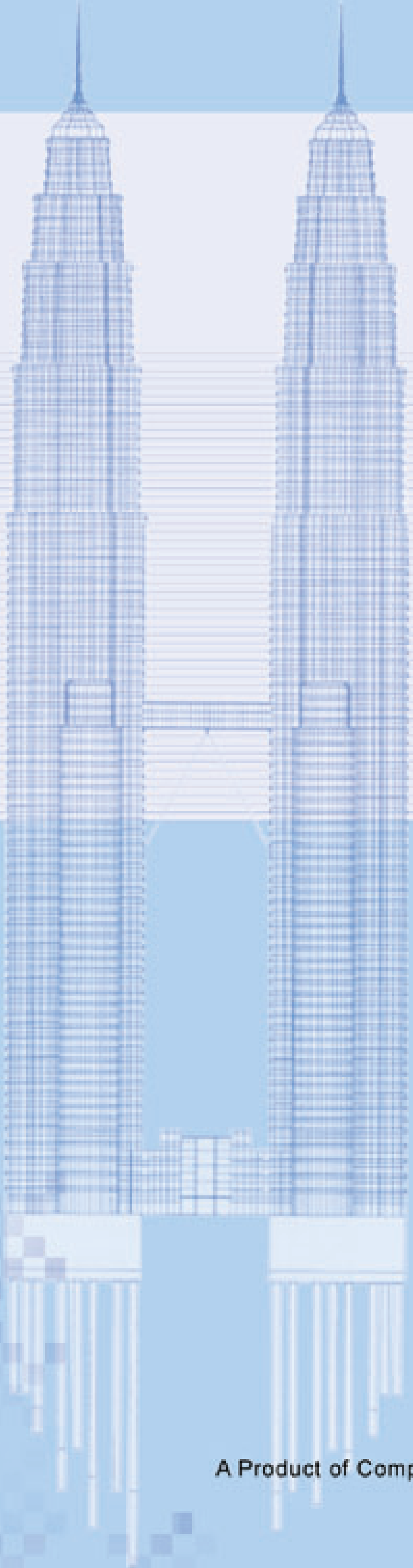

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS

version 8

Integrated Building Design Software

STEEL FRAME DESIGN MANUAL

A Product of Computers & Structures, Inc.

COMPUTERS & STRUCTURES INC.

ETABS®

Integrated Building Design Software

Steel Frame Design Manual

COMPUTERS & STRUCTURES INC.

Computers and Structures, Inc.
Berkeley, California, USA

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THIS PROGRAM IS A VERY PRACTICAL TOOL FOR THE DESIGN/CHECK OF STEEL STRUCTURES. HOWEVER, THE USER MUST THOROUGHLY READ THE MANUAL AND CLEARLY RECOGNIZE THE ASPECTS OF STEEL DESIGN THAT THE PROGRAM ALGORITHMS DO NOT ADDRESS.

THE USER MUST EXPLICITLY UNDERSTAND THE ASSUMPTIONS OF THE PROGRAM AND MUST INDEPENDENTLY VERIFY THE RESULTS.

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Technical Note 1
General Design Information

This Technical Note presents some basic information and concepts that you should know before performing steel frame design using this program.

Design Codes

The design code is set using the **Options menu > Preferences > Steel Frame Design** command. You can choose to design for any one design code in any one design run. You cannot design some elements for one code and others for a different code in the same design run. You can however perform different design runs using different design codes without rerunning the analysis.

Units

For steel frame design in this program, any set of consistent units can be used for input. Typically, design codes are based on one specific set of units. The documentation in this series of Technical Notes is typically presented in kip-inch-seconds units.

Again, any system of units can be used to define and design a building in this program. You can change the system of units that you are using at any time.

Overwriting the Frame Design Procedure for a Steel Frame

The three procedures possible for steel beam design are:

- Steel frame design
- Composite beam design
- No design

By default, steel sections are designed using the steel frame design procedure or the composite beam design procedure. A steel frame element qualifies for the Composite Beam Design procedure if it meets all of the following criteria:

- The line type is Beam; that is, the line object is horizontal.
- The frame element is oriented with its positive local 2-axis in the same direction as the positive global Z-axis (vertical upward).
- The frame element has I-section or channel section properties.

If a steel frame member meets the above criteria for composite beams, it defaults to the composite beam design procedure. Otherwise, it defaults to the steel frame design procedure.

A steel frame element can be switched between the Steel Frame Design, Composite Beam Design (if it qualifies), and the "None" design procedure. Assign a steel frame element the "None" design procedure if you do not want it designed by the Steel Frame Design or the Composite Beam Design post-processor.

Change the default design procedure used for steel frame elements by selecting the beam(s) and clicking **Design menu > Overwrite Frame Design Procedure**. This change is only successful if the design procedure assigned to an element is valid for that element. For example, if you select a steel beam and attempt to change the design procedure to Concrete Frame Design, the program will not allow the change because a steel frame element cannot be changed to a concrete frame element.

Design Load Combinations

The program creates a number of default design load combinations for steel frame design. You can add in your own design load combinations. You can also modify or delete the program default load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. See UBC97-ASD Steel Frame Design Technical Note 8 Design Load Combinations, UBC97-LRFD Steel Frame Design Technical Note 22 Design Load Combinations, AISC-ASD89 Steel Frame Design Technical Note 36 Design Load Combinations and AISC-LRFD93 Steel Frame Design Technical Note 46 Design Load Combinations for more information.

Analysis Sections and Design Sections

Analysis sections are those section properties used for a frame element to analyze the model when you click the **Analyze menu > Run Analysis** command. The design section is whatever section has most currently been designed and thus designated the current design section.

It is possible for the last used analysis section and the current design section to be different. For example, you may have run your analysis using a W18X35 beam and then found in the design that a W16X31 beam worked. In this case, the last used analysis section is the W18X35 and the current design section is the W16X31. Before you complete the design process, verify that the last used analysis section and the current design section are the same using the **Design menu > Steel Frame Design > Verify Analysis vs Design Section** command.

The program keeps track of the analysis section and the design section separately. Note the following about analysis and design sections:

- Assigning a line object a frame section property using the **Assign menu > Frame/Line > Frame Section** command assigns this section as both the analysis section and the design section.

- Running an analysis using the **Analyze menu > Run Analysis** command (or its associated toolbar button) always sets the analysis section to be the same as the current design section.

- Using the **Assign menu > Frame/Line > Frame Section** command to assign an auto select list to a frame section initially sets the analysis and design section to be the section with the median weight in the auto select list.

- Unlocking the model deletes design results, but it does not delete or change the design section.

- Using the **Design menu > Steel Frame Design > Select Design Combo** command to change a design load combination deletes design results, but it does not delete or change the design section.
- Using the **Define menu > Load Combinations** command to change a design load combination deletes your design results, but it does not delete or change the design section.
- Using the **Options menu > Preferences > Steel Frame Design** command to change any of the steel frame design preferences deletes design results, but it does not delete or change the design section.
- Deleting the static nonlinear analysis results also deletes the design results for any load combination that includes static nonlinear forces. Typically, static nonlinear analysis and design results are deleted when one of the following actions is taken:
  - ✓ Use the **Define menu > Frame Nonlinear Hinge Properties** command to redefine existing or define new hinges.
  - ✓ Use the **Define menu > Static Nonlinear/Pushover Cases** command to redefine existing or define new static nonlinear load cases.
  - ✓ Use the **Assign menu > Frame/Line > Frame Nonlinear Hinges** command to add or delete hinges.

Again note that this only deletes results for load combinations that include static nonlinear forces.

Second Order P-Delta Effects

Typically design codes require that second order P-Delta effects be considered when designing steel frames. The P-Delta effects come from two sources. They are the global lateral translation of the frame and the local deformation of elements within the frame.

Consider the frame element shown in Figure 1, which is extracted from a story level of a larger structure. The overall global translation of this frame element is indicated by Δ. The local deformation of the element is shown as δ.

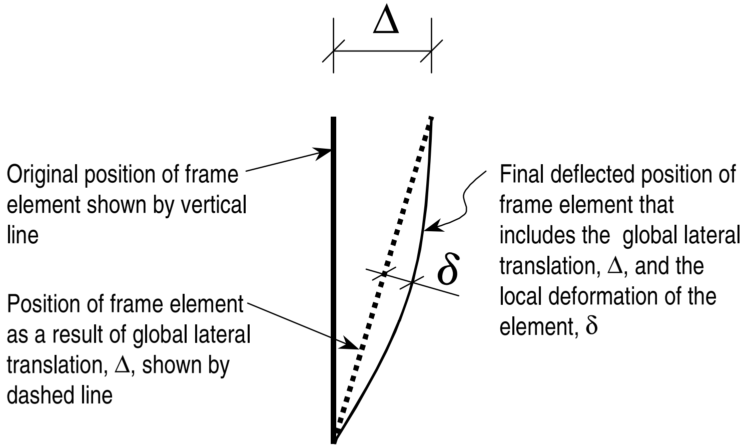

Figure 1 The total Second Order P-Delta Effects on a Frame Element Caused by Both Δ and δ

The total second order P-Delta effects on this frame element are those caused by both Δ and δ.

The program has an option to consider P-Delta effects in the analysis. Controls for considering this effect are found using the **Analyze menu > Set Analysis Options** command and then clicking the Set P-Delta Parameters button. When you consider P-Delta effects in the analysis, the program does a good job of capturing the effect due to the Δ deformation shown in Figure 1, but it does not typically capture the effect of the δ deformation (unless, in the model, the frame element is broken into multiple pieces over its length).

In design codes, consideration of the second order P-Delta effects is generally achieved by computing the flexural design capacity using a formula similar to that shown in Equation. 1.

$$M_{CAP} = aM_{nt} + bM_{lt} \qquad \text{Eqn. 1}$$

where,

M_{CAP} = flexural design capacity

M_{nt} = required flexural capacity of the member assuming there is no translation of the frame (i.e., associated with the δ deformation in Figure 1)

M_{lt} = required flexural capacity of the member as a result of lateral translation of the frame only (i.e., associated with the Δ deformation in Figure 1)

a = unitless factor multiplying M_{nt}

b = unitless factor multiplying M_{lt} (assumed equal to 1 by the program, see below)

When the program performs steel frame design, it assumes that the factor b is equal to 1 and it uses code-specific formulas to calculate the factor a. That b = 1 assumes that you have considered P-Delta effects in the analysis, as previously described. Thus, in general, if you are performing steel frame design in this program, you should consider P-Delta effects in the analysis before running the design.

Element Unsupported Lengths

The column unsupported lengths are required to account for column slenderness effects. The program automatically determines these unsupported lengths. They can also be overwritten by the user on an element-by-element basis, if desired, using the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

There are two unsupported lengths to consider. They are l_{33} and l_{22}, as shown in Figure 2. These are the lengths between support points of the element in the corresponding directions. The length l_{33} corresponds to instability about the 3-3 axis (major axis), and l_{22} corresponds to instability about the 2-2 axis (minor axis). The length l_{22} is also used for lateral-torsional buckling caused by major direction bending (i.e., about the 3-3 axis).

In determining the values for l_{22} and l_{33} of the elements, the program recognizes various aspects of the structure that have an effect on these lengths, such as member connectivity, diaphragm constraints and support points. The program automatically locates the element support points and evaluates the corresponding unsupported length.

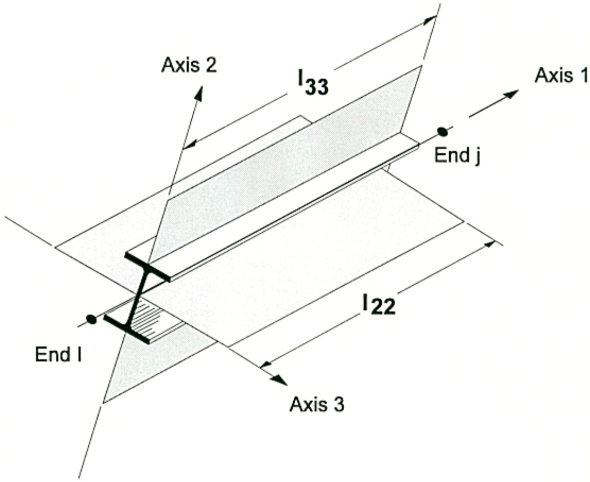

Figure 2 Major and Minor Axes of Bending

It is possible for the unsupported length of a frame element to be evaluated by the program as greater than the corresponding element length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In this case, the column is assumed to be supported in one direction only at that story level, and its unsupported length in the other direction will exceed the story height.

Effective Length Factor (*K*)

The program automatically determines K-factors for frame elements. These K-factors can be overwritten by the user if desired using the **Design menu > Steel Frame Design > View/Revise Overwrites** command. See the bulleted list at the end of this section for some important tips about how the program calculates the K-factors.

The K-factor algorithm has been developed for building-type structures, where the columns are vertical and the beams are horizontal, and the behavior is basically that of a moment-resisting nature for which the K-factor calculation is relatively complex. For the purpose of calculating K-factors, the elements are identified as columns, beams and braces. All elements parallel to the Z-axis are classified as columns. All elements parallel to the X-Y plane are classified as beams. The rest are braces.

The beams and braces are assigned K-factors of unity. In the calculation of the K-factors for a column element, the program first makes the following four stiffness summations for each joint in the structural model:

$$S_{cx} = \sum \left(\frac{E_c I_c}{L_c} \right)_x \qquad S_{bx} = \sum \left(\frac{E_b I_b}{L_b} \right)_x$$

$$S_{cy} = \sum \left(\frac{E_c I_c}{L_c} \right)_y \qquad S_{by} = \sum \left(\frac{E_b I_b}{L_b} \right)_y$$

where the *x* and *y* subscripts correspond to the global *X* and *Y* directions and the *c* and *b* subscripts refer to column and beam. The local 2-2 and 3-3 terms EI_{22}/L_{22} and EI_{33}/L_{33} are rotated to give components along the global *X* and *Y* directions to form the $(EI/L)_x$ and $(EI/L)_y$ values. Then for each column, the joint summations at END-I and the END-J of the member are transformed back to the column local 1-2-3 coordinate system and the *G*-values for END-I and the END-J of the member are calculated about the 2-2 and 3-3 directions as follows:

$$G^I{}_{22} = \frac{S^I{}_{c22}}{S^I{}_{b22}} \qquad G^J{}_{22} = \frac{S^J{}_{c22}}{S^J{}_{b22}}$$

$$G^I{}_{33} = \frac{S^I{}_{c33}}{S^I{}_{b33}} \qquad G^I{}_{33} = \frac{S^I{}_{c33}}{S^I{}_{b33}}$$

If a rotational release exists at a particular end (and direction) of an element, the corresponding value is set to 10.0. If all degrees of freedom for a particular joint are deleted, the *G*-values for all members connecting to that joint will be set to 1.0 for the end of the member connecting to that joint. Finally, if G^I and G^J are known for a particular direction, the column K-factor for the corresponding direction is calculated by solving the following relationship for α:

$$\frac{\alpha^2 G^I G^J - 36}{6(G^I + G^J)} = \frac{\alpha}{\tan \alpha}$$

from which K = π/α. This relationship is the mathematical formulation for the evaluation of K-factors for moment-resisting frames assuming sidesway to be uninhibited. For other structures, such as braced frame structures, the K-factors for all members are usually unity and should be set so by the user. The following are some important aspects associated with the column K-factor algorithm:

- An element that has a pin at the joint under consideration will not enter the stiffness summations calculated above. An element that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated *EI* value. Also, beam elements that have no column member at the far end from the joint under consideration, such as cantilevers, will not enter the stiffness summation.

- If there are no beams framing into a particular direction of a column element, the associated *G*-value will be infinity. If the *G*-value at any one end of a column for a particular direction is infinity, the K-factor corresponding to that direction is set equal to unity.

- If rotational releases exist at both ends of an element for a particular direction, the corresponding K-factor is set to unity.

- The automated K-factor calculation procedure can occasionally generate artificially high K-factors, specifically under circumstances involving skewed beams, fixed support conditions, and under other conditions where the program may have difficulty recognizing that the members are laterally supported and K-factors of unity are to be used.

- All K-factors produced by the program can be overwritten by the user. These values should be reviewed and any unacceptable values should be replaced.

- The beams and braces are assigned K-factors of unity.

Continuity Plates and Doubler Plates

When a beam frames into the flange of a column, continuity plates and doubler plates may be required, as illustrated in Figure 3. The design of these plates is based on the major moment in the beam. If the beam frames into the column flange at an angle, the doubler and continuity plate design is based on a component of the beam major moment, rather than the full beam moment.

The design equations for doubler and continuity plates are described further in the following Technical Notes:

UBC-ASD Steel Frame Design Technical Note 16 Doubler Plates

UBC-LRFD Steel Frame Design Technical Note 30 Doubler Plates

UBC-ASD Steel Frame Design Technical Note 15 Continuity Plates

UBC-LRFD Steel Frame Design Technical Note 29 Continuity Plates

Technical Note 2
Steel Frame Design Process

This Technical Note describes a basic steel frame design process using this program. Although the *exact* steps you follow may vary, the *basic* design process should be similar to that described herein. The other Technical Notes in the Steel Frame Design series provide additional information.

Steel Frame Design Procedure

The following sequence describes a typical steel frame design process for a new building. Note that although the sequence of steps you follow may vary, the basic process probably will be essentially the same.

1. Use the **Options menu > Preferences > Steel Frame Design** command to choose the steel frame design code and to review other steel frame design preferences and revise them if necessary. Note that default values are provided for all steel frame design preferences, so it is unnecessary to define any preferences unless you want to change some of the default values. See UBC97-ASD Steel Frame Design Technical Note 6 Preferences, UBC97-LRFD Steel Frame Design Technical Note 20 Preferences, AISC-ASD89 Steel Frame Design Technical Note 34 Preferences, and AISC-LRFD93 Steel Frame Design Technical Note 44 Preferencesfor more information.

2. Create the building model.

3. Run the building analysis using the **Analyze menu > Run Analysis** command.

4. Assign steel frame overwrites, if needed, using the **Design menu > Steel Frame Design > View/Revise Overwrites** command. Note that you must select frame elements first using this command. Also note that default values are provided for all steel frame design overwrites so it is unnecessary to define overwrites unless you want to change some of the default values. Note that the overwrites can be assigned before or after

the analysis is run. See UBC97-ASD Steel Frame Design Technical Note 7 Overwrites, UBC97-LRFD Steel Frame Design Technical Note 21 Overwrites, AISC-ASD89 Steel Frame Design Technical Note 35 Overwrites, and AISC-LRFD93 Steel Frame Design Technical Note 45 Overwrites for more information.

5. Designate design groups, if desired, using the **Design menu > Steel Frame Design > Select Design Group** command. Note that you must have already created some groups by selecting objects and clicking the **Assign menu > Group Names** command.

6. To use design load combinations other than the defaults created by the program for your steel frame design, click the **Design menu > Steel Frame Design > Select Design Combo** command. Note that you must have already created your own design combos by clicking the **Define menu > Load Combinations** command. See UBC97-ASD Steel Frame Design Technical Note 8 Design Load Combinations, UBC97-LRFD Steel Frame Design Technical Note 22 Design Load Combinations, AISC-ASD89 Steel Frame Design Technical Note 36 Design Load Combinations, and AISC-LRFD93 Steel Frame Design Technical Note 46 Design Load Combinations for more information.

7. Designate lateral displacement targets for various load cases using the **Design menu > Steel Frame Design > Set Lateral Displacement Targets** command.

8. Click the **Design menu > Steel Frame Design > Start Design/Check of Structure** command to run the steel frame design.

9. Review the steel frame design results by doing one of the following:

   a. Click the **Design menu > Steel Frame Design > Display Design Info** command to display design input and output information on the model. See Steel Frame Design Technical Note 4 Output Data Plotted Directly on the Model.

   b. Right click on a frame element while the design results are displayed on it to enter the interactive design mode and interactively design the frame element. Note that while you are in this mode, you can revise overwrites and immediately see the results of the new design.

If design results are not currently displayed (and the design has been run), click the **Design menu > Steel Frame Design > Interactive Steel Frame Design** command and right click a frame element to enter the interactive design mode for that element. See Steel Frame Design Technical Note 3 Interactive Steel Frame Design for more information.

c. Use the **File menu > Print Tables > Steel Frame Design** command to print steel frame design data. If you select frame elements before using this command, data is printed only for the selected elements. See UBC97-ASD Steel Frame Design Technical Note 17 Input Data, UBC97-LRFD Steel Frame Design Technical Note 31 Input Data, AISC-ASD89 Steel Frame Design Technical Note 41 Input Data, and AISC-LRFD93 Steel Frame Design Technical Note 51 Input Data, and UBC97-ASD Steel Frame Design Technical Note 18 Output Details, UBC97-LRFD Steel Frame Design Technical Note 32 Output Details, AISC-ASD89 Steel Frame Design Technical Note 42 Output Details, and AISC-LRFD93 Steel Frame Design Technical Note 52 Output Details for more information.

10. Use the **Design menu > Steel Frame Design > Change Design Section** command to change the design section properties for selected frame elements.

11. Click the **Design menu > Steel Frame Design > Start Design/Check of Structure** command to rerun the steel frame design with the new section properties. Review the results using the procedures described above.

