check the address and your network connection, then select **Try again**. If it persists, ask your administrator to check that the ILX container is running. |
| **The sign-in screen comes back while I am working** | Your session expired, you changed your ILX password, or an administrator turned your access off. Sign in again. Unsaved typing in the last second may need to be re-entered. |
| **"Check the highlighted inputs."** | A value is out of range or inconsistent (for example an HSS support with a W shape). The field shows the message. The results show the last valid design, marked *Showing the last valid result*, until you correct it. |
| **"support: hss_wall needs a HSS shape"** | The support member is a W shape on an HSS wall, for example in an imported file. Pick an HSS in **Support member** and an HSS material (A500 Gr. C, A1085 or A53 Gr. B). Changing **Support** in the editor now picks a matching member and material for you. |
| **Save failed** (red message) | The change could not be saved, for example because the network dropped. Your edits stay in the tab; ILX saves again on your next change. Do not close the tab until the ● mark disappears. |
| **A report or sheet says "not available on this server"** | The PDF engine (WeasyPrint) is missing. It is included in the standard ILX container; ask your administrator. Drawing sheets and DXF do not need it. |
| **Preview report fails** | The preview renders a real PDF on the server and needs the same PDF engine. |
| **Compare is greyed out** | Open at least two connections in tabs. |
| **I cannot pick an angle configuration** | Angles cannot be bolted to an HSS wall. Change **Support** first, or use a welded plate. |
| **The plate does not fit the beam** (NOT FEASIBLE, *Plate within the beam web clear depth*) | Reduce the number of rows or Lev, move the plate (**Top of beam to top of plate**), or change the cope. The fix hint gives the limits. |
| **The beam clashes with the girder flange** | Apply the **Suggested fixes** (a top cope sized from the girder kdes and flange projection), or set **Girder TOS − beam TOS**. |
| **A batch row says "Load mode switched…"** | The row's load columns belong to another load mode than the base connection's (for example RD / RL with a **Required strength** base). ILX switched the row to the mode of its columns. Add a `load_mode` column to choose the mode yourself. |
| **Restore is greyed out in the trash** | The connection's project is also in the trash. Restore the project first. |
| **Import says "Not an ILX Connection project file"** | Only `*.ilxproj.json` files exported by ILX can be imported (formats 1 to 4, up to 5 MB). |
| **The drawings are dark when I print a screenshot** | Turn off **Dark drawings**, or switch to the light theme. |
| **Forgot password** | Select **Forgot your password?** on the sign-in page; we email you a code. |

CHAPTER 35

# Glossary

## Terms

| Term | Meaning |
|---|---|
| **a** | Distance from the weld line (or the angle heel) to the first beam-side bolt column (Manual Part 10). |
| **Advisory** | A failed recommendation. It is shown, but it does not make the connection inadequate. |
| **ASD / LRFD** | Allowable strength design (Rn/Ω ≥ Ra) and load and resistance factor design (φRn ≥ Ru), AISC 360-22 B3. |
| **Auto-design** | ILX's search for an adequate design, keeping pinned fields. |
| **Batch design** | Designing a schedule of marks from one base connection. |
| **Bolt group method** | How the strength of an eccentric bolt group is assembled from per-bolt strengths: *sum* (default), *min* or *split*. |
| **C, C′** | Instantaneous-center coefficients of an eccentric bolt group: C for the combined load, C′ for moment only (Manual Part 7). |
| **Calc package** | One PDF report for every connection of a project, with a cover and contents. |
| **Code compliance** | The detailing and applicability requirements (spacing, edge distances, hole types, procedure limits, weld rules), listed by topic. |
| **Combo** | The load combination that governs a check. |
| **Configuration** | The support-side / beam-side fastening: Weld / Bolt, Bolt / Bolt, Bolt / Weld or Weld / Weld. |
| **Conventional / extended** | The two single-plate procedures of Manual Part 10: Table 10-9 (conventional) and Muir and Hewitt (extended). |
| **Cope, c, dc** | A cut-out of the beam flange and web at the end; c is its length, dc its depth. |
| **Envelope** | The worst result over all load combinations or cases. |
| **Feasibility** | Whether the geometry can physically be built. A connection that cannot is NOT FEASIBLE. |
| **Fix hint** | The "How to fix" text under a failing check. |
| **Governing check** | The strength check with the highest ratio. |
| **Input hash** | A short fingerprint of a connection's inputs, printed on every report page. |
| **Keynote** | A numbered tag that replaces a long callout on a letter-size sheet; the text is listed under the view. |
| **Leh, Lev** | Horizontal and vertical edge distances to the bolt centre. |
| **lc** | Clear distance used for tearout, measured along the line of action of the bolt force. |
| **Mark** | The connection's identifier, such as B-1. |
| **My defaults** | Your own title block and report settings, which apply to every project that does not override them. |
| **NG** | Not good: a strength or code-compliance check fails. |