12. Rerun the building analysis using the **Analyze menu > Run Analysis** command. Note that the section properties used for the analysis are the last specified design section properties.

13. Compare your lateral displacements with your lateral displacement targets.

14. Click the **Design menu > Steel Frame Design > Start Design/Check of Structure** command to rerun the steel frame design with the new analysis results and new section properties. Review the results using the procedures described in Item 9.

Note:

Steel frame design in this program is an iterative process. Typically, the analysis and design will be rerun multiple times to complete a design.

15. Again use the **Design menu > Steel Frame Design > Change Design Section** command to change the design section properties for selected frame elements, if necessary.

16. Repeat the processes in steps 12, 13, 14 and 15 as many times as necessary.

17. Select all frame elements and click the **Design menu > Steel Frame Design > Make Auto Select Section Null** command. This removes any auto select section assignments from the selected frame elements (if they have the Steel Frame design procedure).

18. Rerun the building analysis using the **Analyze menu > Run Analysis** command. Note that the section properties used for the analysis are the last specified design section properties.

19. Verify that your lateral displacements are within acceptable limits.

20. Click the **Design menu > Steel Frame Design > Start Design/Check of Structure** command to rerun the steel frame design with the new section properties. Review the results using the procedures described in step 9.

21. Click the **Design menu > Steel Frame Design > Verify Analysis vs Design Section** command to verify that all of the final design sections are the same as the last used analysis sections.

22. Use the **File menu > Print Tables > Steel Frame Design** command to print selected steel frame design results if desired. See UBC97-ASD Steel Frame Design Technical Note 18 Output Details, UBC97-LRFD Steel Frame Design Technical Note 32 Output Details, AISC-ASD89 Steel Frame Design Technical Note 42 Output Details, and AISC-LRFD93 Steel Frame Design Technical Note 52 Output Details for more information.

It is important to note that design is an iterative process. The sections used in the original analysis are not typically the same as those obtained at the end of the design process. Always run the building analysis using the final frame section sizes and then run a design check using the forces obtained from that analysis. Use the **Design menu > Steel Frame Design > Verify Analysis vs Design Section** command to verify that the design sections are the same as the analysis sections.

Automating the Iterative Design Process

If frame elements have been assigned as auto select sections, the program can automatically perform the iterative steel frame design process. To initiate this process, first use the **Options menu > Preferences > Steel Frame Design** command and set the Maximum Auto Iterations item to the maximum number of design iterations you want the program to run automatically. Next run the analysis. Then, *making sure that no elements are selected,* use the **Design menu > Steel Frame Design > Start Design/Check of Structure** command to begin the design of the structure. The program will then start a cycle of (1) performing the design, (2) comparing the last-used Analysis Sections with the Design Sections, (3) setting the Analysis Sections equal to the Design Sections, and (4) rerunning the analysis. This cycle will continue until one of the following conditions has been met:

- the Design Sections and the last-used Analysis Sections are the same
- the number of iterations performed is equal to the number of iterations you specified for the Maximum Auto Iterations item on the Preferences form

If the maximum number of iterations is reached before the Design Sections and Analysis Sections match, the program will report any differences on screen.

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STEEL FRAME DESIGN

Technical Note 3

Interactive Steel Frame Design

General

Interactive steel frame design allows you to review the design results for any frame element and to interactively change the design overwrites and immediately review the results.

Note that a design must have been run for the interactive design mode to be available. To run a design, click **Design menu > Steel Frame Design > Start Design/Check of Structure** command.

Right click on a frame element while the design results are displayed on it to enter the interactive design mode and interactively design the element. If design results are not currently displayed (and the design has been run), click the **Design menu > Steel Frame Design > Interactive Steel Frame Design** command and then right click a frame element to enter the interactive design mode for that element and display the Steel Stress Check Information form.

Steel Stress Check Information Form

Table 1 identifies the features that are included in the Steel Stress Check Information form.

Table 1 Steel Stress Check Information Form

FEATURE	DESCRIPTION
Story ID	This is the story level ID associated with the frame element.
Beam	This is the label associated with a frame element that is a beam.
Column	This is the label associated with a frame element that is a column.

Table 1 Steel Stress Check Information Form

FEATURE	DESCRIPTION
Brace	This is the label associated with a frame element that is a brace.
Tip: The section property displayed for the Design Section item is used by the program as the section property for the next analysis run.	
Analysis section	This is the section property that was used for this frame element in the last analysis. Thus, the design forces are based on a frame element of this section property. For your final design iteration, the Design Section and the last-used Analysis Section should be the same.
Design section	This is the current design section property. If the frame element is assigned an auto select list, the section displayed in this form initially defaults to the optimal section. If no auto select list has been assigned to the frame element, the element design is performed for the section property specified in this edit box. It is important to note that subsequent analyses use the section property specified in this list box for the next analysis section for the frame element. Thus, the forces and moments obtained in the next analysis will be based on this section. To change the Design Section, click the **Overwrites** button.
Stress Details Table	
The stress details table shows the stress ratios obtained for each design load combination at each output station along the frame element. Initially the worst stress ratio is highlighted. Following are the headings in the table:	
Combo ID	This is the name of the design load combination considered.
Station location	This is the location of the station considered, measured from the i-end of the frame element.

Table 1 Steel Stress Check Information Form

FEATURE	DESCRIPTION
Moment Interaction Checks	
Ratio	This is the total PMM stress ratio for the element. When stress ratios are reported for this item, they are followed by either (T) or (C). The (T) item indicates that the axial component of the stress ratio is tension. The (C) item indicates that the axial component of the stress ratio is compression. Note that typically the interaction formulas are different, depending on whether the axial stress is tension or compression.
Axl	This is the axial component of the PMM stress ratio.
B-Maj	This is the bending component of the PMM stress ratio for bending about the major axis.
B-Min	This is the bending component of the PMM stress ratio for bending about the minor axis.
Maj Shr Ratio	This is the shear stress ratio for shear acting in the major direction of the frame element.
Min Shr Ratio	This is the shear stress ratio for shear acting in the minor direction of the frame element.

Overwrites Button

Click this button to access and make revisions to the steel frame overwrites and then immediately see the new design results. If you modify some overwrites in this mode and exit both the Steel Frame Design Overwrites form and the Steel Stress Check Information form by clicking their respective OK buttons, the changes made to the overwrites are saved permanently.

Exiting the Steel Frame Design Overwrites form by clicking the **OK** button temporarily saves changes. Subsequently exiting the Steel Stress Check Information form by clicking the **Cancel** button, cancels the changes made. Permanent saving of the overwrites does not occur until you click the **OK** button in the Steel Stress Check Information form as well as the Steel Frame Design Overwrites form.

Details Button

Clicking this button displays design details for the frame elements. Print this information by selecting Print from the File menu that appears at the top of the window displaying the design details.

Technical Note 4
Output Data Plotted Directly on the Model

This Technical Note describes the input and output data that can be plotted directly on the model.

Overview

Use the **Design menu > Steel Frame Design > Display Design Info** command to display on-screen output plotted directly on the program model. If desired, the screen graphics can then be printed using the **File menu > Print Graphics** command. The on-screen display data provides design input and output data.

Design Input

Table 1 identifies the types of data that can be displayed directly on the model by selecting the data type (shown in bold type) from the drop-down list on the Display Design Results form. Display this form by selecting the **Design menu > Steel Frame Design > Display Design Info** command.

Table 1 Data Displayed Directly on the Model

DATA TYPE	DESCRIPTION
Design Sections	The current design section property.
Design Type	Steel, concrete or other. In this section, steel would be selected.
Live Load Red Factors	These reduction factors are used by the program to automatically reduce the live load in the design post-processor. They are set using the **Options menu > Preferences** command.
Unbraced L Ratios	Ratio of unbraced length divided by total length.

Table 1 Data Displayed Directly on the Model

DATA TYPE	DESCRIPTION
Effective Length K-Factors	As defined in AISC-ASD Table C-C2.1 or AISC-LRFD Table C-C2.1.
Axial Allowables	
Bending Allowables	
Shear Allowables	

Note that you *cannot* simultaneously display multiple listed items on the model.

Design Output

Table 2 identifies the types of data that can be displayed directly on the model *after the model has been run* by selecting the data type (shown in bold type) from the drop-down list on the Display Design Results form. Display this form by selecting he **Design menu > Steel Frame Design > Display Design Info** command.

Table 2 Data Available After a Model Has Been Run

DATA TYPE	DESCRIPTION
PM Ratio Colors & Values	Colors indicating stress ranges for ratio of acting axial and bending stresses or forces divided by the allowable numerical values.
PM Colors/Shear Ratio Values	Colors indicating axial and bending ratio, and numerical values indicating shear stress ratio.
PM Ratio Color/no Values	Colors indicating axial and bending ratio only.

To display color-coded P-M interaction ratios with values, use the **Design menu > Steel Frame Design > Display Design Info** command. Click the Design Output check box on the Display Design Results form. Note that a design must have been run for the output selection to be available. Select P-M Ratios Colors & Values from the drop-down box. Click the **OK** button and your selection will display on the model in the active window. Access the other two display options in the same manner.

Note that you *cannot* simultaneously display multiple listed items on the model.

Technical Note 5
General and Notation

Introduction to the UBC97-ASD Series of Technical Notes

The UBC97-ASD design code in this program implements the International Conference of Building Officials' *1997 Uniform Building Code: Volume 2: Structural Engineering Design Provisions,* Chapter 22, Division III, "Design Standard for Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design" (ICBO 1997).

For referring to pertinent sections and equations of the UBC code, a unique prefix "UBC" is assigned. For referring to pertinent sections and equations of the AISC-ASD code, a unique prefix "ASD" is assigned. However, all references to the "Specifications for Allowable Stress Design of Single-Angle Members" (AISC 1989b) carry the prefix of "ASD SAM." Various notations used in the Steel Frame Design UBC97-ASD series of Technical Notes are described herein.

When using the UBC97-ASD option, the following Framing Systems are recognized (UBC 1627, 2213):

- Ordinary Moment Frame (OMF)
- Special Moment-Resisting Frame (SMRF)
- Concentrically Braced Frame (CBF)
- Eccentrically Braced Frame (EBF)
- Special Concentrically Braced Frame (SCBF)

By default the frame type is taken as Special-Moment Resisting (SMRF) in the program. However, the frame type can be overwritten in the Preferences (**Options menu > Preferences > Steel Frame Design**) to change the default values and in the Overwrites (**Design menu > Steel Frame Design >**

View/Revise Overwrites) on a member-by-member basis. If any member is assigned with a frame type, the change of the frame type in the Preference will not modify the frame type of the individual member for which it is assigned.

When using the UBC97-LRFD option, a frame is assigned to one of the following five Seismic Zones (UBC 2213, 2214):

- Zone 0
- Zone 1
- Zone 2
- Zone 3
- Zone 4

By default the Seismic Zone is taken as Zone 4 in the program. However, the frame type can be overwritten in the Preferences to change the default (**Options menu > Preferences > Steel Frame Design**).

The design is based on user-specified loading combinations. To facilitate use, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures. See UCB-ASD Steel Frame Design Technical Note 8 Design Load Combinations for more information.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this series of Technical Notes. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates overstress. Similarly, a shear capacity ratio is also calculated separately. Algorithms for completing these calculations are described in UBC97-ASD Steel Frame Design Technical Notes 10 Calculation of Stresses, 11 Calculation of Allowable Stresses, and 12 Calculation of Stress Ratios.

Further information is available from UBC97-ASD Steel Frame Design Technical Notes 9 Classification of Sections, 14 Joint Design, 15 Continuity Plates, and 16 Doubler Plates.

Information about seismic requirements is provided in UBC97-ASD Steel Frame Design Technical Note 13 Seismic Requirements.

The program uses preferences and overwrites, which are described in UBC97-ASD Steel Frame Design Technical Notes 6 Preferences and 7 Overwrites. It also provides input and output data summaries, which are described in UBC97-ASD Steel Frame Design Technical Notes 17 Input Data and 18 Output Details.

English as well as SI and MKS metric units can be used for input. But the code is based on Kip-Inch-Second units. For simplicity, all equations and descriptions presented in this series of Technical Notes correspond to Kip-Inch-Second units unless otherwise noted.

Notations

A	Cross-sectional area, in^2
A_e	Effective cross-sectional area for slender sections, in^2
A_f	Area of flange, in^2
A_g	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	Major and minor shear areas, in^2
A_w	Web shear area, dt_w, in^2
C_b	Bending Coefficient
C_m	Moment Coefficient
C_w	Warping constant, in^6
D	Outside diameter of pipes, in
E	Modulus of elasticity, ksi

F_a Allowable axial stress, ksi

F_b Allowable bending stress, ksi

F_{b33}, F_{b22} Allowable major and minor bending stresses, ksi

F_{cr} Critical compressive stress, ksi

F'_{e33} $\dfrac{12\pi^2 E}{23(K_{33}l_{33} / r_{33})^2}$

F'_{e22} $\dfrac{12\pi^2 E}{23(K_{22}l_{22} / r_{22})^2}$

F_v Allowable shear stress, ksi

F_y Yield stress of material, ksi

K Effective length factor

K_{33}, K_{22} Effective length K-factors in the major and minor directions

M_{33}, M_{22} Major and minor bending moments in member, kip-in

M_{ob} Lateral-torsional moment for angle sections, kin-in

P Axial force in member, kips

P_e Euler buckling load, kips

Q Reduction factor for slender section, = Q_aQ_s

Q_a Reduction factor for stiffened slender elements

Q_s Reduction factor for unstiffened slender elements

S Section modulus, in^3

S_{33}, S_{22} Major and minor section moduli, in^3

$S_{eff,33}$,$S_{eff,22}$ Effective major and minor section moduli for slender sections, in^3

S_c	Section modulus for compression in an angle section, in^3
V_2, V_3	Shear forces in major and minor directions, kips
b	Nominal dimension of plate in a section, in longer leg of angle sections, b_f — $2t_w$ for welded and b_f — $3t_w$ for rolled box sections, etc.
b_e	Effective width of flange, in
b_f	Flange width, in
d	Overall depth of member, in
f_a	Axial stress, either in compression or in tension, ksi
f_b	Normal stress in bending, ksi
f_{b33}, f_{b22}	Normal stress in major and minor direction bending, ksi
f_v	Shear stress, ksi
f_{v2}, f_{v3}	Shear stress in major and minor direction bending, ksi
h	Clear distance between flanges for I shaped sections ($d - 2t_f$), in
h_e	Effective distance between flanges, less fillets, in
k	Distance from outer face of flange to web toes of fillet, in
k_c	Parameter used for classification of sections, $\frac{4.05}{[h/t_w]^{0.46}}$ *if* $h/t_w > 70$, 1 *if* $h/t_w \le 70$
l_{33}, l_{22}	Major and minor direction unbraced member length, in
l_c	Critical length, in
r	Radius of gyration, in

r_{33}, r_{22} Radii of gyration in the major and minor directions, in

r_z Minimum radius of gyration for angles, in

t Thickness of a plate in I, box, channel, angle, and T sections, in

t_f Flange thickness, in

t_w Web thickness, in

β_w Special section property for angles, in

References

American Institute of Steel Construction (AISC). 1989a. *Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design, June 1, 1989 with Commentary,* 2nd Impression. Chicago, Illinois.

American Institute of Steel Construction (AISC). 1989b. *Manual of Steel Construction, Allowable Stress Design, 9th Edition.* Chicago, Illinois.

International Conference of Building Officials (ICBO). 1997. 1997 *Uniform Building Code, Volume 2, Structural Engineering Design Provisions.* Whittier, California.

STEEL FRAME DESIGN UBC97-ASD

Technical Note 6
Preferences

This Technical Note describes the items in the Preferences form.

General

The steel frame design preferences in this program are basic assignments that apply to all steel frame elements. Use the **Options menu > Preferences > Steel Frame Design** command to access the Preferences form where you can view and revise the steel frame design preferences.

Default values are provided for all steel frame design preference items. Thus, it is not required that you specify or change any of the preferences. You should, however, at least review the default values for the preference items to make sure they are acceptable to you.

Using the Preferences Form

To view preferences, select the **Options menu > Preferences > Steel Frame Design.** The Preferences form will display. The preference options are displayed in a two-column spreadsheet. The left column of the spreadsheet displays the preference item name. The right column of the spreadsheet displays the preference item value.

To change a preference item, left click the desired preference item in either the left or right column of the spreadsheet. This activates a drop-down box or highlights the current preference value. If the drop-down box appears, select a new value. If the cell is highlighted, type in the desired value. The preference value will update accordingly. You cannot overwrite values in the drop-down boxes.

When you have finished making changes to the composite beam preferences, click the **OK** button to close the form. You must click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the preferences are ignored and the form is closed.

Preferences

For purposes of explanation in this Technical Note, the preference items are presented in Table 1. The column headings in the table are described as follows:

- **Item:** The name of the preference item as it appears in the cells at the left side of the Preferences form.
- **Possible Values:** The possible values that the associated preference item can have.
- **Default Value:** The built-in default value that ETABS assumes for the associated preference item.
- **Description:** A description of the associated preference item.

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	AISC-ASD89	Design code used for design of steel frame elements.
Time History Design	Envelopes, Step-by-Step	Envelopes	Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has *more than one* time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.
Frame Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, EBF	Ordinary MRF	

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Zone	Zone 0, Zone 1, Zone 2, Zone 3, Zone 4	Zone 4	Seismic zone
Omega 0	≥0	2.8	
Stress Ratio Limit	>0	.95	Program will select members from the auto select list with stress ratios less than or equal to this value.
Maximum Auto Iteration	≥1	1	Sets the number of iterations of the analysis-design cycle that the program will complete automatically assuming that the frame elements have been assigned as auto select sections.

STEEL FRAME DESIGN UBC97-ASD

Technical Note 7
Overwrites

General

The steel frame design overwrites are basic assignments that apply only to those elements to which they are assigned. This Technical Note describes steel frame design overwrites for UBC97-ASD. To access the overwrites, select an element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

Default values are provided for all overwrite items. Thus, you do not need to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned; that is, to the elements that are selected when the overwrites are changed.

Overwrites

For explanation purposes in this Technical Note, the overwrites are presented in Table 1. The column headings in the table are described as follows.

- **Item:** The name of the overwrite item as it appears in the program. To save space in the forms, these names are generally short.

- **Possible Values:** The possible values that the associated overwrite item can have.

- **Default Value:** The default value that the program assumes for the associated overwrite item. If the default value is given in the table with an associated note "Program Calculated," the value is shown by the program before the design is performed. After design, the values are calculated by the program and the default is modified by the program-calculated value.

- **Description:** A description of the associated overwrite item.

An explanation of how to change an overwrite is provided at the end of this Technical Note.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Current Design Section			Indicates selected member size used in current design.
Element Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, EBF	From Preferences	
Live Load Reduction Factor	≥0	1 (Program Calculated)	Live load is multiplied by this factor.
Horizontal Earthquake Factor	≥0	1	Earthquake loads are multiplied by this factor.
Unbraced Length Ratio (Major)	≥0	1 (Program Calculated)	Ratio of unbraced length divided by total length.
Unbraced Length Ratio (Minor, LTB)	≥0	1 (Program Calculated)	Ratio of unbraced length divided by total length.
Effective Length Factor (K Major)	≥0	1 (Program Calculated for Columns)	As defined in AISC-ASD Table C-C2.1, page 5-135.
Effective Length Factor (K Minor)	≥0	1 (Program Calculated for Columns)	As defined in AISC-ASD Table C-C2.1, page 5-135.
Moment Coefficient (Cm Major)	≥0	0.85 (Program Calculated)	As defined in AISC-ASD, page 5-55.
Moment Coefficient (Cm Minor)	≥0	0.85 (Program Calculated)	As defined in AISC-ASD, page 5-55.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Bending Coefficient (Cb)	≥0	1 (Program Calculated)	As defined in AISC-ASD, page 5-47.
Yield stress, Fy	≥0	0	If zero, yield stress defined for material property data used.
Omega0	≥0	From Preferences	Seismic force amplification factor as required by the UBC.
Compressive stress, Fa	≥0	0	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Tensile stress, Ft	≥0	0	If zero, as defined for material property data used and AISC-ASD Chapter D.
Major Bending stress, Fb3	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Minor Bending stress, Fb2	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Major Shear stress, Fv2	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Minor Shear stress, Fv3	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.

Making Changes in the Overwrites Form

To access the steel frame overwrites, select a frame element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

The overwrites are displayed in the form with a column of check boxes and a two-column spreadsheet. The left column of the spreadsheet contains the name of the overwrite item. The right column of the spreadsheet contains the overwrites values.

Initially, the check boxes in the Steel Frame Design Overwrites form are all unchecked and all of the cells in the spreadsheet have a gray background to indicate that they are inactive and the items in the cells cannot be changed. The names of the overwrite items are displayed in the first column of the spreadsheet. The values of the overwrite items are visible in the second column of the spreadsheet if only one frame element was selected before the overwrites form was accessed. If multiple elements were selected, no values show for the overwrite items in the second column of the spreadsheet.

After selecting one or multiple elements, check the box to the left of an overwrite item to change it. Then left click in either column of the spreadsheet to activate a drop-down box or highlight the contents in the cell in the right column of the spreadsheet. If the drop-down box appears, select a value from the box. If the cell is highlighted, type in the desired value. The overwrite will reflect the change. You cannot change the values of the drop-down boxes.

When changes to the overwrites have been completed, click the **OK** button to close the form. The program then changes all of the overwrite items whose associated check boxes are checked for the selected members. You *must* click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the overwrites are ignored and the form is closed.

Resetting Steel Frame Overwrites to Default Values

Use the **Design menu > Steel Frame Design > Reset All Overwrites** command to reset all of the steel frame overwrites. All current design results will be deleted when this command is executed.

Important note about resetting overwrites: The program defaults for the overwrite items are built into the program. The steel frame overwrite values that were in a .edb file that you used to initialize your model may be different from the built-in program default values. When you reset overwrites, the program resets the overwrite values to its built-in values, not to the values that were in the .edb file used to initialize the model.

Technical Note 8
Design Load Combinations

The design load combinations are the various combinations of the load cases for which the structural members and joints need to be designed or checked. For the UBC97-ASD code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL) and considering that wind and earthquake forces are reversible, the following load combinations may need to be defined (UBC 1612.3):

DL	(UBC 1612.3.1 12-7)
DL + LL	(UBC 1612.3.1 12-8)
DL $\pm$ WL	(UBC 1612.3.1 12-9)
DL + 0.75LL $\pm$ 0.75 WL	(UBC 1612.3.1 12-11)
DL $\pm$ EL/1.4	(UBC 1612.3.1 12-9)
0.9 DL $\pm$ EL/1.4	(UBC 1612.3.1 12-10)
DL + 0.75 LL $\pm$ 0.75 EL/1.4	(UBC 1612.3.1 12-11)

These are also the default design load combinations in the program whenever the UBC97-ASD code is used. The user should use other appropriate load combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

When designing for combinations involving earthquake and wind loads, allowable stresses are **NOT** increased by a factor of 4/3 of the regular allowable value (UBC 1612.3.1, 2209.3).

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading. See UBC97-ASD Steel Frame Design Technical Note 7 Overwrites for more information.

It is noted here that whenever **special seismic loading combinations** are required by the code for special circumstances, the program automatically generates those load combinations internally. The following additional seismic load combinations are frequently checked for specific types of members and special circumstances.

$1.0\ DL + 0.7\ LL \pm \Omega_o\ EL$ (UBC 2213.5.1.1.)

$0.85\ DL \pm \Omega_o\ EL$ (UBC 2213.5.1.2)

where Ω_o is the seismic force amplification factor, which is required to account for structural overstrength. The default value of Ω_o is taken as 2.8 in the program. However, Ω_o can be overwritten in the Preferences to change the default and in the Overwrites on a member-by-member basis. If any member is assigned a value for Ω_o, the change of Ω_o in the Preferences will not modify the Ω_o of the individual member for which Ω_o is assigned, unless the member had been selected. The guidelines for selecting a reasonable value can be found in UBC 1630.3.1 and UCB Table 16-N. Other similar special design load combinations described in UBC97-ASD Steel Frame Design Technical Notes 13 Seismic Requirements and 14 Joint Design.

Those special seismic load combinations are internal to program. The user does **NOT** need to create additional load combinations for those load combinations. The special circumstances for which the load combinations are additionally checked are described as appropriate in the other Technical Notes. It is assumed that any required scaling (such as may be required to scale response spectra results) has already been applied to the program load cases.

STEEL FRAME DESIGN UBC97-ASD

Technical Note 9
Classification of Sections

This Technical Note explains the classification of sections when the user selects the UBC97-ASD design code.

Overview

The allowable stresses for axial compression and flexure depend on the classification of sections. The sections are classified in UBC97-ASD as either Compact, Noncompact, Slender or Too Slender in the same way as described in AISC-ASD89 Steel Frame Design Technical Note 37 Classification of Sections. The program classifies the individual members according to the limiting width/thickness ratios given in Table 1 of AISC-ASD89 Steel Frame Design Technical Note 37 Classification of Sections (UBC 2208, 2212, 2213, ASD B5.1, F3.1, F5, G1, A-B5-2). The definition of the section properties required in this table is given in Figure 1 of AISC-ASD89 Steel Frame Design Technical Note 37 Classification of Sections and AISC-ASD89 Steel Frame Design Technical Note 33 General and Notation.

In general the design sections need not necessarily be Compact to satisfy UBC97-ASD codes (UBC 2213.4.2). However, for certain special seismic cases they must be Compact and must satisfy special slenderness requirements. See UBC97-ASD Steel Frame Design Technical Note 13 Seismic Requirements. The sections that do satisfy these additional requirements are classified and reported as "SEISMIC" in the program. These special requirements for classifying the sections as "SEISMIC" in the program ("Compact" in UBC) are given in Table 1 (UBC 2213.7.3, 2213.8.2.5, 2213.9.24, 2213.9.5, 2212.10.2). If these criteria are not satisfied when the code requires them to be satisfied, the user must modify the section property. In that case, the program gives a warning message in the output file.

Table 1 Limiting Width-Thickness Ratios for Classification of Sections When Special Seismic Conditions Apply in accordance with UBC97-ASD

Description of Section	Width-Thickness Ratio λ	SEISMIC (Special requirements in seismic design) (λ_p)	Section References
I-SHAPE	$b_f / 2t_f$ (beam)	$\leq 52 / \sqrt{F_y}$	UBC 2213.7.3 (SMRF) UBC 2213.10.2 (EBF)
	$b_f / 2t_f$ (column)	8.5 for $F_y \leq 36$ 8.0 for $36 \leq F_y \leq 42$ 7.4 for $42 \leq F_y \leq 45$ 7.0 for $45 \leq F_y \leq 50$ 6.6 for $50 \leq F_y \leq 55$ 6.3 for $55 \leq F_y \leq 60$ 6.0 for $F_y > 60$	UBC2213.7.3 (SMRF) UBC 2213.9.5 (SCBF) ASD N7
BOX	b / t_f and h_c / t_w (column)	$\leq 110 / \sqrt{F_y}$	UBC 2213.7.3 (SMRF) UBC 2213.9.5 (SCBF)
	b / t_f and h_c / t_w (brace)	$\leq 110 / \sqrt{F_y}$	UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)
ANGLE	b / t (brace)	$\leq 52 / \sqrt{F_y}$	UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)
DOUBLE-ANGLE	b / t (brace)	$\leq 52 / \sqrt{F_y}$	UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)
PIPE	D / t (brace)	$\leq 1{,}300 / \sqrt{F_y}$	UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)
CHANNEL	b_f / t_f h_c / t_w	No special requirement No special requirement	
T-SHAPE	$b_f / 2t_f$ d / t_w	No special requirement No special requirement	
ROUND BAR	—	No special requirement	
RECTANGULAR	—	No special requirement	
GENERAL	—	No special requirement	

Technical Note 10
Calculation of Stresses

The axial, flexural, and shear stresses at each of the previously defined stations for each load combination in UBC97-ASD are calculated in the same way as described in AISC-ASD89 Steel Frame Design Technical Note 38 Calculation of Stresses without any exception (UBC 2208, ASD A-B5.2d). For non-slender sections, the stresses are based on the gross cross-sectional areas (ASD A-B5.2c); for slender sections, the stresses are based on effective section properties (ASD A-B5.2c); and for Single-Angle sections, the stresses are based on the principal properties of the sections (ASD SAM 6.1.5).

The flexural stresses are calculated based on the properties about the principal axes. For I, Box, Channel, T, Double-angle, Pipe, Circular and Rectangular sections, the principal axes coincide with the geometric axes. For Single-angle sections, the design considers the principal properties. For general sections, it is assumed that all section properties are given in terms of the principal directions.

For Single-angle sections, the shear stresses are calculated for directions along the geometric axes. For all other sections, the program calculates the shear stresses along the geometric and principle axes.

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Technical Note 11
Calculation of Allowable Stresses

The allowable stress in compression, tension, bending, and shear for Compact, Noncompact, and Slender sections in accordance with UBC97-ASD are calculated in the same way as described in the AISC-ASD89 Steel Frame Design Technical Note 39 Calculation of Allowable Stresses without any exceptions (UBC 2208, ASD A-B5.2d). The allowable stresses for Seismic sections are calculated in the same way as for Compact sections.

The allowable flexural stresses for all shapes of sections are calculated based on their principal axes of bending. For the I, Box, Channel, Circular, Pipe, T, Double-angle and Rectangular sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations related to flexural stresses are based on that.

The allowable shear stress is calculated along geometric axes for all sections. For I, Box, Channel, T, Double-Angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-Angle sections, principal axes do not coincide with the geometric axis.

All limitations and warnings related to allowable stress calculations in AISC-ASD89 Steel Frame Design Technical Note 39 Calculation of Allowable Stresses also apply when the user selects this code in the program.

If the user specifies nonzero allowable stresses for one or more elements in the Steel Frame Overwrites dialog box (display using the **Design menu > Steel Frame Design > Review/Revise Overwrites** command), the nonzero values **will be used rather than the calculated values for those elements.** The specified allowable stresses should be based on the principal axes of bending.

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Technical Note 12
Calculation of Stress Ratios

This Technical Note explains that the stress ratios in UBC97-ASD are calculated in the same way as described in AISC-ASD89 Steel Frame Design Technical Note 40 Calculation of Stress Ratios, with some modifications as described herein.

In the calculation of the axial and bending stress ratios, first, for each station along the length of the member, the actual stresses are calculated for each load combination. Then the corresponding allowable stresses are calculated. Then, the stress ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling stress ratio is then obtained, along with the associated station and load combination. A stress ratio greater than 1.0 indicates an overstress. Similarly, a shear capacity ratio is also calculated separately.

During the design, the effect of the presence of bolts or welds is not considered.

Axial and Bending Stresses

With the computed allowable axial and bending stress values and the factored axial and bending member stresses at each station, an interaction stress ratio is produced for each of the load combinations as follows (ASD H1, H2, SAM 6):

- If f_a is compressive and f_a / F_a, > 0.15, the combined stress ratio is given by the larger of

$$\frac{f_a}{F_a} + \frac{C_{m33} f_{b33}}{\left(1 - \frac{f_a}{F'_{e33}}\right) F_{b33}} + \frac{C_{m22} f_{b22}}{\left(1 - \frac{f_a}{F'_{e22}}\right)}, \text{ and} \qquad \text{(ASD H1-1, SAM 6.1)}$$

$$\frac{f_a}{Q(0.60F_y)} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}\ ,\quad \text{where} \qquad \text{(ASD H1-2, SAM 6.1)}$$

f_a = axial stress

f_{b33} = bending stress about the local 3-axis

f_{b22} = bending stress about the local 2-axis

F_a = allowable axial stress

F_{b33} = allowable bending stress about the local 3-axis

F_{b22} = allowable bending stress about the local 2-axis

$$F'_e = \frac{12\pi^2 E}{23(Kl\,/\,r)^2}\,. \qquad \text{(ASD H1)}$$

A factor of 4/3 is NOT applied on F_e and $0.6F_y$ if the load combination includes any wind load or seismic load (UBC 1612.3.1).

C_{m33} and C_{m22} are coefficients representing distribution of moment along the member length. They are calculated as described in AISC-ASD89 Steel Frame Design Technical Note 40 Calculation of Stress Ratios.

When the stress ratio is calculated for Special Seismic Load Combinations, the column axial allowable stress in compression is taken to be $1.7F_a$ instead of F_a (UBC 2213.4.2).

- If f_a is compressive and f_a / $F_a \le 0.15$, a relatively simplified formula is used for the combined stress ratio.

$$\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}} \qquad \text{(ASD H1-3, SAM 6.1)}$$

- If f_a is tensile or zero, the combined stress ratio is given by the larger of

$$\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}\ ,\ \text{and} \qquad \text{(ASD H2-1, SAM 6.2)}$$

$$\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \text{ where}$$

f_a, f_{b33}, f_{b22}, F_a, F_{b33}, and F_{b22} are as defined earlier in this Technical Note. However, either F_{b33} or F_{b22} need not be less than $0.6F_y$ in the first equation (ASD H2-1). The second equation considers flexural buckling without any beneficial effect from axial compression.

When the stress ratio is calculated for Special Seismic Load Combinations, the column axial allowable stress in tension is taken to be F_y instead of F_a (UBC 2213.4.2).

For circular and pipe sections, an SRSS combination is first made of the two bending components before adding the axial load component, instead of the simple addition implied by the above formula.

For Single-angle sections, the combined stress ratio is calculated based on the properties about the principal axis (ASD SAM 5.3, 6.1.5). For I, Box, Channel, T, Double-angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes are determined in the program. For general sections, it is assumed that all section properties are given in terms of the principal directions, and consequently, no effort is made to determine the principal directions.

In contrast to the AISC-ASD code, when designing for combinations involving earthquake and wind loads, allowable stresses are NOT increased by a factor of 4/3 of the regular allowable value (UBC 1612.3.1, 2209.3).

Shear Stresses

From the allowable shear stress values and the factored shear stress values at each station, shear stress ratios for major and minor directions are computed for each of the load combinations as follows:

$$\frac{f_{v2}}{F_v}, \quad \text{and}$$

$$\frac{f_{v3}}{F_v}.$$

For Single-angle sections, the shear stress ratio is calculated for directions along the geometric axis. For all other sections, the shear stress is calculated along the principle axes that coincide with the geometric axes.

In contrast to AISC-ASD code, when designing for combinations involving earthquake and wind loads, allowable shear stresses are NOT increased by a factor of 4/3 of the regular allowable value (UBC 1612.3.1, 2209.3).

STEEL FRAME DESIGN UBC97-ASD

Technical Note 13
Seismic Requirements

This Technical Note explains the special seismic requirements checked by the program for member design. Those requirements are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and on UBC Section 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2205.3, 2208, 2212, 2213, 2214). No special requirement is checked for frames in Seismic Zone 0.

Ordinary Moment Frames

For this framing system, the following additional requirements are checked and reported:

- In Seismic Zones 3 and 4, whenever the axial stress, f_a, in columns caused by the prescribed loading combinations exceeds 0.3 F_y, the Special Seismic Load Combinations as described below are checked with respect to the column axial load capacity only (UBC 2213.5.1).

  $1.0DL + 0.7\ LL \pm \Omega_o\ EL$ (UBC 2213.5.1.1)

  $0.85\ DL \pm \Omega_o\ EL$ (UBC 2213.5.1.2)

  In this case, column forces are replaced by the column forces for the Special Seismic Load Combinations, whereas the other forces are taken as zeros. For this case, the column axial allowable stress in compression is taken to be 1.7 F_a instead of F_a, and the column axial allowable stress in tension is taken to be F_y instead of F_a (UBC 2213.5.1, 2213.4.2).

Special Moment Resisting Frames

For this framing system, the following additional requirements are checked or reported:

- In Seismic Zones 3 and 4, when the axial stress, f_a, in columns caused by the prescribed loading combinations exceeds 0.3 F_y, the Special Seismic

Load Combinations as described below are checked with respect to the column axial load capacity only (UBC 2213.5.1).

$1.0DL + 0.7\ LL \pm \Omega_o\ EL$ (UBC 2213.5.1.1)
$0.85\ DL \pm \Omega_o\ EL$ (UBC 2213.5.1.2)

In this case, column forces are replaced by the column forces for the Special Seismic Load Combinations, whereas the other forces are taken as zeros. For this case, the column axial allowable stress in compression is taken to be 1.7 F_a instead of F_a, and the column axial allowable stress in tension is taken to be F_y instead of F_a (UBC 2213.5.1, 2213.4.2).

- In Seismic Zones 3 and 4, the I-shaped beams, I-shaped columns, and Box-shaped columns are additionally checked for compactness criteria as described in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections (UBC 2213.7.3). Compact I-shaped beam sections are also checked for $b_f/2t_f$ to be less than $52/\sqrt{F_y}$. Compact I-shaped column sections are additionally checked for $b_f/2t_f$ to be less than the numbers given for plastic sections in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. Compact box shaped column sections are also checked for b/t_f to be less than $110/\sqrt{F_y}$.

  If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. If this criterion is not satisfied, the user must modify the section property

- In Seismic Zones 3 and 4, the program checks the laterally unsupported length of beams to be less than $96r_y$. If the check is not satisfied, it is noted in the output (UBC 2213.7.8).

Braced Frames

For this framing system, the following additional requirements are checked or reported:

- In Seismic Zones 3 and 4, when the axial stress, f_a, in columns resulting from the prescribed loading combinations exceeds 0.3 F_y, the Special Seismic Load Combinations as described below are checked with respect to the column axial load capacity only (UBC 2213.5.1).

$1.0DL + 0.7\ LL \pm \Omega_o\ EL$ (UBC 2213.5.1.1)

$0.85\ DL \pm \Omega_o\ EL$ (UBC 2213.5.1.2)

In this case, column forces are replaced by the column forces for the Special Seismic Load Combinations, whereas the other forces are taken as zeros. For this case, the column axial allowable stress in compression is taken to be $1.7F_a$ instead of F_a, and the column axial allowable stress in tension is taken to be F_y instead of F_a (UBC 2213.5.1, 2213.4.2).

- In Seismic Zones 3 and 4, the program checks the laterally unsupported length of beams to be less than $96r_y$. If the check is not satisfied, it is noted in the output (UBC 2213.8.1, 2213.7.8).

- In Seismic Zones 3 and 4, the maximum l/r ratio of the braces is checked not to exceed $720/\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2213.8.2.1).

- In Seismic Zones 3 and 4, the Angle, Double-angle, Box, and Pipe shaped braces are additionally checked for compactness criteria, as described in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections (UBC223.8.2.5). For angles and double-angles, b/t is limited to $52/\sqrt{F_y}$; for box sections, b/t_f and d/t_w is limited to $110/\sqrt{F_y}$; for pipe sections, D/t is limited to $1{,}300/F_y$. If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. If this criterion is not satisfied, the user must modify the section property.

- In Seismic Zones 3 and 4, the allowable compressive stress for braces is reduced by a factor of B where

$$B = \frac{1}{1 + \dfrac{Kl/r}{2C_c}} \qquad \text{(UBC 2213.8.2.2)}$$

In Seismic Zones 1 and 2, the allowable compressive stress for braces is reduced by the same factor B where

$$B \geq 0.8 \qquad \text{(UBC 2214.6.2.1)}$$

- In Seismic Zones 3 and 4, Chevron braces are designed for 1.5 times the specified load combination (UBC 2213.8.4.1).

Eccentrically Braced Frames

For this framing system, the program looks for and recognizes the eccentrically braced frame configuration shown in Figure 1. The following additional requirements are checked or reported for the beams, columns and braces associated with these configurations. Special seismic design of eccentrically braced frames in Seismic Zones 1 and 2 is the same as that in Seismic Zones 3 and 4 (UBC 2214.8).

- In all Seismic zones except Zone 0, the I-shaped beam sections are also checked for compactness criteria as described in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. Compact I-shaped beam sections are also checked for $b_f/2t_f$ to be less than $52/\sqrt{F_y}$.

  If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. If this criterion is not satisfied, the user must modify the section property (UBC 2213.10.2). Other sections meeting this criterion are also reported as SEISMIC.

- In all Seismic Zones except Zone 0, the link beam strength in shear V_s=0.55$F_y d t_w$ and moment M_s=ZF_y are calculated. If $V_s \leq 2.0M_s/e$, the link beam strength is assumed to be governed by shear and is so reported. If the above condition is not satisfied, the link beam strength is assumed to be governed by flexure and is so reported. When link beam strength is governed by shear, the axial and flexural properties (area, A and section modulus, S) for use in the interaction equations are calculated based on the beam flanges only (UBC 2213.10.3).

- In all Seismic Zones except Zone 0, if the link beam is connected to the column, the link beam length e is checked not to exceed the following (UBC 2213.10.12):

  $$e \leq 1.6\frac{M_p}{V_p} \qquad \text{(UBC 2213.10.12)}$$

  If the check is not satisfied, it is noted in the output.

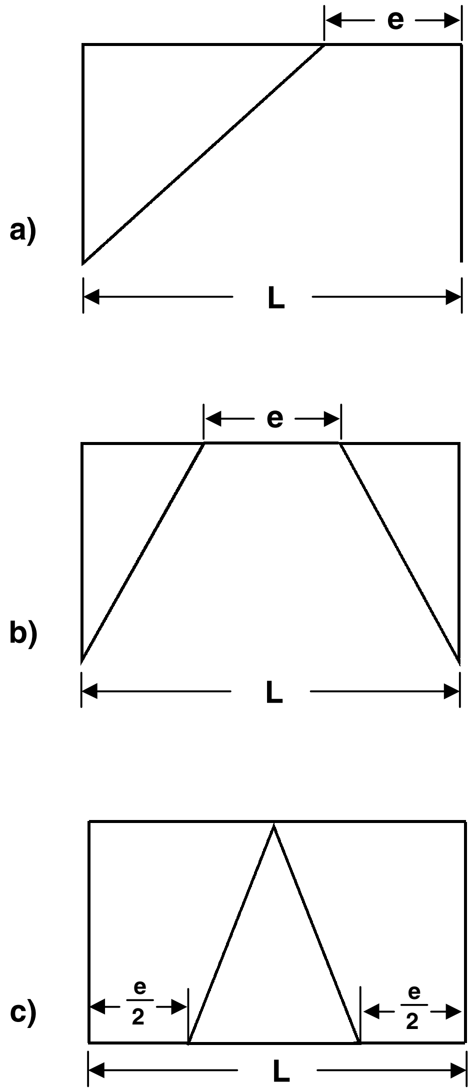


Figure 1 Eccentrically Braced Frame Configurations

- In all Seismic Zones except Zone 0, the link beam rotation θ of the individual bay relative to the rest of the beam is calculated as the story drift $delta_M$ times bay length divided by the total lengths of link beams in the bay divided by height of the story. The link beam rotation θ is checked to be less than the following values (UBC 2213.10.4).