| Term | Meaning |
|---|---|
| **N/F, NOT FEASIBLE** | The geometry cannot be built; strength results are not meaningful. |
| **OSL** | Outstanding leg of an angle (the leg on the beam). |
| **Pin (field)** | Locks an input so auto-design keeps its value. |
| **Pin (connection, project)** | Keeps it at the top of the project panel. |
| **Project override** | A setting that a project changes from your defaults. |
| **Ratio** | Required ÷ available (or, for a detailing check, the measured value against the limit). OK when ≤ 1.0. |
| **Setback** | The gap between the support face and the beam end. |
| **SSL, LSL, OVS, STD** | Short-slotted, long-slotted, oversized and standard holes (AISC 360-22 Table J3.3). *Horizontal* or *vertical* is the slot direction on the drawing. |
| **Suggested fix** | A value ILX computes to resolve a problem, applied with one click. |
| **Template** | A saved set of connection inputs for reuse in any project. |
| **tmax** | The maximum plate thickness for rotational ductility (Table 10-9, or Mmax for the extended procedure). |
| **TOS** | Top of steel. |

## Clause labels

Checks cite AISC 360-22 sections directly (for example **J3.4**, **J4.2(b)**, **B4.3b**, **M2.2**) or a short label for a Manual part or other standard:

| Label | Source |
|---|---|
| **P7-IC** | Eccentric bolt groups, instantaneous center (Manual Part 7) |
| **P7-CLR** | Entering and tightening clearances (Manual Part 7) |
| **P8-IC** | Eccentric weld groups (Manual Part 8) |
| **P8-1SIDE** | Single-sided fillet welds (Manual Part 8) |
| **P9-BASE** | Base metal to develop the welds (Manual Part 9) |
| **P9-COPE** | Coped beams (Manual Part 9) |
| **P9-PRY** | Prying action (Manual Part 9) |
| **P9-OOP** | Webs under out-of-plane loads (Manual Part 9) |
| **P9-YL** | Column-web yield line (Manual Part 9, Eq. 9-45) |
| **P10-SP** | Single-plate connections (Manual Part 10) |
| **P10-T109** | Manual Table 10-9 |
| **P10-EXT** | Extended single-plate connections (Manual Part 10) |
| **P10-STAB** | Stabilizer plates (Manual Part 10) |
| **P10-ROT** | End rotation of simple connections |
| **P10-DUCT** | Rotational ductility |
| **P10-HSS** | Single plate to HSS (Manual Part 10) |

| Label | Source |
|---|---|
| **P10-GEO** | Framing geometry |
| **P10-MINT** | Minimum element thickness |
| **P12-INT** | Plate interaction (Manual Part 12) |
| **A3** | AISC 360-22 Appendix 3, fatigue |
| **A341** | AISC 341, gravity connections in braced-frame bays |
| **ASCE7-2.3, ASCE7-2.4** | ASCE 7-22 strength and allowable stress load combinations |
| **RCSC-2.3, RCSC-6** | RCSC Specification: bolt length and threads; washers |
| **K-AREA** | Welding in the k-area |
| **DE-IIA1B** | AISC Design Example II.A-1B (lc along the resultant) |

The **Code clauses appendix** of the report gives a short summary of every clause cited.

# ILX **Connection**

Steel connection design to AISC 360-22

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