$\theta \leq 0.090$, where link beam clear length, $e \leq 1.6\ M_s/V_s$

$\theta \leq 0.030$, where link beam clear length, $e \geq 3.0\ M_s/V_s$, and

$\theta \leq$ value interpolated between 0.090 and 0.030 as the link beam clear length varies from $1.6\ M_s/V_s$ to $3.0\ M_s/V_s$.

- In all Seismic zones except Zone 0, the link beam shear under the specified loading combinations is checked not to exceed $0.8V_s$ (UBC 2213.10.5).

- In all Seismic Zones except Zone 0, the brace strength is checked to be at least 1.5 times the axial force corresponding to the controlling link beam strength (UBC 2213.10.13). The controlling link beam strength is either the shear strength, V_s, as $V_s=0.55F_y dt_w$, or the reduced flexural strength M_{rs}, whichever produces the lower brace force. The value of M_{rs} is taken as $M_{rs} = Z(F_y - f_a)$(UBC 2213.10.3), where f_a is the lower of the axial stress in the link beam corresponding to yielding of the link beam web in shear or the link beam flanges in flexure. The correspondence between brace force and link beam force is obtained from the associated load cases, whichever has the highest link beam force of interest.

- In all Seismic Zones except Zone 0, the column is checked to not become inelastic for gravity loads plus 1.25 times the column forces corresponding to the controlling link beam strength (UBC 2213.10.14). The controlling link beam strength and the corresponding forces are as obtained by the process described above. If this condition governs, the column axial allowable stress in compression is taken to be $1.7F_a$ instead of F_a, and the column axial allowable stress in tension is taken to be F_y instead of F_a.

- In all Seismic Zones except Zone 0, axial forces in the beams are included in checking of beams (UBC 2211.10.17). The user is reminded that using a rigid diaphragm model will result in zero axial forces in the beams. The user must disconnect some of the column lines from the diaphragm to allow beams to carry axial loads. It is recommended that only one column line per eccentrically braced frame be connected to the rigid diaphragm or a flexible diaphragm model be used.

- In all Seismic Zones except Zone 0, the beam laterally unsupported length is checked to be less than 76 $b_f/\sqrt{F_y}$. If not satisfied, it is so noted as a warning in the output file (UBC 2213.10.18).

Note: The beam strength in flexure, of the beam outside the link, is **NOT** currently checked to be at least 1.5 times the moment corresponding to the controlling link beam strength (UBC 2213.10.13). Users need to check for this requirement.

Special Concentrically Braced Frames

Special seismic design of special concentrically braced frames in Seismic Zones 1 and 2 is the same as those in Seismic zones 3 and 4 (UBC 2214.7). For this framing system, the following additional requirements are checked or reported:

- In all Seismic Zones except Zone 0, when the axial stress f_a in columns resulting from the prescribed loading combinations exceeds $0.3F_y$, the Special Seismic Load Combinations as described below are checked with respect to the column axial load capacity only (UBC 2213.9.5, 2213.5.1).

  $1.0\ DL + 0.7\ LL \pm \Omega_o EL$ (UBC 2213.5.1.1)

  $0.85\ DL \pm \Omega_o EL$ (UBC 2213.5.1.2)

  In this case, column forces are replaced by the column forces for the Special Seismic Load Combinations, whereas the other forces are taken as zeros. For this case, the column axial allowable stress in compression is taken to be 1.7 F_a instead of F_a, and the column axial allowable stress in tension is taken to be F_y instead of F_a (UBC 2213.5.1, 2213.4.2).

- In all Seismic Zones except Zone 0, the I-shaped and Box-shaped columns are also checked for compactness criteria as described in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. Compact I-shaped column sections are also checked for $b_f/2t_f$ to be less than the numbers given for plastic sections in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. Compact Box-shaped column sections also are checked for b/t_f and d/t_w to be less than $110/\sqrt{F_y}$. If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-ASD Steel Frame Design Technical Note 9

Classification of Sections. If this criterion is not satisfied, the user must modify the section property (UBC 2213.9.5, 2213.7.3).

- In all Seismic Zones except Zone 0, bracing members are checked to be compact and are so reported. The Angle, Box, and Pipe sections used as braces are also checked for compactness criteria as described in Table 1 of UBC97-ASD Steel Frame Design Technical Note 9 Classification of Sections. For angles, b/t is limited to $52/\sqrt{F_y}$; for pipe sections, D/t is limited to $1{,}300/F_y$. If this criterion is satisfied, the section is reported as SEISMIC. If this criterion is not satisfied, the user must modify the section property (UBC 2213.9.2.4).

- In all Seismic Zones except Zone 0, the maximum Kl/r ratio of the braces is checked not to exceed $1{,}000/\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2213.9.2.1).

Note: Beams intersected by Chevron braces are **NOT** currently checked to have a strength to support loads represented by the following loading combination (UBC 2213.9.14):

$$1.2\ DL + 0.5LL \pm P_b \qquad \text{(UBC 2213.9.4.1)}$$

$$0.9\ DL \pm P_b \qquad \text{(UBC 2213.9.4.1)}$$

where P_b is given by the difference of F_yA for the tension brace and 0.3 times $1.7F_aA$ for the compression brace. Users need to check for this requirement (UBC 2213.9.4.1, 2213.4.2).

Technical Note 14
Joint Design

This Technical Note describes how the program checks or designs joints. When using UBC97-ASD design code, the structural joints are checked or designed for the following:

- Check for the requirement of continuity plate and determination of its area (see UBC97-ASD Steel Frame Design Technical Note 15 Continuity Plates)

- Check for the requirement of doubler plate and determination of its thickness (see Steel Frame Design UBC97-ASD Technical Note 16 Doubler Plates)

- Check for ratio of beam flexural strength to column flexural strength

- Reporting the beam connection shear

- Reporting the brace connection force

Beam/Column Plastic Moment Capacity Ratio

In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the code requires that the sum of beam flexure strengths at a joint should be less than the sum of column flexure strengths (UBC 2213.7.5). The column flexure strength should reflect the presence of axial force present in the column. To facilitate the review of the strong-column/weak-beam criterion, the program reports a beam/column plastic moment capacity ratio for every joint in the structure.

For the major direction of any column (top end) the beam-to-column strength ratio is obtained as:

$$R_{maj} = \frac{\sum_{n=1}^{n_b} M_{pbn} \cos \theta_n}{M_{pcax} + M_{pcbx}} \qquad \text{(UBC 2213.7.5)}$$

For the minor direction of any column the beam-to-column strength ratio is obtained as:

$$R_{min} = \frac{\sum_{n=1}^{n_b} M_{pbn} \sin\theta_n}{M_{pcay} + M_{pcby}} \qquad \text{(UBC 2213.7.5)}$$

where,

$R_{maj,\ min}$ = Plastic moment capacity ratios, in the major and minor directions of the column, respectively

M_{pbn} = Plastic moment capacity of n-th beam connecting to column,

θ_n = Angle between the n-th beam and the column major direction,

$M_{pcax,y}$ = Major and minor plastic moment capacities, reduced for axial force effects, of column above story level. Currently, it is taken equal to $M_{pcbx,y}$ if there is a column above the joint. If there is no column above the joint, $M_{pcax,y}$ is taken as zero.

$M_{pcbx,y}$ = Major and minor plastic moment capacities, reduced for axial force effects, of column below story level, and

n_b = Number of beams connecting to the column.

The plastic moment capacities of the columns are reduced for axial force effects and are taken as

$$M_{pc} = Z_c(F_{yc} - f_a), \qquad \text{(UBC 2213.7.5)}$$

where,

Z_c = Plastic modulus of column,

F_{yc} = Yield stress of column material, and

f_a = Maximum axial stress in the column.

For the above calculations, the section of the column above is taken to be the same as the section of the column below assuming that the column splice will be located some distance above the story level.

Evaluation of Beam Connection Shears

For each steel beam in the structure, the program will report the maximum major shears at each end of the beam for the design of the beam shear connections. The beam connection shears reported are the maxima of the factored shears obtained from the loading combinations.

For special seismic design, the beam connection shears are not taken less than the following special values for different types of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and on UBC Section 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2213, 2214). No special requirement is checked for frames in Seismic Zone 0.

- In all Seismic Zones except Zone 0, for Ordinary Moment Frames, the beam connection shears reported are the maximum of the specified loading combinations and the following additional loading combinations (UBC 2213.6.2, 2214.4.2):

$$1.0 \text{ DL} + 1.0\text{LL} \pm \Omega_0 \text{ EL} \qquad \text{(UBC 2213.6.2, 2214.4.2)}$$

- In all Seismic Zones except Zone 0, for Special Moment-Resisting Frames, the beam connection shears that are reported allow for the development of the full plastic moment capacity of the beam (UBC 2213.7.1, 22145.5.1.1). Thus:

$$V = \frac{CM_{pb}}{L} + V_{DL+LL} \qquad \text{(UBC 2213.7.1.1, 2214.5.1.1)}$$

where,

V = Shear force corresponding to END I and END J of beam

C = 0 if beam ends are pinned or for cantilever beam,

= 1 if one end of the beam is pinned,

= 2 if no ends of the beam are pinned,

M_{pb} = Plastic moment capacity of the beam, ZF_y,

L = Clear length of the beam, and

V_{DL+LL} = Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the dead load and live load combinations only.

- In all Seismic Zones except Zone 0, for Eccentrically Braced Frames, the beam connection shears reported are the maximum of the specified loading combinations and the following additional loading combination:

$$1.0 \text{ DL} + 1.0\text{LL} \pm \Omega_0 \text{ EL}$$

Evaluation of Brace Connection Forces

For each steel brace in the structure, the program reports the maximum axial force at each end of the brace for the design of the brace-to-beam connections. The brace connection forces reported are the maxima of the factored brace axial forces obtained from the loading combinations.

For special seismic design, the brace connection forces are not taken less than the following special values for different types of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and on UBC 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2213, 2214). No special requirement is checked for frames in Seismic Zone 0.

- In all Seismic zones except Zone 0, for Ordinary Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace (F_yA) and the following special loading combination (UBC 2213.8.3.1, 2214.6.3.1).

$$1.0 \text{ DL} + 1.0\text{LL} \pm \Omega_0 \text{ EL} \qquad \text{(UBC 2213.8.3.1, 2214.6.3.1)}$$

- In all Seismic Zones except Zone 0, for Special Concentrically Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace (F_yA) and the following special loading combination (UBC 2213.9.3.1, 2214.7):

$$1.0\ DL + 1.0LL \pm \Omega_0\ EL \qquad \text{(UBC 2213.9.3, 2214.7)}$$

- In all Seismic Zones except Zone 0, for Eccentrically Braced Frames, the bracing connection force is reported as at least the brace strength in compression that is computed as $1.7F_a A$ (UBC 2213.10.6, 2214.8). $1.7F_a A$ is limited not to exceed $F_y A$.

Technical Note 15
Continuity Plates

This Technical Note describes how this program can be used in the design of continuity plates.

In a plan view of a beam/column connection, a steel beam can frame into a column in the following ways:

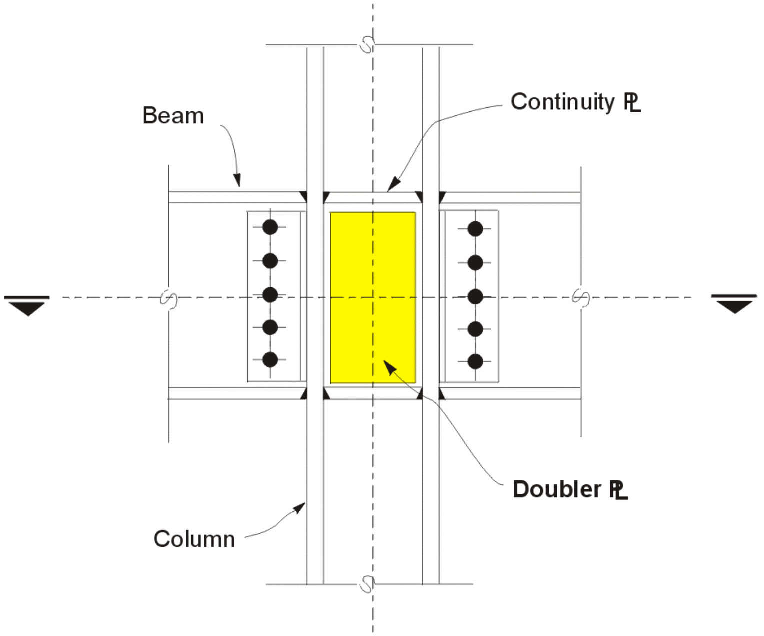
1. The steel beam frames in a direction parallel to the column major direction, i.e., the beam frames into the column flange.

2. The steel beam frames in a direction parallel to the column minor direction, i.e., the beam frames into the column web.

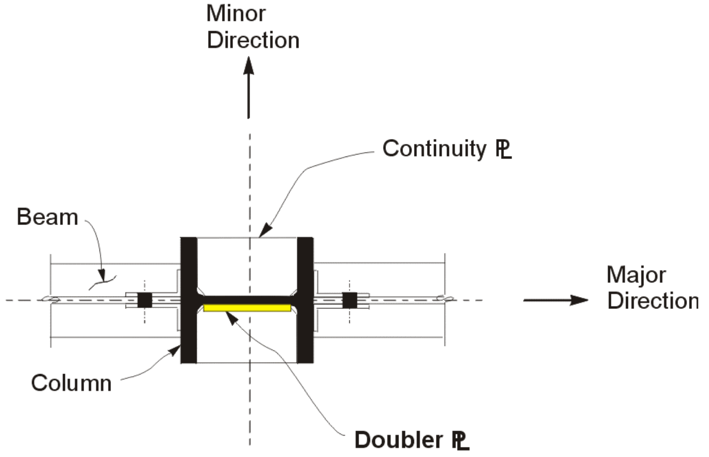
3. The steel beam frames in a direction that is at an angle to both of the principal axes of the column, i.e., the beam frames partially into the column web and partially into the column flange.

To achieve a beam/column moment connection, continuity plates such as shown in Figure 1 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.

For the connection described in conditions 2 and 3 above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam. However, for the connection described by condition 1, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends on the magnitude of the beam-flange force and the properties of the column. This is the condition that the program investigates. Columns of I-sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e., parallel to the column major direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.



ELEVATION



SECTION

Figure 1 Elevation and Plan of Doubler Plates for a Column of I-Section

The continuity plate area required for a particular beam framing into a column is given by:

$$A_{cp} = \frac{P_{bf}}{F_{yc}} - t_{wc}\,(t_{fb} + 5k_c) \qquad \text{(ASD K1-9)}$$

If $A_{cp} \leq 0$, no continuity plates are required, provided the following two conditions are also satisfied:

- The depth of the column clear of the fillets, i.e., d_c - $2k_c$, is less than or equal to:

$$\frac{4{,}100 t_{wc}^{3} \sqrt{F_{yc}}}{P_{bf}} \qquad \text{(ASD K1-8)}$$

- The thickness of the column flange, t_{fc}, is greater than or equal to:

$$0.4\sqrt{\frac{P_{bf}}{F_{yc}}}\text{, where} \qquad \text{(ASD K1-1)}$$

$$P_{bf} = f_b A_{bf}.$$

f_b is the bending stress calculated from the larger of 5/3 of loading combinations with gravity loads only $(5/3)M/[(d\text{-}t_f)A_{fb}]$ and 4/3 of the loading combination with lateral loads $(4/3)M/[(d\text{-}t_f)A_{fb}]$ (ASD K1.2). For special seismic design, f_a is specified to be beam flange strength.

If continuity plates are required, they must satisfy a minimum area specification defined as follows:

- The thickness of the stiffeners is at least .05 t_{fb}, or

$$t_{cp}^{\min} = 0.5\ t_{fb} \qquad \text{(ASD K1.8.2)}$$

- The width of the continuity plate on each side plus 1/2 the thickness of the column web shall not be less than 1/3 of the beam flange width, or

$$b_{cp}^{\min} = \left(\frac{b_{fb}}{3} - \frac{t_{wc}}{2}\right) \qquad \text{(ASD K1.8.1)}$$

- So that the minimum area is given by:

$$A_{cp}^{\min} = t_{cp}^{\min}\ b_{cp}^{\min}$$

Therefore, the continuity plate area provided by the program is either zero or the greater of A_{cp} and $A_{cp}^{\min}$.

Where

A_{bf} = Area of beam flange

A_{cp} = Required continuity plate area

F_{yb} = Yield stress of beam material

F_{yc} = Yield stress of the column and continuity plate material

t_{fb} = Beam flange thickness

t_{wc} = Column web thickness

k_c = Distance between outer face of the column flange and web toe of its fillet

d_c = Column depth

d_b = Beam depth

f_b = Beam flange depth

t_{cp} = Continuity plate thickness

b_{cp} = Continuity plate width

f_b = Bending stress calculated from the larger of 5/3 of loading combinations with gravity loads only $(5/3)M/[(d\text{-}t_f)A_{fb}]$ and 4/3 of the loading combinations with lateral loads $(4/3)M/[(d\text{-}t_f)A_{fb}]$ (ASD K1.2).

The special seismic requirements additionally checked by the program are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and UBC Section 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2212, 2214). No special requirement is checked for frames in Seismic Zone 0.

- In all Seismic Zones except Zone 0, for Ordinary Moment Frames the continuity plates are checked and designed for a beam flange force, P_{bf}.

  $P_{bf} = f_{yb}A_{bf}$ (UBC 2213.6.1, 2213.7.1.1, 2214.4.1, 2214.5.1.1)

- In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, for determining the need for continuity plates at joints resulting from tension transfer from the beam flanges, the force P_{bf} is taken as 1.8 $f_{yb}A_{bf}$ (UBC 2213.7.4). For design of the continuity plate, the beam flange force is taken as $f_{yb}A_{bf}$ (UBC 2213.7.1.1).

  In Seismic Zones 1 and 2, for Special Moment-Resisting Frames, for determining the need for continuity plates at joints resulting from tension transfer from the beam flanges, the force P_{bf} is taken as $f_{yb}A_{bf}$. For design of the continuity plate, the beam flange force is taken as $f_{yb}A_{bf}$ (UBC 2214.5.1.1).

- In all Seismic zones except Zone 0, for Eccentrically Braced Frames, the continuity plates are checked and designed for a beam flange force, P_{bf}.

  $P_{bf} = f_{yb}A_{bf}$ (UBC 2213.10.12, 2213.10.19)

STEEL FRAME DESIGN UBC97-ASD

Technical Note 16
Doubler Plates

This Technical Note explains how the program can be used in the design of doubler plates.

One aspect of the design of a steel frame system is an evaluation of the shear forces that exist in the region of the beam column intersection known as the panel zone.

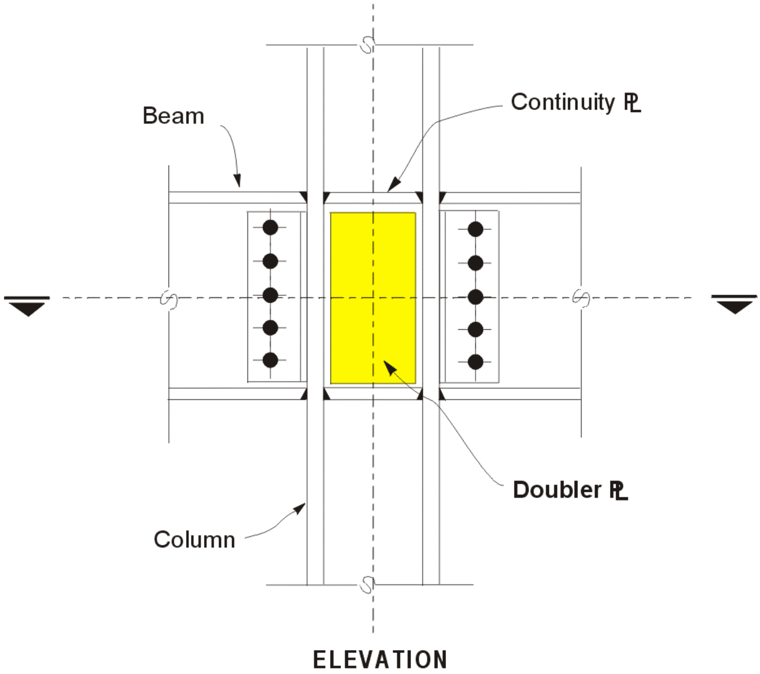
Shear stresses seldom control the design of a beam or column member. However, in a Moment-Resisting frame, the shear stress in the beam-column joint can be critical, especially in framing systems when the column is subjected to major direction bending and the joint shear forces are resisted by the web of the column. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear stresses are seldom critical, and this condition is therefore not investigated by the program.

Shear stresses in the panel zone, resulting from major direction bending in the column, may require additional plates to be welded onto the column web, depending on the loading and geometry of the steel beams that frame into the column, either along the column major direction, or at an angle so that the beams have components along the column major direction. See Figure 1. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames only. No check is made for braced frames.

The shear fore in the panel zone is given by:

$$V_p = P - V_c \text{, or}$$

$$V_p = \sum_{n=1}^{n_b} \frac{M_{bn} \cos \theta_n}{d_n - t_{f_n}} - V_c$$



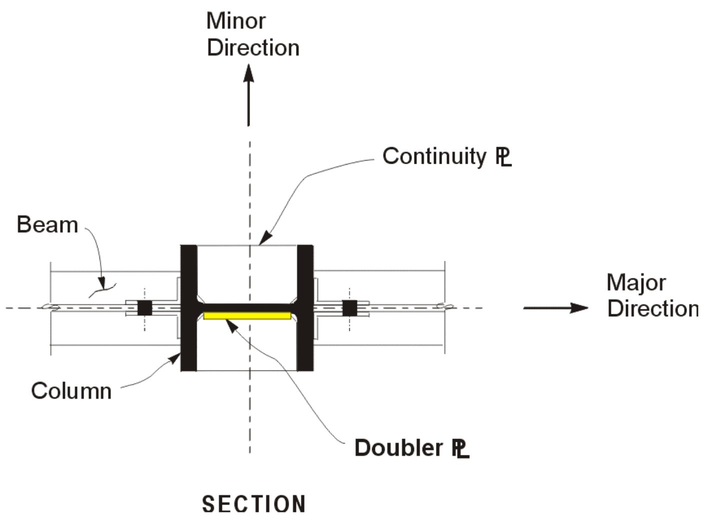


Figure 1 Elevation and Plan of Doubler Plates for a Column of I-Section

The required web thickness to resist the shear force, V_p, is given by

$$t_r = \frac{V_p}{F_v d_c} \geq \frac{h}{380 / \sqrt{F_{yc}}} \qquad \text{(ASD F4)}$$

The extra thickness, or thickness of the doubler plate is given by

$t_{dp} = t_r - t_{wc}$, where

F_v = $0.40F_{yc}$ (ASD F4)

F_{yc} = Yield stress of the column and doubler plate material

t_r = Required column web thickness

t_{dp} = Required doubler plate thickness

t_{fn} = Thickness of flange of the *n*-th beam connecting to column

t_{wc} = Column web thickness

V_p = Panel zone shear

V_c = Column shear in column above

P = Beam flange forces

n_b = Number of beams connecting to column

d_n = Depth of *n*-th beam connecting to column

h = d_c - $2t_{fc}$ if welded, d_c - $2k_c$ if rolled

θ_n = Angle between *n*-th beam and column major direction

d_c = Depth of column

M_{bn} = Calculated factored beam moment from the corresponding loading combination

The largest calculated value of V_p calculated for any of the load combinations based on the factored beam moments is used to calculate doubler plate areas.

The special seismic requirements checked by the program for calculating doubler plate areas are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and on UBC Section 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2213, 2214). No special requirement is checked for frames in Seismic Zones 0, 1 and 2.

- In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the panel zone doubler plate requirements that are reported will develop the lesser of beam moments equal to 0.8 of the plastic moment capacity of the beam ($0.8\Sigma M_{pb}$), or beam moments caused by gravity loads plus 1.85 times the seismic load (UBC 2213.7.2.1).

  The capacity of the panel zone in resisting this shear is taken as (UBC 2213.7.2.1):

  $$V_P = 0.55 F_{yc} d_c t_r \left(1 + \frac{3 b_c t_{cf}^2}{d_b d_c t_r}\right) \qquad \text{(UBC 2213.7.2.1)}$$

  giving the required panel zone thickness as

  $$t_r = \frac{V_p}{0.55 F_{yc} d_c} - \frac{3 b_c t_{cf}^2}{d_b d_c} \geq \frac{h}{380 / \sqrt{F_{yc}}} \qquad \text{(UBC 2213.7.2.1, ASD F4)}$$

  and the required doubler plate thickness as

  $$t_{dp} = t_r - t_{wc}$$

  where

  b_c = width of column flange

  h = d_c-$2t_{fc}$ if welded, d_c-$2k_c$ if rolled,

  t_{cf} = thickness of column flange, and

  d_b = depth of deepest beam framing into the major direction of the column

- In Seismic Zones 3 and 4 for Special Moment-Resisting Frames, the program checks the following panel zone column web thickness requirement:

$$t_{wc} \geq \frac{(d_c - 2t_{fc}) + (d_b - 2t_{fb})}{90} \qquad \text{(UBC 2213.7.2.2)}$$

If the check is not satisfied, it is noted in the output.

- In Seismic Zones 3 and 4, for Eccentrically Braced Frames, the doubler plate requirements are checked similar to the doubler plate checks for special Moment-Resisting frames as described previously (UBC 2213.10.19).

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STEEL FRAME DESIGN UBC97-ASD

Technical Note 17
Input Data

This Technical Note describes the steel frame design input data for UBC97-ASD. The input can be printed to a printer or to a text file when you click the **File menu > Print Tables > Steel Frame Design** command. A printout of the input data provides the user with the opportunity to carefully review the parameters that have been input into the program and upon which program design is based. Further information about using the Print Design Tables Form is provided at the end of this Technical Note.

Input Data

The program provides the printout of the input data in a series of tables. The column headings for input data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Material Property Data	
Material Name	Steel, concrete or other.
Material Type	Isotropic or orthotropic.
Design Type	Concrete, steel or none. Postprocessor available if steel is specified.
Material Dir/Plane	"All" for isotropic materials; specify axis properties define for orthotropic.
Modulus of Elasticity	
Poisson's Ratio	
Thermal Coeff	
Shear Modulus	
Material Property Mass and Weight	
Material Name	Steel, concrete or other.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Mass Per Unit Vol	Used to calculate self mass of the structure.
Weight Per Unit Vol	Used to calculate the self weight of the structure.
Material Design Data for Steel Materials	
Material Name	Steel.
Steel FY	Minimum yield stress of steel.
Steel FU	Maximum tensile stress of steel.
Steel Cost ($)	Cost per unit weight used in composite beam design if optimum beam size specified to be determined by cost.
Material Design Data for Concrete Materials	
Material Name	Concrete.
Lightweight Concrete	Check this box if this is a lightweight concrete material.
Concrete FC	Concrete compressive strength.
Rebar FY	Bending reinforcing yield stress.
Rebar FYS	Shear reinforcing yield stress.
Lightwt Reduc Fact	Define reduction factor if lightweight concrete box checked. Usually between 0.75 ad 0.85.
Frame Section Property Data	
Frame Section Name	User specified or auto selected member name.
Material Name	Steel, concrete or none.
Section Shape Name or Name in Section Database File	Name of section as defined in database files.
Section Depth	Depth of the section.
Flange Width Top	Width of top flange per AISC database.
Flange Thick Top	Thickness of top flange per AISC database.
Web Thick	Web thickness per AISC database.
Flange Width Bot	Width of bottom flange per AISC database.
Flange Thick Bot	Thickness of bottom flange per AISC database.
Section Area	

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Torsional Constant	
Moments of Inertia	I33, I22
Shear Areas	A2, A3
Section Moduli	S33, S22
Plastic Moduli	Z33, Z22
Radius of Gyration	R33, R22
Load Combination Multipliers	
Combo	Load combination name.
Type	Additive, envelope, absolute, or SRSS as defined in **Define > Load Combination**.
Case	Name(s) of case(s) to be included in this load combination.
Case Type	Static, response spectrum, time history, static nonlinear, sequential construction.
Factor	Scale factor to be applied to each load case.
Beam Steel Stress Check Element Information	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member section assigned.
Framing Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, ERF
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Beam Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
CM Major	As defined in AISC-ASD, page 5-55.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
CM Minor	As defined in AISC-ASD, page 5-55.
Cb Factor	As defined in AISC-ASD, page 5-47.
Beam Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Fa	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Ft	If zero, as defined for material property data used and AISC-ASD Chapter D.
Fb Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fb Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Column Steel Stress Check Element Information	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Framing Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, ERF
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Column Steel Moment Magnification Overwrites	
Story Level	Name of the story level.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Column Line	Column line identifier.
CM Major	As defined in AISC-ASD, page 5-55.
CM Minor	As defined in AISC-ASD, page 5-55.
Cb Factor	As defined in AISC-ASD, page 5-47.
Column Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
Fa	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Ft	If zero, as defined for material property data used and AISC-ASD Chapter D.
Fb Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fb Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.

Using the Print Design Tables Form

To print steel frame design input data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Input Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design input data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

Technical Note 18
Output Details

This Technical Note describes the steel frame design output for UBC97-ASD that can be printed to a printer or to a text file. The design output is printed when you click the **File menu > Print Tables > Steel Frame Design** command and select Output Summary on the Print Design Tables form. Further information about using the Print Design Tables form is provided at the end of this Technical Note.

The program provides the output data in tables. The column headings for output data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Beam Steel Stress Check Output	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
Axl	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Beam Special Seismic Requirements	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Section Class	Classification of section for the enveloping combo.
Connection Shear	
Combo	Name of the load combination that provides maximum End-I connection shear.
END-I	Maximum End-I connection shear.
Combo	Name of the load combination that provides maximum End-J connection shear.
END-J	Maximum End-J connection shear.
Column Steel Stress Check Output	
Story Level	Name of the story level.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
AXL	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Column Special Seismic Requirements	
Story Level	Story level name.
Column Line	Column line identifier.
Section ID	Name of member section assigned.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Section Class	Classification of section for the enveloping combo.
Continuity Plate	
Combo	Name of load combination that produces maximum continuity plate area.
Area	Cross-section area of the continuity plate.
Doubler Plate	
Combo	Name of load combination that produces maximum doubler plate thickness.
Thick	Thickness of the doubler plate.
B/C Ratios	
Major	Beam/column capacity ratio for major direction.
Minor	Beam/column capacity ratio for minor direction.

Using the Print Design Tables Form

To print steel frame design output data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Output Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design output data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

Technical Note 19
General and Notation

Introduction to the UBC97-LRFD Series of Technical Notes

The UBC97-LRFD design code in this program implements the International Conference of Building Officials *1997 Uniform Building Code: Volume 2: Structural Engineering Design Provisions*, Chapter 22, Division II, "Design Standard for Load and Resistance Factor Design Specification for Structural Steel Buildings (ICBO 1997).

Chapter 22, Division III of UBC adopted the American Institute of Steel Construction's Load and Resistance Factor Design Specification for Structural Steel Buildings (AISC 1993), which has been implemented in the AISC-LRFD93 code in ETABS.

For referring to pertinent sections and equations of the UBC code, a unique prefix "UBC" is assigned. For referring to pertinent sections and equations of the AISC-LRFD code, a unique prefix "LRFD" is assigned. However, all references to the "Specifications for Load and Resistance Factored Design of Single-Angle Members" (AISC 1994) carry the prefix of "LRFD SAM." Moreover, all sections of the "Seismic Provisions for Structural Steel Buildings June 15, 1992" (AISC 1994) are referred to as Section 2211.4 of the UBC code. In the UBC97-LRFD Technical Notes, all sections and subsections referenced by "UBC 2211.4" or "UBC 2211.4.x" refer to the LRFD Seismic Provisions after UBC amendments through UBC Section 2210. Various notations used in the Steel Frame Design UBC97-LRFD series of Technical Notes are described herein.

When using the UBC97-LRFD option, the following Framing Systems are recognized (UBC 1627, 2210):

- Ordinary Moment Frame (OMF)
- Special Moment-Resisting Frame (SMRF)

- Concentrically Braced Frame (CBF)
- Eccentrically Braced Frame (EBF)
- Special Concentrically Braced Frame (SCBF)

By default the frame type is taken as Special-Moment Resisting (SMRF) in the program. However, the frame type can be overwritten in the Preferences (**Options menu > Preferences > Steel Frame Design**) to change the default values and in the Overwrites (**Design menu > Steel Frame Design > View/Revise Overwrites**) on a member-by-member basis. If any member is assigned with a frame type, the change of the frame type in the Preference will not modify the frame type of the individual member for which it is assigned.

When using the UBC97-LRFD option, a frame is assigned to one of the following five Seismic Zones (UBC 2210):

- Zone 0
- Zone 1
- Zone 2
- Zone 3
- Zone 4

By default the Seismic Zone is taken as Zone 4 in the program. However, the frame type can be overwritten in the Preferences to change the default (**Options menu > Preferences > Steel Frame Design**).

The design is based on user-specified loading combinations. To facilitate use, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures. See UBC97-LRFD Steel Frame Design Technical Note 22 Design Load Combinations for more information.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then, the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this series of Technical Notes. The controlling capacity ration is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ration is also calculated separately. Algorithms for completing these calculations are described in UBC97-LRFD Steel Frame Design Technical Note 24 Calculation of Factored Forces and Moments, Technical Note 25 Calculation of Nominal Strengths, and Technical Note 26 Calculation of Capacity Ratios.

Further information is available from UBC97-LRFD Steel Frame Design Technical Notes 23 Classification of Sections, Technical Notes 28 Joint Design, Technical Notes 29 Continuity Plates, and Technical Notes 30 Doubler Plates.

Information about seismic requirements is provided in UBC97-LRFD Steel Frame Design Technical Note 27 Seismic Requirements.

The program uses preferences and overwrites, which are described in UBC97-LRFD Steel Frame Design Technical Note 20 Preferences and Technical Note 21 Overwrites. It also provides input and output data summaries, which are described in UBC97-LRFD Steel Frame Design Technical Note 31 Input Data and Technical Note 32 Output Details.

English as well as SI and MKS metric units can be used for input. The code is based on Kip-Inch-Second units. For simplicity, all equations and descriptions presented in the UBC97-LRFD series of Technical Notes correspond to Kip-Inch-Second units unless otherwise noted.

Notation

A	Cross-sectional area, in^2
A_e	Effective cross-sectional area for slender sections, in^2
A_g	Gross cross-sectional area, in^2
A_{v2},A_{v3}	Major and minor shear areas, in^2
A_w	Shear area, equal dt_w per web, in^2
B_1	Moment magnification factor for moments not causing sidesway

B_2	Moment magnification factor for moments causing sidesway
C_b	Bending coefficient
C_m	Moment coefficient
C_w	Warping constant, in^6
D	Outside diameter of pipes, in
E	Modulus of elasticity, ksi
F_{cr}	Critical compressive stress, ksi
F_r	Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi
F_y	Yield stress of material, ksi
G	Shear modulus, ksi
I_{22}	Minor moment of inertia, in^4
I_{33}	Major moment of inertia, in^4
J	Torsional constant for the section, in^4
K	Effective length factor
K_{33}, K_{22}	Effective length K-factors in the major and minor directions
L_b	Laterally unbraced length of member, in
L_p	Limiting laterally unbraced length for full plastic capacity, in
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, in
M_{cr}	Elastic buckling moment, kip-in
M_{lt}	Factored moments causing sidesway, kip-in
M_{nt}	Factored moments not causing sidesway, kip-in

M_{n33}, M_{n22}	Nominal bending strength in major and minor directions, kip-in
M_{ob}	Elastic lateral-torsional buckling moment for angle sections, kip-in
M_{r33}, M_{r22}	Major and minor limiting buckling moments, kip-in
M_u	Factored moment in member, kip-in
M_{u33}, M_{u22}	Factored major and minor moments in member, kip-in
P_e	Euler buckling load, kips
P_n	Nominal axial load strength, kip
P_u	Factored axial force in member, kips
P_y	A_gF_y, kips
Q	Reduction factor for slender section, = Q_aQ_s
Q_a	Reduction factor for stiffened slender elements
Q_s	Reduction factor for unstiffened slender elements
S	Section modulus, in^3
S_{33}, S_{22}	Major and minor section moduli, in^3
$S_{eff,33}, S_{eff,22}$	Effective major and minor section moduli for slender sections, in^3
S_c	Section modulus for compression in an angle section, in^3
V_{n2}, V_{n3}	Nominal major and minor shear strengths, kips
V_{u2}, V_{v3}	Factored major and minor shear loads, kips
Z	Plastic modulus, in^3
Z_{33}, Z_{22}	Major and minor plastic moduli, in^3
b	Nominal dimension of plate in a section, in

longer leg of angle sections,
$b_f - 2t_w$ for welded and $b_f - 3t_w$ for rolled box sections, etc.

b_e Effective width of flange, in

b_f Flange width, in

d Overall depth of member, in

d_e Effective depth of web, in

h_c Clear distance between flanges less fillets, in
assumed $d - 2k$ for rolled sections, and $d - 2t_f$ for welded sections

k Distance from outer face of flange to web toe of fillet, in

k_c Parameter used for section classification,
$4\Big/\sqrt{h/t_w}$, $0.35 \le k_c \le 0.763$

l_{33}, l_{22} Major and minor directions unbraced member lengths, in

r Radius of gyration, in

r_{33}, r_{22} Radii of gyration in the major and minor directions, in

t Thickness, in

t_f Flange thickness, in

t_w Thickness of web, in

β_w Special section property for angles, in

λ Slenderness parameter

λ_c, λ_e Column slenderness parameters

λ_p Limiting slenderness parameter for compact element

λ_r Limiting slenderness parameter for non-compact element

λ_s Limiting slenderness parameter for seismic element

$\lambda_{slender}$	Limiting slenderness parameter for slender element
φ_b	Resistance factor for bending, 0.9
φ_c	Resistance factor for compression, 0.85
φ_t	Resistance factor for tension, 0.9
φ_v	Resistance factor for shear, 0.9

References

American Institute of Steel Construction (AISC). 1993. *Load and Resistance Factor Design Specification for Structural Steel Building.* Chicago, Illinois.

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International Conference of Building Officials (ICBO). 1997. *1997 Uniform Building Code Volume 2, Structural Engineering Design Provisions.* Whittier, California.

STEEL FRAME DESIGN UBC97-LRFD

Technical Note 20
Preferences

This Technical Note describes the items in the Preferences form.

General

The steel frame design preferences in this program are basic assignments that apply to all steel frame elements. Use the **Options menu > Preferences > Steel Frame Design** command to access the Preferences form where you can view and revise the steel frame design preferences.

Default values are provided for all steel frame design preference items. Thus, it is not required that you specify or change any of the preferences. You should, however, at least review the default values for the preference items to make sure they are acceptable to you.

Using the Preferences Form

To view preferences, select the **Options menu > Preferences > Steel Frame Design.** The Preferences form will display. The preference options are displayed in a two-column spreadsheet. The left column of the spreadsheet displays the preference item name. The right column of the spreadsheet displays the preference item value.

To change a preference item, left click the desired preference item in either the left or right column of the spreadsheet. This activates a drop-down box or highlights the current preference value. If the drop-down box appears, select a new value. If the cell is highlighted, type in the desired value. The preference value will update accordingly. You cannot overwrite values in the drop-down boxes.

When you have finished making changes to the composite beam preferences, click the **OK** button to close the form. You must click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the preferences are ignored and the form is closed.

Preferences

For purposes of explanation in this Technical Note, the preference items are presented in Table 1. The column headings in the table are described as follows:

- **Item:** The name of the preference item as it appears in the cells at the left side of the Preferences form.
- **Possible Values:** The possible values that the associated preference item can have.
- **Default Value:** The built-in default value that the program assumes for the associated preference item.
- **Description:** A description of the associated preference item.

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	AISC-ASD89	Design code used for design of steel frame elements.
Time History Design	Envelopes, Step-by-Step	Envelopes	Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has *more than one* time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Frame Type	Ordinary MRF; Special MRF; Braced Frame; Special CBF; EBF	Ordinary MRF	
Zone	Zone 0, Zone 1, Zone 2, Zone 3, Zone 4	Zone 4	Seismic zone.
Omega0	≥0	2.8	
Stress Ratio Limit	>0	.95	Program will select members from the auto select list with stress ratios less than or equal to this value.
Maximum Auto Iteration	≥1	1	Sets the number of iterations of the analysis-design cycle that the program will complete automatically assuming that the frame elements have been assigned as auto select sections.

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STEEL FRAME DESIGN UBC97-LRFD

Technical Note 21
Overwrites

General

The steel frame design overwrites are basic assignments that apply only to those elements to which they are assigned. This Technical Note describes steel frame design overwrites for UBC97-LRFD. To access the overwrites, select an element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

Default values are provided for all overwrite items. Thus, you do not need to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned; that is, to the elements that are selected when the overwrites are changed.

Overwrites

For explanation purposes in this Technical Note, the overwrites are presented in Table 1. The column headings in the table are described as follows.

- **Item:** The name of the overwrite item as it appears in the program. To save space in the forms, these names are generally short.

- **Possible Values:** The possible values that the associated overwrite item can have.

- **Default Value:** The default value that the program assumes for the associated overwrite item. If the default value is given in the table with an associated note "Program Palculated," the value is shown by the program before the design is performed. After design, the values are calculated by the program and the default is modified by the program-calculated value.

- **Description:** A description of the associated overwrite item.

An explanation of how to change an overwrite is provided at the end of this Technical Note.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Current Design Section			Indicates selected member size used in current design.
Element Type	Ordinary MRF; Special MRF; Braced Frame; Special CBF; EBF	From Preferences	
Live Load Reduction Factor	≥0	1	Live load is multiplied by this factor.
Horizontal Earthquake Factor	≥0	1	Earthquake loads are multiplied by this factor.
Unbraced Length Ratio (Major)	≥0	1	Ratio of unbraced length divided by total length.
Unbraced Length Ratio (Minor, LTB)	≥0	1	Ratio of unbraced length divided by total length.
Effective Length Factor (K Major)	≥0	1	As defined in AISC-LRFD Table C-C2.1, page 6-184.
Effective Length Factor (K Minor)	≥0	1	As defined in AISC-LRFD Table C-C2.1, page 6-184.
Moment Coefficient (Cm Major)	≥0	0.85	As defined in AISC-LRFD specification Chapter C.
Moment Coefficient (Cm Minor)	≥0	0.85	As defined in AISC-LRFD specification Chapter C.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Bending Coefficient (Cb)	≥0	1	As defined in AISC-LRFD specification Chapter F.
NonSway Moment Factor (B1 Major)	≥0	1	As defined in AISC-LRFD specification Chapter C.
NonSway Moment Factor (B1 Minor)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Sway Moment Factor (B2 Major)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Sway Moment Factor (B2 Minor)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Yield stress, Fy	≥0	0	If zero, yield stress defined for material property data used.
Omega0	≥0	From Preferences	Seismic force amplification factor as required by the UBC.
Compressive Capacity, phi*Pnc	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
Tensile Capacity, phi*Pnt	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
Major Bending Capacity, phi*Mn3	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
Minor Bending Capacity, phi*Mn2	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Major Shear Capacity, phi*Vn2	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
Minor Shear Capacity, phi*Vn3	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.

Making Changes in the Overwrites Form

To access the steel frame overwrites, select a frame element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

The overwrites are displayed in the form with a column of check boxes and a two-column spreadsheet. The left column of the spreadsheet contains the name of the overwrite item. The right column of the spreadsheet contains the overwrites values.

Initially, the check boxes in the Steel Frame Design Overwrites form are all unchecked and all of the cells in the spreadsheet have a gray background to indicate that they are inactive and the items in the cells cannot be changed. The names of the overwrite items are displayed in the first column of the spreadsheet. The values of the overwrite items are visible in the second column of the spreadsheet if only one frame element was selected before the overwrites form was accessed. If multiple elements were selected, no values show for the overwrite items in the second column of the spreadsheet.

After selecting one or multiple elements, check the box to the left of an overwrite item to change it. Then left click in either column of the spreadsheet to activate a drop-down box or highlight the contents in the cell in the right column of the spreadsheet. If the drop-down box appears, select a value from the box. If the cell contents is highlighted, type in the desired value. The overwrite will reflect the change. You cannot change the values of the drop-down boxes.

When changes to the overwrites have been completed, click the **OK** button to close the form. The program then changes all of the overwrite items whose associated check boxes are checked for the selected members. You *must* click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the overwrites are ignored and the form is closed.

Resetting Steel Frame Overwrites to Default Values

Use the **Design menu > Steel Frame Design > Reset All Overwrites** command to reset all of the steel frame overwrites. All current design results will be deleted when this command is executed.

Important note about resetting overwrites: The program defaults for the overwrite items are built into the program. The steel frame overwrite values that were in a .edb file that you used to initialize your model may be different from the built-in program default values. When you reset overwrites, the program resets the overwrite values to its built-in values, not to the values that were in the .edb file used to initialize the model.

Technical Note 22
Design Load Combinations

The design load combinations are the various combinations of the load cases for which the structural members and joints need to be designed or checked. For the UBC97-LRFD code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, the following load combinations may need to be defined (UBC 2204.1, 2206, 2207.3, 2210.3, 1612.2.1):

1.4 DL	(UBC 1612.2.1 12-1)
1.2 DL + 1.4 LL	(UBC 1612.2.1 12-2)
1.2 DL ± 0.8 WL	(UBC 1612.2.1 12-3)
0.9 DL ± 1.3 WL	(UBC 1612.2.1 12-6)
1.2 DL + 0.5 LL ± 1.3 WL	(UBC 12.12.2.1 12-4)
1.2 DL ± 1.0 EL	(UBC 1612.2.1 12-5)
0.9 DL ± 1.0 EL	(UBC 1612.2.1 12-6)
1.2 DL + 0.5 LL ± EL	(UBC 1612.2.1 12-5)

These are also the default design load combinations in the program whenever the UBC97-LRFD code is used. The user should include other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading. See UBC97-LRFD Steel Frame Design Technical Note 21 Overwrites for more information.

When using the UBC97-LRFD code, the program design assumes that a P-delta analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-delta analysis be completed at the factored load level of 1.2 DL plus 0.5 LL (White and Hajjar 1991).

It is noted here that whenever **special seismic loading combinations** are required by the code for special circumstances, the program automatically generates those load combinations internally. The following additional seismic load combinations are frequently checked for specific types of members and special circumstances.

$0.9 \text{ DL} \pm \Omega_o \text{ EL}$ (UBC 2210.3, 2211.4.3.1)

$1.2 \text{ DL} + 0.5 \text{ LL} \pm \Omega_o \text{ EL}$ (UBC 2210.3, 2211.4.3.1)

where Ω_o is the seismic force amplification factor that is required to account for structural overstrength. The default value of Ω_o is taken as 2.8 in the program. However, Ω_o can be overwritten in the Preferences (**Options menu > Preferences > Steel Frame Design** command) to change the default and in the Overwrites (**Design menu > Steel Frame Design > View/Revise Overwrites** command) on a member-by-member basis. If any member is assigned a value for Ω_o, the change of Ω_o in the Preferences will not modify Ω_o of the individual member for which Ω_o has been assigned. The guidelines for selecting a reasonable value can be found in UBC 1630.3.1 and UBC Table 16-N. Other similar special loading combinations are described in UBC97-LRFD Steel Frame Design Technical Note 27 Seismic Requirements and Technical Note 28 Joint Design.

The combinations described herein are internal to the program. The user does NOT need to create additional load combinations for these load combinations. The special circumstances for which these load combinations are additionally checked are described in UBC97-LRFD Steel Frame Design Technical Note 27 Seismic Requirements and Technical Note 28 Joint Design. The special loading combination factors are applied directly to the program load cases. It is assumed that any required scaling (such as may be required to scale response spectra results) has already been applied to the progam load cases.

Reference

White, D.W. and J.F. Hajjar. 1991. Application of Second-Order Elastic Analysis in LRFD: Research to Practice. *Engineering Journal*. American Institute of Steel Construction, Inc. Vol. 28. No. 4.

Technical Note 23
Classification of Sections

This Technical Note explains the classification of sections when the user selects the UBC97-LRFD design code.

The nominal strengths for axial compression and flexure depend on the classification of the section as Compact, Noncompact, Slender or Too Slender. The section classification in UBC97-LRFD is the same as described in the AISC-LRFD93 Steel Frame Design Technical Note 47 Classification of Sections, with the exceptions described in the next paragraph. The program classifies individual members according to the limiting width/thickness ratios given in Table 1 and Table 2 of AISC-LRFD93 Technical Note 47 Classification of Sections (UBC 2204.1, 2205, 2206, and 2210; LRFD B5.1, A-G1, and Table A-F1.1). The definition of the section properties required in these tables is given in Figure 1 of AISC-LRFD93 Technical Note 47 Classification of Sections and Technical Note 43 General and Notations. The same limitations apply.

In general, the design sections need not necessarily be Compact to satisfy UBC97-LRFD codes (UBC 2213.2). However, for certain special seismic cases, they must be Compact and must satisfy special slenderness requirements. See the UBC97-LRFD Steel Frame Design Technical Note 27 Seismic Requirements. The sections that satisfy the additional requirements are classified and reported by the program as "SEISMIC." Those special requirements for classifying the sections as SEISMIC (i.e., "Compact" in UBC) are summarized herein in Table 1 (UBC 2210.8, 2210.10c, 2211.4.8.4.b, 2211.9.2.d, 2210.10g, 2211.4.10.6.e). If these criteria are not satisfied when the code requires it, the user must modify the section property. In that case, the program gives a warning message in the output file.

Table 1 Limiting Width-Thickness Ratios for Classification of Sections When Special Seismic Conditions Apply in Accordance with UBC97-LRFD

Description of Section	Width-Thickness Ratio λ	SEISMIC (Speical requirements in seismic design) (λ_p)	Section References
I-SHAPE	$b_f / 2t_f$	$\leq 52 / \sqrt{F_y}$	UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)
	h_c / t_w	For $P_u / \varphi_b P_y \leq 0.125$, $\leq \frac{520}{\sqrt{F_y}}\left(1 - 1.54\frac{P_u}{\varphi_b P_y}\right)$ For $P_u / \varphi_b P_y > 0.125$, $\leq \left\{\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\varphi_b P_y}\right) \geq \frac{253}{\sqrt{F_y}}\right\}$	UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)
BOX	b / t_f or h_c / t_w	$\leq 110 / \sqrt{F_y}$ (Beam and column in SMRF, column in SCBF, Braces in BF)	UBC 2210.8 (SMRF) UBC 2210.10.g (SCBF) UBC 2211.4.9.2.d (BF)
	b / t_f or h_c / t_w	$\leq 100 / \sqrt{F_y}$ (Braces in SCBF)	UBC 2210.10.c (SCBF)
CHANNEL	b_f / t_f h_c / t_w	Same as I-Shapes Same as I-Shapes	UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)
ANGLE	b / t	$\leq 52 / \sqrt{F_y}$ (Braces in SCBF)	UBC 2210.10.c (SCBF) UBC 2211.4.9.2.d (SCBF)
DOUBLE-ANGLE	b / t	$\leq 52 / \sqrt{F_y}$ (Braces in SCBF)	UBC 2210.10.c (SCBF) UBC 2211.4.9.2.d (SCBF)
PIPE	D/t	$\leq 1{,}300 / F_y$	UBC 2210.10.c (Braces in SCBF) UBC 2211.4.9.2.d (Braces in BF)
T-SHAPE	$b_f / 2t_f$ d / t_w	No special requirement No special requirement	
ROUND BAR	—	No special requirement	
RECTANGULAR	—	No special requirement	
GENERAL	—	No special requirement	

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Technical Note 24
Calculation of Factored Forces and Moments

This Technical Note explains how the program calculates factored forces and moments when the user selects the UBC97-LRFD code.

The factored member loads that are calculated for each load combination are P_u, M_{u33}, M_{u22}, V_{u2} and V_{u3} corresponding to factored values of the axial load, the major moment, the minor moment, the major direction shear force and the minor direction shear force, respectively. These factored loads are calculated at each of the previously defined stations for each load combination. They are calculated in the same way as described in the AISC-LRFD93 Steel Frame Design Technical Note 48 Calculation of Factored Forces and Moments without any exception (UBC 2204.1, 2205.2, 2205.3, 2206, 2210).

The bending moments are obtained along the principal directions. For I, Box, Channel, T, Double-Angle, Pipe, Circular, and Rectangular sections, the principal axes coincide with the geometric axes. For the Angle sections, the principal axes are determined and all computations related to bending moment are based on that. For general sections, it is assumed that all section properties are given in terms of the principal directions and consequently no effort is made to determine the principal directions.

The shear forces for Single-Angle sections are obtained for directions along the geometric axes. For all other sections, the shear stresses are calculated along the geometric/principal axes.

For loading combinations that cause compression in the member, the factored moment M_u (M_{u33} and M_{u22} in the corresponding directions) is magified to consider second order effects. The magnified moment in a particular direction is given by:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(LRFD C1-1, SAM 6)}$$

where

B_1 = Moment magnification factor for non-sidesway moments,

B_2 = Moment magnification factor for sidesway moments,
M_{nt} = Factored moments not causing sidesway, and
M_{lt} = Factored moments causing sidesway.

B1 is calculated as shown in AISC-LRFD93 Steel Frame Design Technical Note 48 Calculation of Factored Forces and Moments.

Similar to AISC-LRFD93, the program design assumes the analysis includes P-delta effects; therefore, B_2 is taken as unity for bending in both directions. If the program assumptions are not satisfactory for a particular structural model or member, the user has a choice of explicitly specifying the values of B_1 and B_2 for any member.

When using UBC97-LRFD code, the program design assumes that a P-delta analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-delta analysis be performed at the factored load level of 1.2 DL plus 0.5 LL (White and Hajjar 1991).

The same conditions and limitations as AISC-LRFD93 apply.

Reference

White, D.W. and J. F. Hajjar. 1991. Application of Second-Order Elastic Analysis in LRFD: Research to Practice. *Engineering Journal.* American Institute of Steel Construction, Inc. Vol. 28, No. 4.

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Technical Note 25
Calculation of Nominal Strengths

The program calculates the nominal strengths in compression, tension, bending and shear for Seismic, Compact, Noncompact, and Slender sections in accordance with UBC97-LRFD the same way as described in the AISC-LRFD93 Steel Frame Design Technical Note 49 Calculation of Nominal Strengths without any exceptions (UBC 2204.1, 2205.2, 2205.3, 2206, 2210.2, 2210.3). The nominal strengths for Seismic sections are calculated in the same way as for Compact sections.

The nominal flexural strengths for all shapes of sections, including Single-Angle sections are calculated based on their principal axes of bending. For the I, Box, Channel, Circular, Pipe, T, Double-Angle, and Rectangular sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations related to flexural strengths are based on that.

The nominal shear strengths are calculated along the geometric axes for all sections. For I, Box, Channel, T, Double-Angle, Pipe, Circular, and Rectangular sections, the principal axes coincide with their geometric axes. For Single-Angle sections, principal axes do not coincide with the geometric axes.

If the user specifies nonzero factored strengths for one or more elements in the Capacity Overwrites (accessed using the **Design menu > Steel Frame Design > Review/Revise Overwrites** command), the user-specified values will override the calculated values described herein for those elements. The specified factored strengths should be based on the principal axes of bending.

The strength reduction factor, φ, is taken as follows (LRFD A5.3):

φ_t = Resistance factor for tension, 0.9 (LRFD D1, H1, SAM 2, 6)

φ_c = Resistance factor for compression, 0.85 (LRFD E2, E3, H1)

φ_c = Resistance factor for compression in angles, 0.90 (LRFD SAM 4,6)

φ_b = Resistance factor for bending, 0.9 (LRFD F1, H1, A-F1, A-G2, SAM 5)

φ_v = Resistance factor for shear, 0.9 (LRFD F2, A-F2, A-G3, SAM 3)

All limitations and warnings related to nominal strengths calculations in AISC-LRFD93 Steel Frame Design Technical Note 49 Calculation of Nominal Strengths also apply to this code.

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Technical Note 26
Calculation of Capacity Ratios

This Technical Note describes the calculation of capacity ratios when the user selects the UBC97-LRFD code, including axial and bending stresses and shear stresses.

Overview

The capacity ratios in UBC97-LRFD are calculated in the same way as described in AISC-LRFD93 Steel Frame Design Technical Note 50 Calculation of Capacity Ratios, with some modifications as described herein.

In the calculation of the axial force/biaxial moment capacity ratios, first, for each station along the length of the member, the actual member force/moment components are calculated for each load combination. Then the corresponding capacities are calculated. Then the capacity ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling capacity ratio is then obtained, along with the associated station and load combination. A capacity ratio greater than 1.0 indicates exceeding a limit state.

During the design, the effect of the presence of bolts or welds is not considered.

Axial and Bending Stresses

The interaction ratio is determined based on the ratio $\frac{P_u}{\varphi P_n}$. If P_u is tensile, P_n is the nominal axial tensile strength and $\varphi = \varphi_t = 0.9$; and if P_u is compressive, P_n is the nominal axial compressive strength and $\varphi = \varphi_c = 0.85$, except for angle sections $\varphi = \varphi_c = 0.9$ (LRFD SAM 6). In addition, the resistance factor for bending, $\varphi_b = 0.9$.

For $\frac{P_u}{\varphi P_n} \geq 0.2$, the capacity ratio is given as

$$\frac{P_u}{\varphi P_n} + \frac{8}{9}\left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}}\right). \qquad \text{(LRFD H1-1a, SAM 6-1a)}$$

For $\frac{P_u}{\varphi P_n}$ < 0.2, the capacity ratio is given as

$$\frac{P_u}{2\varphi P_n} + \left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}}\right). \qquad \text{(LRFD H1-1b, SAM 6-1a)}$$

For circular sections, an SRSS (Square Root of Sum of Squares) combination is first made of the two bending components before adding the axial load component instead of the simple algebraic addition implied by the above formulas.

For Single-Angle sections, the combined stress ratio is calculated based on the properties about the principal axes (LRFD SAM 5.3.6). For I, Box, Channel, T, Double-Angle, Pipe, Circular, and Rectangular sections, the principal axes coincides with their geometric axes. For Single-Angles sections, principal axes are determined in the program. For general sections, it is assumed that all section properties are given in terms of the principal directions; consequently, no effort is made to determine the principal directions.

Shear Stresses

Similar to the normal stresses, from the factored shear force values and the nominal shear strength values at each station for each of the load combinations, shear capacity ratios for major and minor directions are calculated as follows:

$$\frac{V_{u2}}{\varphi_v V_{n2}}, \text{ and}$$

$$\frac{V_{u3}}{\varphi_v V_{n3}},$$

where φ_v = 0.9.

For Single-angle sections, the shear stress ratio is calculated for directions along the geometric axis. For all other sections, the shear stress is calculated along the principal axes that coincides with the geometric axes.

Technical Note 27
Seismic Requirements

This Technical Note explains the special seismic requirements checked by this program for member design, which are dependent on the type of framing used. Those requirements are described herein for each type of framing (UBC 2204.1, 2205.2, 2205.3).

The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

Ordinary Moment Frames

For this framing system, the following additional requirements are checked and reported (UBC 2210.2, 2211.4.2.2.c, 211.4.2.3.c):

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_u/\varphi P_n$ > 0.5 in columns resulting from the prescribed load combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.b, 2211.4.2.3.b, 2210.5, 2211.4.6.1).

  $0.9DL \pm \Omega_o\ EL$ (UBC 2210.3, 2211.4.3.1)

  $1.2DL + 0.5\ LL \pm \Omega_o\ EL$ (UBC 2210.3, 2211.4.3.1)

Special Moment Resisting Frames

For this framing system, the following additional requirements are checked or reported (UBC 2210.2, 2211.4.2.2.d, 2211.4.2.3.d):

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_u/\varphi P_n$ > 0.5 in columns resulting from the prescribed load combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.d, 2211.4.2.3.d, 2210.5, 2211.4.6.1).

  $0.9DL \pm \Omega_o EL$ (UBC 2210.3, 2211.4.3.1)

  $1.2DL + 0.5LL \pm \Omega_o EL$ (UBC 2210.3, 2211.4.3.1)

- In Seismic zones 3 and 4, the I-shaped beams or columns, Channel-shaped beams or columns, and Box-shaped columns are also checked for compactness criteria as described in Table 1 of UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections (UBC 2210.8, 2211.4.8.4.b, Table 2211.4.8-1). Compact I-shaped beam sections are also checked for $b_f/2t_f$ to be less than $52/\sqrt{F_y}$. Compact Channel-shaped beam and column sections are also checked for b_f/t_f to be less than $52/\sqrt{F_y}$. Compact I-shaped and Channel-shaped column sections are also checked for web slenderness h/t_w to be less than the numbers given in Table 1 of UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. Compact box-shaped column sections are also checked for b/t_f and d/t_w to be less than $110/\sqrt{F_y}$. If this criterion is satisfied, the section is reported as SEISMIC as described in Technical Note UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. If this criterion is not satisfied, the user must modify the section property

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the program checks the laterally unsupported length of beams to be less than $(2{,}500/F_y)r_y$. If the check is not satisfied, it is noted in the output (UBC 2211.4.8.8).

Braced Frames

For this framing system, the following additional requirements are checked or reported (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e):

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_u/\varphi P_n$ > 0.5 in columns as a result of the prescribed load combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e, 2210.5, 2211.4.6.1).

$0.9DL \pm \Omega_o$ EL (UBC 2210.3, 2211.4.3.1)

$1.2DL + 0.5LL \pm \Omega_o$ EL (UBC 2210.3, 2211.4.3.1)

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the maximum *l*/*r* ration of the braces is checked not to exceed $720/\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2211.4.9.2.a).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the compressive strength for braces is reduced as $0.8\varphi_c P_n$ (UBC 2211.4.9.2.b).

$$P_u \le 0.8\varphi_c P_n \quad \text{(UBC 2211.4.9.2.b)}$$

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all braces are checked to be either Compact or Noncompact according to Table 2 of AISC-LRFD93 Steel Frame Design Technical Note 47 Classification of Sections (UBC 2211.4.9.2.d). The Box and Pipe-shaped braces are also checked for compactness criteria as described in Table 1 of UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections (UBC 2211.4.9.2.d). For box sections, b/t_f and d/t_w are limited to $110/\sqrt{F_y}$; for pipe sections D/t is limited to $1{,}300/\sqrt{F_y}$. If these criteria are satisfied, the section is reported as SEISMIC as described in Technical Note UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. If these criteria are not satisfied, the user must modify the section property.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, Chevron braces are designed for 1.5 times the specified load combinations (UBC 2211.4.9.4.a.1).

Eccentrically Braced Frames

For this framing system, the program looks for and recognizes the eccentrically braced frame configuration shown in Figure 1. The following additional requirements are checked or reported for the beams, columns and braces associated with these configurations (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e).

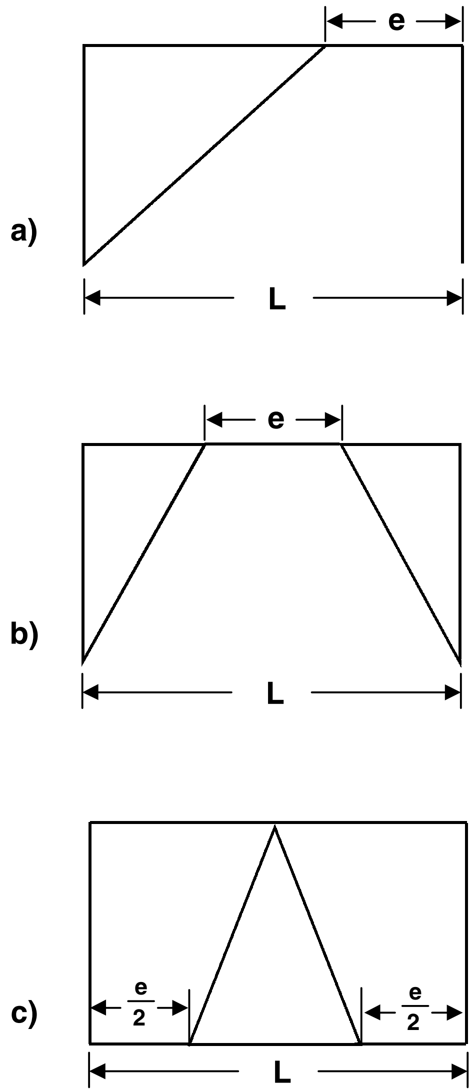


Figure 1 Eccentrically Braced Frame Configurations

- In Seismic Zone 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_u/\varphi P_n > 0.5$ in columns as a result of the prescribed load combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.b, 2211.4.2.3.b, 2210.5, 2211.4.6.1).

$$0.9DL \pm \Omega_o\ EL \qquad \text{(UBC 2210.3, 2211.4.3.1)}$$

$$1.2DL + 0.5LL \pm \Omega_o\ EL \qquad \text{(UBC 2210.3, 2211.4.3.1)}$$

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the I-shaped and Channel-shaped beams are also checked for compactness criteria as described in Table 1 of UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections (UBC 2211.4.10.2.a, 2210.8, 2211.4.8.4.b, Table 2211.4.8-1). Compact I-shaped and Channel-shaped beam sections are also checked for $b_f/2t_f$ to be less than 52 / $\sqrt{F_y}$. If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. If this criterion is not satisfied, the user must modify the section property.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the link beam yield strength, F_y, is checked not to exceed the following (UBC 2211.4.10.2.b):

$$F_y \leq 50 \text{ ksi} \qquad \text{(UBC 2211.4.10.2.b)}$$

  If the check is not satisfied, it is noted in the output.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the shear strength for link beams is taken as follows (UBC 2210.10.b, 2211.4.12.2.d):

$$V_u \leq \varphi_v V_n, \qquad \text{(UBC 2211.4.10.2.d)}$$

  where

$$\varphi V_n = \min\left(\varphi V_{pa},\ \varphi\, 2M_{pa}/e\right) \qquad \text{(UBC 2211.4.10.2.d)}$$

$$V_{pa} = V_p \sqrt{1 - \left(\frac{P_u}{P_y}\right)^2}, \qquad \text{(UBC 2211.4.10.2.f)}$$

$$M_{pa} = 1.18\, M_p \left[1 - \frac{P_u}{P_y}\right], \qquad \text{(UBC 2211.4.10.2.f)}$$

$$V_p = 0.6F_y(d - 2t_f)t_w \qquad \text{(UBC 2211.4.10.2.d)}$$

$$M_p = ZF_y \qquad \text{(UBC 2211.4.10.2.d)}$$

$$\varphi = \varphi_v \text{ (default is 0.9)} \qquad \text{(UBC 2211.4.10.2.d, 2211.4.10.2.f)}$$

$$P_y = A_gF_y \qquad \text{(UBC 2211.4.10.2.e)}$$

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, if $P_u > 0.15\, A_gF_y$, the link beam length, e, is checked not to exceed the following (UBC 2211.4.10.2.f):

$$e \le \begin{cases} \left[1.15 - 0.5\rho \dfrac{A_w}{A_g}\right] \left[1.6 \dfrac{M_p}{V_p}\right] & \textit{if} \quad \rho \dfrac{A_w}{A_g} \ge 0.3 \\ \left[1.6 \dfrac{M_p}{V_p}\right] & \textit{if} \quad \rho \dfrac{A_w}{A_g} < 0.3 \end{cases} \qquad \text{(UBC 2211.4.10.2.f)}$$

  where,

  $A_w = (d - 2t_f)t_w$, and (UBC 2211.4.10.2.f)

  $\rho = P_u/V_u$ (UBC 2211.4.10.2.f)

  If the check is not satisfied, it is noted in the output.

- The link beam rotation, θ, of the individual bay relative to the rest of the beam is calculated as the story drift delta_m times bay length divided by the total lengths of link beams in the bay. In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the link beam rotation, θ, is checked as follows (UBC 2211.4.10.2.g):

  $\theta \le 0.090$, where link beam clear length, $e \le 1.6\, M_s/V_s$

  $\theta \le 0.030$, where link beam clear length, $e \ge 2.6\, M_s/V_s$ and

  $\theta \le$ value interpolated between 0.090 and 0.030 as the link beam clear length varies from $1.6\, M_s/V_s$ to $2.6\, M_s/V_s$.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the brace strength is checked to be at least 1.25 times the axial force corresponding to the controlling link beam strength (UBC 2211.4.10.6.a). The controlling link beam nominal strength is taken as follows:

  $$\min\left(V_{pa},\ 2M_{pa}/e\right) \qquad \text{(UBC 2211.4.10.2.d)}$$

  The values of V_{pa} and M_{pa} are calculated following the procedures described above. The correspondence between brace force and link beam force is obtained from the associated load cases, whichever has the highest link beam force of interest.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the column strength is checked for 1.25 times the column forces corresponding to the controlling link beam nominal strength (UBC 2211.4.10.8). The controlling link beam strength and the corresponding forces are as obtained by the process described above.

- Axial forces in the beams are included in checking the beams. The user is reminded that using a rigid diaphragm model will result in zero axial forces in the beams. The user must disconnect some of the column lines from the diaphragm to allow beams to carry axial loads. It is recommended that only one column line per eccentrically braced frame be connected to the rigid diaphragm or that a flexible diaphragm model be used.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the beam laterally unsupported length is checked to be less than 76 $b_f/\sqrt{F_y}$. If not satisfied, it is so noted as a warning in the output file (UBC 2210.11, 2211.4.10.5).

Note: The program does **NOT** check that the strength in flexure of the beam outside the link is at least 1.25 times the moment corresponding to the controlling link beam strength (UBC 2211.4.10.6.b). Users need to check for this requirement.

Special Concentrically Braced Frames

For this framing system, the following additional requirements are checked or reported (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e):

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_u/\varphi P_n$ > 0.5 in columns as a result of the prescribed load combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e, 2210.5, 2211.4.6.1):

  $0.9\ DL \pm \Omega_0 EL$ (UBC 2210.2, 2211.4.3.1)

  $1.2\ DL + 0.5\ LL \pm \Omega_0 EL$ (UBC 2210.3, 2211.4.3.1)

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all columns are checked to be Compact in accordance with Table 2 in AISC-LRFD93 Steel Frame Design Technical Note 47 Classification of Section. Compact box-shaped column sections are also checked for b/t_f and d/t_w to be less than $100/\sqrt{F_y}$ as described in Table 1 in UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections (UBC 2211.4.12.5.a). If this criterion is satisfied, the section is reported as SEISMIC as described in UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. If this criterion is not satisfied, the user must modify the section property (UBC 2210.10.g, 2211.4.12.5.a).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all braces are checked to be Compact in accordance with Table 2 in AISC-LRFD93 Steel Frame Design Technical Note 47 Classification of Section (UBC 2210.10.c, 2211.4.12.2.d). The Angle-, Double-Angle, Box- and Pipe-shaped braces are also checked for compactness criteria as described in Table 1 in UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections (UBC 2210.10.c, 2211.4.12.2.d). For box sections b/t_f and d/t_w are limited to $100/\sqrt{F_y}$; for pipe sections, D/t is limited to $1{,}300/F_y$. If these criteria are satisfied, the section is reported as SEISMIC as described in UBC97-LFRD Steel Frame Design Technical Note 23 Classification of Sections. If these criteria are not satisfied, the user must modify the section property.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the compressive strength for braces is taken as $\varphi_c P_n$ (UBC 2210.10.b, 1122.4.12.2.b). Unlike Braced Frames, no reduction is required.

  $P_u \le \varphi_c P_n$ (UBC 2211.4.12.2.b)

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the maximum l/r ratio of the braces is checked not to exceed 1,000/$\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2210.10.a, 2211.4.12.2.a).

  Note: Beams intersected by Chevron braces are **NOT** currently checked to have a strength to support loads represented by the following combinations (UBC 2213.9.4.1):

  $1.0DL + 0.7LL \pm P_b$ (UBC 2210.10.e, 2211.4.12.4.a.3)

  $0.9DL \pm P_b$ (UBC 2210.10.e, 2211.4.12.4.a.3)

  where P_b is given by the difference of F_yA for the tension brace and $0.3\varphi_c P_n$ for the compression brace. Users need to check for this requirement.

Technical Note 28
Joint Design

When using UBC97-LRFD design code, the structural joints are checked or designed for the following:

- Check for the requirement of continuity plate and determination of its area (see UBC97-LRFD Steel Frame Design Technical Note 29 Continuity Plates)
- Check for the requirement of doubler plate and determination of its thickness (see UBC97-LRFD Steel Frame Design Technical Note 30 Doubler Plates)
- Check for the ratio of beam flexural strength to column flexural strength
- Reporting the beam connection shear
- Reporting the brace connection force

Weak-Beam / Strong-Column Measure

In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the code requires that the sum of beam flexure strengths at a joint should be less than the sum of column flexure strengths (UBC 2211.4.8.6). The column flexure strength should reflect the presence of the axial force present in the column. To facilitate the review of the strong-column/weak-beam criterion, the program reports a beam/column plastic moment capacity ratio for every joint in the structure.

For the major direction of any column (top end), the beam-to-column strength ratio is obtained as:

$$R_{maj} = \frac{\sum_{n=1}^{n_b} M_{pbn} \cos \theta_n}{M_{pcax} + M_{pcbx}} \qquad \text{(UBC 2211.4.8.6 8-3)}$$

For the minor direction of any column, the beam-to-column-strength ratio is obtained as:

$$R_{min} = \frac{\sum_{n=1}^{n_b} M_{pbn} \cos \theta_n}{M_{pcay} + M_{pcby}} \qquad \text{(UBC 2211.4.8.6 8-3)}$$

where,

$R_{maj, min}$ = Plastic moment capacity ratios, in the major and minor directions of the column, respectively

M_{pbn} = Plastic moment capacity of n-th beam connecting to column

θ_n = Angle between the n-th beam and the column major direction

$M_{pcax,y}$ = Major and minor plastic moment capacities, reduced for axial force effects, of column above story level

$M_{pcbx,y}$ = Major and minor plastic moment capacities, reduced for axial force effects, of column below story level

n_b = Number of beams connecting to the column

The plastic moment capacities of the columns are reduced for axial force effects and are taken as:

$$M_{pc} = Z_c \left(F_{yc} - \left| P_{uc} / A_{gc} \right| \right), \qquad \text{(UBC 2211.4.8.6 8-3)}$$

where,

Z_c = Plastic modulus of column

F_{yc} = Yield stress of column material

P_{uc} = Maximum axial strength in column in compression, $P_{uc} \geq 0$, and

A_{gc} = Gross area of column

For the above calculations, the section of the column above is taken to be the same as the section of the column below, assuming that the column splice will be located some distance above the story level.

Evaluation of Beam Connection Shears

For each steel beam in the structure, the program will report the maximum major shears at each end of the beam for the design of the beam shear connections. The beam connection shears reported are the maxima of the factored shears obtained from the load combinations.

For special seismic design, the beam connection shears are not taken less than the following special values for different types of framing. The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Ordinary Moment Frames, the beam connection shears reported are the maximum of the specified load combinations and the following additional load combinations (UBC 2211.4.7.2.a, 2211.4.8.2.b):

  $$0.9\text{DL} \pm \Omega_0 \text{ EL} \qquad \text{(UBC 2210.3, 2211.4.3.1)}$$
  $$1.2\text{DL} + 0.5\text{LL} \pm \Omega_0 \text{ EL} \qquad \text{(UBC 2210.3, 2211.4.3.1)}$$

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Special Moment-Resisting Frames, the beam connection shears that are reported allow for the development of the full plastic moment capacity of the beam. Thus:

  $$V_u = \frac{CM_{pb}}{L} + 1.2V_{DL} + 0.5V_{LL} \qquad \text{(UBC 2211.4.8.2.b)}$$

where

V = Shear force corresponding to END I and END J of beam,

C = 0 if beam ends are pinned, or for cantilever beam,

= 1 if one end of the beam is pinned

= 2 if no ends of the beam are pinned,

M_{pb} = Plastic moment capacity of the beam, ZF_y

L = Clear length of the beam,

V_{DL} = Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the dead load only, and

V_{LL} = Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the live load only.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Eccentrically Braced Frames, the link beam connection shear is reported as equal to the link beam web shear capacity (UBC 2211.4.10.7).

Evaluation of Brace Connection Forces

For each steel brace in the structure, the program reports the maximum axial force at each end of the brace for the design of the brace-to-beam connections. The brace connection forces reported are the maxima of the factored brace axial forces obtained from the load combinations.

For special seismic design, the brace connection forces are not taken less than the following special values for different types of framing. The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for ordinary Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace (F_yA) (UBC 2211.4.9.3.a.1) and the following special load combinations (UBC 2211.4.9.3.a.2):

  $0.9\ DL \pm \Omega_0\ EL$ (UBC 2210.3, 2211.4.3.1)
  $1.2\ DL + 0.5\ LL \pm \Omega_0\ EL$ (UBC 2210.3, 2211.4.3.1)

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Eccentrically Braced Frames, the bracing connection force is reported as at least the nominal strength of the brace (UBC 221.4.10.6.d).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Special Concentrically Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace (F_yA) (UBC 2210.10, 2211.4.12.3.a.1) and the following special load combinations (UBC 2211.10, 2211.4.12.3.a.2):

  $0.9\ DL \pm \Omega_0\ EL$ (UBC 2210.3, 2211.4.3.1)
  $1.2\ DL + 0.5\ LL \pm \Omega_0\ EL$ (UBC 2210.3, 2211.4.3.1)

STEEL FRAME DESIGN UBC97-LRFD

Technical Note 29
Continuity Plates

In a plan view of a beam/column connection, a steel beam can frame into a column in the following ways:

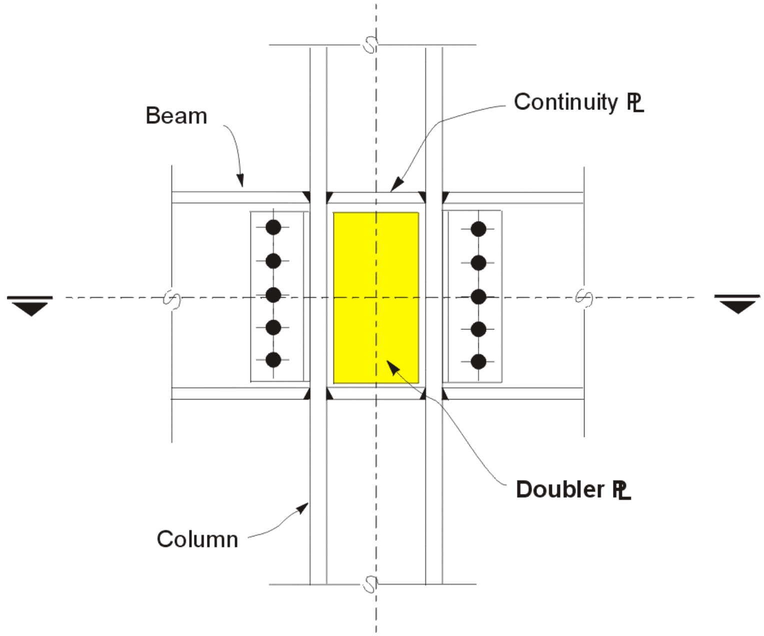
1. The steel beam frames in a direction parallel to the column major direction, i.e., the beam frames into the column flange.

2. The steel beam frames in a direction parallel to the column minor direction, i.e., the beam frames into the column web.

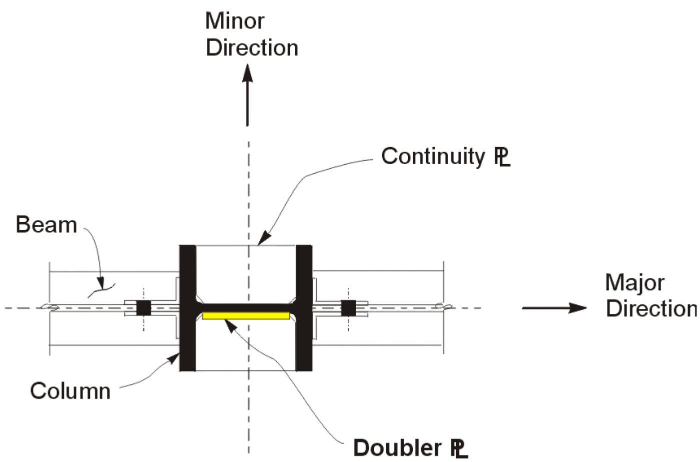
3. The steel beam frames in a direction that is at an angle to both the principal axes of the column, i.e., the beam frames partially into the column web and partially into the column flange.

To achieve a beam/column moment connection, continuity plates such as shown in Figure 1 are usually placed on the column in line with the top and bottom flanges of the beam to transfer the compression and tension flange forces from the beam into the column.

For connection conditions described in items 2 and 3 above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam. However, for the connection condition described by item 1 above, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends on the magnitude of the beam-flange force and the properties of the column. This is the condition that the program investigates. Columns of I-sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e., parallel to the column major direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.



ELEVATION



SECTION

Figure 1 Elevation and Plan of Doubler Plates for a Column of I-Section

The program first evaluates the need for continuity plates. Continuity plates will be required if **any** of the following four conditions are not satisfied:

- The column flange design strength in bending must be larger than the beam flange force, i.e.,

$$\varphi R_n = (0.9)6.25\ t_{fc}^2\ F_{yc} \geq P_{bf} \qquad \text{(LRFD K1-1)}$$

- The design strength of the column web against local yielding at the toe of the fillet must be larger than the beam flange force, i.e.,

$$\varphi R_n = (1.0)(5.0\ k_c + t_{fb})\ F_{yc} t_{wc} \geq P_{bf} \qquad \text{(LRFD K1-2)}$$

- The design strength of the column web against crippling must be larger than the beam flange force, i.e.,

$$\varphi R_n = (0.75)\ 68\ t_{wc}^2 \left[1 + 3\left(\frac{t_{fb}}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{F_{yc}\ \frac{t_{fc}}{t_{wc}}} \geq P_{bf} \qquad \text{(LRFD K1-5a)}$$

- The design compressive strength of the column web against buckling must be larger than the beam flange force, i.e.,

$$\varphi R_n = (0.9)\frac{4{,}100 t_{wc}^3 \sqrt{F_{yc}}}{d_c} \geq\ P_{bf} \qquad \text{(LRFD K1-8)}$$

If any of the conditions above are not met, the program calculates the required continuity plate area as:

$$A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{yc})}\ -\ 12\,t_{wc}^2 \qquad \text{(LRFD K1.9,E2)}$$

If $A_{cp} \leq 0$, no continuity plates are required.

The formula above assumes the continuity plate plus a width of web equal to $12t_{wc}$ act as a compression member to resist the applied load (LRFD K1.9). The formula also assumes φ = 0.85 and $F_{cr} = 0.9F_{yc}$. This corresponds to an assumption of λ = 0.5 in the column formulas (LRFD E2-2). The user should choose the continuity plate cross-section such that this is satisfied. As an example, when using F_{yc} = 50 ksi and assuming the effective length of the stiffener as a column to be 0.75h (LRFD K1.9), the required minimum radius gyration of the stiffener cross-section would be r = 0.02h to obtain λ = 0.5 (LRFD E2-4).

If continuity plates are required, they must satisfy a minimum area specification defined as follows:

- The minimum thickness of the stiffeners is taken in the program as follows:

$$t_{cp}^{\min} = \max \left\{ 0.5 t_{fb}, \frac{\sqrt{F_y}}{95} b_{fb} \right\} \qquad \text{(LRFD K1.9.2)}$$

- The minimum width of the continuity plate on each side plus 1/2 the thickness of the column web shall not be less than 1/3 of the beam flange width, or:

$$b_{cp}^{\min} = 2 \left(\frac{b_{fp}}{3} - \frac{t_{wc}}{2} \right) \qquad \text{(LRFD K1.9.1)}$$

- So that the minimum area is given by:

$$A_{cp}^{\min} = t_{cp}^{2}\, b_{cp}^{\min} \qquad \text{(LRFD K1.9.1)}$$

Therefore, the continuity plate area provided by the program is either zero or the greater of A_{cp} and $A_{cp}^{\min}$.

In the equations above,

A_{cp} = Required continuity plate area

F_{yc} = Yield stress of the column and continuity plate material

d_b = Beam depth

d_c = Column depth

h = Clear distance between flanges of column less fillets for rolled shapes

k_c = Distance between outer face of the column flange and web toe of its fillet

M_u = Factored beam moment

P_{bf} = Beam flange force, assumed as Mu / (db - tfb)

R_n = Nominal strength

t_{fb} = Beam flange thickness

t_{fc} = column flange thickness

t_{wc} = Column web thickness

φ = Resistance factor

The program also checks special seismic requirements depending on the type of frame as described below. The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

- In Seismic Zones 3 and 4 and Seismic Zone 2 with Importance factor greater than 1 for Ordinary Moment Frames the continuity plates are checked and designed for a beam flange force, $P_{bf} = M_{pb}/(d_b\text{-}t_{fb})$ (UBC 2211.4.7.2.a, 2211.4.8.2.a.1).

- In Seismic Zones 3 and 4 for Special Moment-Resisting Frames, for determining the need for continuity plates at joints as a result of tension transfer from the beam flanges, the force P_{bf} is taken as $f_{yb}A_{bf}$ for all four checks described above (LRFD K1-1, K1-2, K1-5a, K1-8), except for checking column flange design strength in bending P_{bf} is taken as 1.8 $f_{yb}A_{bf}$ (UBC 2211.4.8.5, LRFD K1-1). In Seismic Zone 2 with Importance factor greater than 1, for Special Moment-Resisting Frames, for determining the need for continuity plates at joints as a result of tension transfer from the beam flanges, the force P_{bf} is taken as $f_{yb}A_{bf}$ (UBC 2211.4.8.2.a.1)

P_{bf} = $1.8f_{yb}A_{bf}$ (Zones 3 and 4) (UBC 2211.4.8.5)

P_{bf} = $f_{yb}A_{bf}$ (Zone 2 with I>1) (UBC 2211.4.8.2.a.1)

For design of the continuity plate, the beam flange force is taken as $P_{bf} = M_{pb}/(d_b\text{-}t_{fb})$ (UBC 211.4.8.2.a.1).

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Eccentrically Braced Frames, the continuity plate requirements are checked and designed for beam flange force of $P_{bf} = f_{yb}A_{bg}$.

STEEL FRAME DESIGN UBC97-LRFD

Technical Note 30
Doubler Plates

One aspect of the design of a steel framing system is an evaluation of the shear forces that exist in the region of the beam column intersection known as the panel zone.

Shear stresses seldom control the design of a beam or column member. However, in a Moment-Resisting frame, the shear stress in the beam-column joint can be critical, especially in framing systems when the column is subjected to major direction bending and the joint shear forces are resisted by the web of the column. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear stresses are seldom critical, and this condition is therefore not investigated by the program.

Shear stresses in the panel zone caused by major direction bending in the column may require additional plates to be welded onto the column web, depending on the loading and the geometry of the steel beams that frame into the column, either along the column major direction or at an angle so that the beams have components along the column major direction. See Figure 1. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements. Also, doubler plate requirements are evaluated for moment frames only. No check is made for braced frames.

The program calculates the required thickness of doubler plates using the following algorithms. The shear force in the panel zone is given by:

$$V_p = \sum_{n=1}^{n_b} \frac{M_{bn} \cos \theta_n}{d_n - t_{fn}} - V_c$$

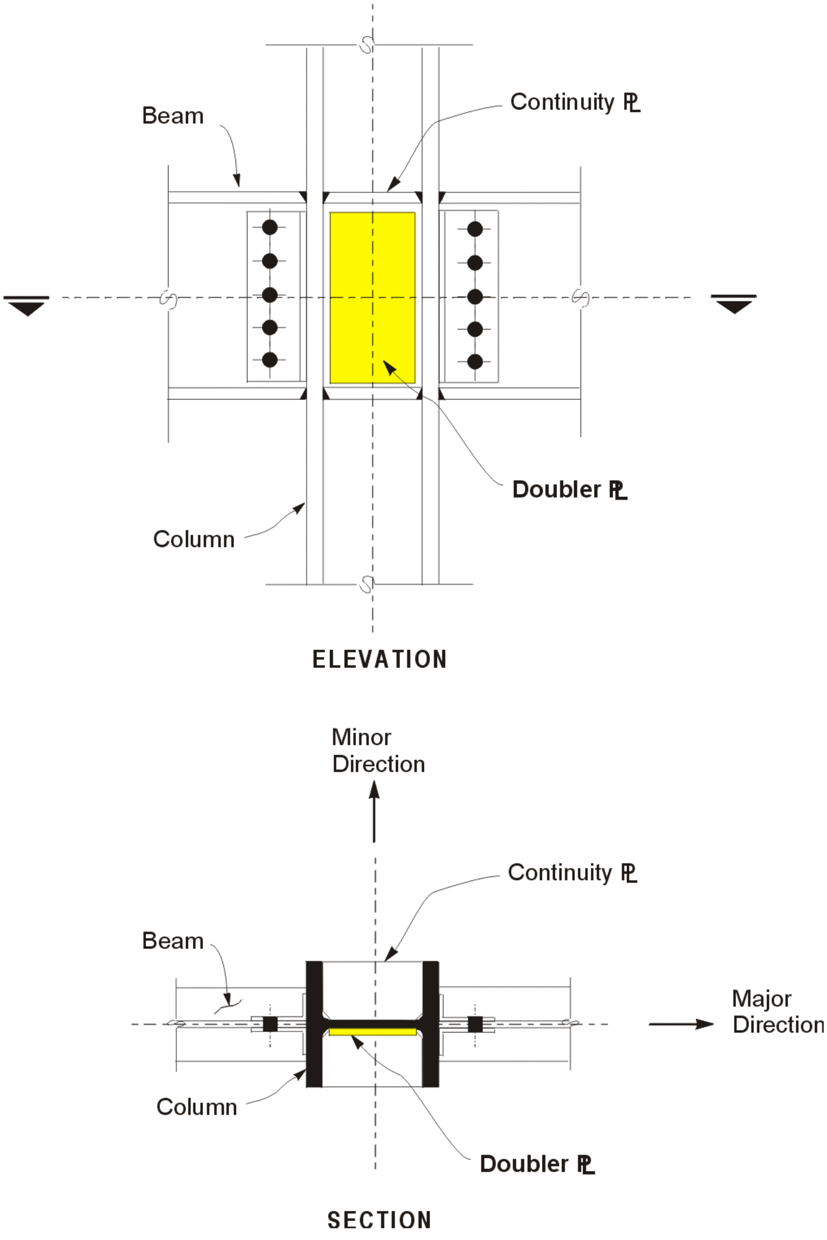


Figure 1 Elevation and Plan of Doubler Plates for a Column of I-Section

The nominal panel shear strength is given by

$$R_v = 0.6F_y d_c t_r, \text{ for } P_u \leq 0.4P_y \text{ or if } P_u \text{ is tensile, and} \qquad \text{(LRFD K1-9)}$$

$$R_v = 0.6F_y d_c t_r, \left[1.4 - \frac{P_u}{P_y}\right] \text{for } P_u > 0.4P_y \qquad \text{(LRFD K1-10)}$$

By using $V_p = \varphi R_v$, with $\varphi = 0.9$, the required column web thickness, t_r, can be found.

The extra thickness, or thickness of the doubler plate is given by

$$t_{dp} \quad = \quad t_r - t_w \geq \frac{h}{418 / \sqrt{F_y}} \qquad \text{(LFRD F2-1)}$$

where,

F_y = Column and doubler plate yield stress

t_r = Required column web thickness

t_{dp} = Required doubler plate thickness

t_w = Column web thickness

h = d_c-2t_{fc} if welded, d_c - 2k_c if rolled

V_p = Panel zone shear

V_c = Column shear in column above

F_y = Beam flange forces

n_b = Number of beams connecting to column

d_n = Depth of n-th beam connecting to column

θ_n = Angle between n-th beam and column major direction

d_c = Depth of column clear of fillets, equals d - 2k

M_{bn} = Calculated factored beam moment from the corresponding load combination

R_v = Nominal shear strength of panel

P_u = Column factored axial load

P_y = Column axial yield strength, F_yA

The program reports the largest calculated value of t_{db} for any of the load combinations based on the factored beam moments and factored column axial loads.

The special seismic requirements checked by the program for calculating doubler plate areas depend on the type of framing used; the requirements checked are described herein for each type of framing. The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2) and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

- In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the panel zone doubler plate requirements that are reported will develop the lesser of beam moments equal to 0.9 of the plastic moment capacity of the beam ($0.9\Sigma\varphi_b M_{pb}$), or beam moments resulting from specified load combinations involving seismic load (UBC 2211.4.8.3.a).

  The capacity of the panel zone in resisting this shear is taken as (UBC 2211.8.3.a):

  $$\varphi_v V_n = 0.6\varphi_v F_y d_c t_p \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right) \qquad \text{(UBC 2211.4.8.3.a)}$$

  giving the required panel zone thickness as

  $$t_p = \frac{V_p}{0.6\varphi_v F_y d_c} - \frac{3b_{cf}t_{cf}^2}{d_b d_c} \geq \frac{h}{418 / \sqrt{F_y}} \qquad \text{(UBC 2211.4.8.3, LRFD F2-1)}$$

  and the required doubler plate thickness as

$t_{dp} = t_p - t_{wc}$

where,

φ_v	=	0.75,
b_{cf}	=	width of column flange
t_{cf}	=	thickness of column flange
t_p	=	required column web thickness
h	=	d_c - 2t_{fc} if welded, d_c - 2k_c if rolled, and
d_b	=	depth of deepest beam framing into the major direction of the column.

- In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the program checks the following panel zone column web thickness requirement:

$$t_{wc} \geq \frac{(d_c - 2t_{fc}) + (d_b - 2t_{fb})}{90} \qquad \text{(UBC 2211.4.8.3.b)}$$

If the check is not satisfied, it is noted in the output.

- In Seismic Zones 3 and 4, for Eccentrically Braced Frames, the doubler plate requirements are checked similar to doubler plate checks for Special Moment-Resisting Frames, as described above (UBC 2211.4.10.7).

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STEEL FRAME DESIGN UBC97-LRFD

Technical Note 31
Input Data

This Technical Note describes the steel frame design input data for UBC97-LRFD. The input can be printed to a printer or to a text file when you click the **File menu > Print Tables > Steel Frame Design** command. A printout of the input data provides the user with the opportunity to carefully review the parameters that have been input into the program and upon which program design is based. Further information about using the Print Design Tables Form is provided at the end of this Technical Note.

Input Data

The program provides the printout of the input data in a series of tables. The column headings for input data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Material Property Data	
Material Name	Steel, concrete or other.
Material Type	Isotropic or orthotropic.
Design Type	Concrete, steel or none. Postprocessor available if steel is specified.
Material Dir/Plane	"All" for isotropic materials; specify axis properties define for orthotropic.
Modulus of Elasticity	
Poisson's Ratio	
Thermal Coeff	
Shear Modulus	
Material Property Mass and Weight	
Material Name	Steel, concrete or other.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Mass Per Unit Vol	Used to calculate self mass of the structure.
Weight Per Unit Vol	Used to calculate the self weight of the structure.
Material Design Data for Steel Materials	
Material Name	Steel.
Steel FY	Minimum yield stress of steel.
Steel FU	Maximum tensile stress of steel.
Steel Cost ($)	Cost per unit weight used in composite beam design if optimum beam size specified to be determined by cost.
Material Design Data for Concrete Materials	
Material Name	Concrete.
Lightweight Concrete	Check this box if this is a lightweight concrete material.
Concrete FC	Concrete compressive strength.
Rebar FY	Bending reinforcing yield stress.
Rebar FYS	Shear reinforcing yield stress.
Lightwt Reduc Fact	Define reduction factor if lightweight concrete box checked. Usually between 0.75 ad 0.85.
Frame Section Property Data	
Frame Section Name	User specified or auto selected member name.
Material Name	Steel, concrete or none.
Section Shape Name or Name in Section Database File	Name of section as defined in database files.
Section Depth	Depth of the section.
Flange Width Top	Width of top flange per AISC database.
Flange Thick Top	Thickness of top flange per AISC database.
Web Thick	Web thickness per AISC database.
Flange Width Bot	Width of bottom flange per AISC database.
Flange Thick Bot	Thickness of bottom flange per AISC database.
Section Area	

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Torsional Constant	
Moments of Inertia	I33, I22
Shear Areas	A2, A3
Section Moduli	S33, S22
Plastic Moduli	Z33, Z22
Radius of Gyration	R33, R22
Load Combination Multipliers	
Combo	Load combination name.
Type	Additive, envelope, absolute, or SRSS as defined in **Define > Load Combination**.
Case	Name(s) of case(s) to be included in this load combination.
Case Type	Static, response spectrum, time history, static nonlinear, sequential construction.
Factor	Scale factor to be applied to each load case.
Beam Steel Stress Check Element Information	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member section assigned.
Framing Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, ERF
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Beam Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
CM Major	As defined in AISC-LRFD specification Chapter C.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
CM Minor	As defined in AISC-LRFD specification Chapter C.
Cb Factor	As defined in AISC-LRFD specification Chapter F.
B1 Major	As defined in AISC-LRFD specification Chapter C.
B1 Minor	As defined in AISC-LRFD specification Chapter C.
B2 Major	As defined in AISC-LRFD specification Chapter C.
B2 Minor	As defined in AISC-LRFD specification Chapter C.
Beam Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier
phi*Pnc	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
phi*Pnt	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
phi*Mn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Mn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Vn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
phi*Vn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
Column Steel Stress Check Element Information	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member section assigned.
Framing Type	Ordinary MRF, Special MRF, Braced Frame, Special CBF, ERF
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ration Minor	Ratio of unbraced length divided by the total member length.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
K Major	Effective length factor.
K Minor	Effective length factor.
Column Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
CM Major	As defined in AISC-LRFD specification Chapter C.
CM Minor	As defined in AISC-LRFD specification Chapter C.
Cb Factor	As defined in AISC-LRFD specification Chapter F.
B1 Major	As defined in AISC-LRFD specification Chapter C.
B1 Minor	As defined in AISC-LRFD specification Chapter C.
B2 Major	As defined in AISC-LRFD specification Chapter C.
B2 Minor	As defined in AISC-LRFD specification Chapter C.
Column Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
phi*Pnc	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
phi*Pnt	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
phi*Mn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Mn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Vn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
phi*Vn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.

Using the Print Design Tables Form

To print steel frame design input data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Input Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design input data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

Technical Note 32
Output Details

This Technical Note describes the steel frame design output for UBC97-LRFD that can be printed to a printer or to a text file. The design output is printed when you click the **File menu > Print Tables > Steel Frame Design** command and select Output Summary on the Print Design Tables form. Further information about using the Print Design Tables form is provided at the end of this Technical Note.

The program provides the output data in a series of tables. The column headings for output data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Output Data

COLUMN HEADING	DESCRIPTION
Beam Steel Stress Check	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces the maximum load/resistance ratio.
Ratio	Ratio of acting load to available resistance.
Axl	Ratio of acting axial load to available axial resistance.
B33	Ratio of acting bending moment to available bending resistance about the 33 axis.

Table 1 Steel Frame Output Data

COLUMN HEADING	DESCRIPTION
B22	Ratio of acting bending moment to available bending resistance about the 22 axis.
Shear22	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting shear divided by available shear resistance.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear divided by available shear resistance.
Beam Special Seismic Requirements	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Section Class	Classification of section for the enveloping combo.
Connection Shear	
Combo	Name of the load combination that provides maximum End-I connection shear.
End-I	Maximum End-I connection shear.
Combo	Name of the load combination that provides maximum End-J connection shear.
End-J	Maximum End-J connection shear.

Table 1 Steel Frame Output Data

COLUMN HEADING	DESCRIPTION
Column Steel Stress Check Output	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
AXL	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Column Special Seismic Requirements	
Story Level	Story level name.

Table 1 Steel Frame Output Data

COLUMN HEADING	DESCRIPTION
Column Line	Column line identifier.
Section ID	Name of member section assigned.
Section Class	Classification of section for the enveloping combo.
Continuity Plate	
Combo	Name of load combination that produces maximum continuity plate area.
Area	Cross-section area of the continuity plate.
Doubler Plate	
Combo	Name of load combination that produces maximum doubler plate thickness.
Thick	Thickness of the doubler plate.
B/C Ratios	
Major	Beam/column capacity ratio for major direction.
Minor	Beam/column capacity ratio for minor direction.

Using the Print Design Tables Form

To print steel frame design ouput data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Output Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design output data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

STEEL FRAME DESIGN AISC-ASD89

Technical Note 33
General and Notation

Introduction to the AISC-ASD89 Series of Technical Notes

The AISC-ASD89 for Steel Frame Design series of Technical Notes describes the details of the structural steel design and stress check algorithms used by the program when the user selects the AISC-ASD89 design code (AISC 1989a). The various notations used in this series are described herein.

For referring to pertinent sections and equations of the original ASD code, a unique prefix "ASD" is assigned. However, all references to the "Specifications for Allowable Stress Design of Single-Angle Members" (AISC 1989b) carry the prefix of "ASD SAM."

The design is based on user-specified loading combinations. To facilitate use, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures. See Steel Frame Design AISC-ASD89 Technical Note 36 Design Load Combinations for more information.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this series of Technical Notes. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates overstress. Similarly, a shear capacity ratio is also calculated separately. Algorithms for completing these calculations are described in AISC-ASD89 Steel Frame Design Technical Notes 38 Calculation of Stresses, 39 Calculation of Allowable Stresses, and 40 Calculation of Stress Ratios.

Further information is available from AISC-ASD89 Steel Frame Design Technical Note 37 Classification of Sections.

The program uses preferences and overwrites, which are described in AISC-ASD89 Steel Frame Design Technical Notes 34 Preferences and 35 Overwrites. It also provides input and output data summaries, which are described in AISC-ASD89 Steel Frame Design Technical Notes 41 Input Data and 42 Output Details.

English as well as SI and MKS metric units can be used for input. But the code is based on Kip-Inch-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Kip-Inch-Second** units unless otherwise noted.

Notation

A	Cross-sectional area, in^2
A_e	Effective cross-sectional area for slender sections, in^2
A_f	Area of flange, in^2
A_g	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	Major and minor shear areas, in^2
A_w	Web shear area, dt_w, in^2
C_b	Bending Coefficient
C_m	Moment Coefficient
C_w	Warping constant, in^6
D	Outside diameter of pipes, in
E	Modulus of elasticity, ksi
F_a	Allowable axial stress, ksi
F_b	Allowable bending stress, ksi
F_{b33}, F_{b22}	Allowable major and minor bending stresses, ksi
F_{cr}	Critical compressive stress, ksi

F'_{e33}	$\dfrac{12\pi^2 E}{23(K_{33}l_{33}\ /\ r_{33})^2}$
F'_{e22}	$\dfrac{12\pi^2 E}{23(K_{22}l_{22}\ /\ r_{22})^2}$
F_v	Allowable shear stress, ksi
F_y	Yield stress of material, ksi
K	Effective length factor
K_{33}, K_{22}	Effective length K-factors in the major and minor directions
M_{33}, M_{22}	Major and minor bending moments in member, kip-in
M_{ob}	Lateral-torsional moment for angle sections, kin-in
P	Axial force in member, kips
P_e	Euler buckling load, kips
Q	Reduction factor for slender section, = Q_aQ_s
Q_a	Reduction factor for stiffened slender elements
Q_s	Reduction factor for unstiffened slender elements
S	Section modulus, in^3
S_{33}, S_{22}	Major and minor section moduli, in^3
$S_{eff,33}$,$S_{eff,22}$	Effective major and minor section moduli for slender sections, in^3
S_c	Section modulus for compression in an angle section, in^3
V_2, V_3	Shear forces in major and minor directions, kips
b	Nominal dimension of plate in a section, in longer leg of angle sections, $b_f — 2t_w$ for welded and $b_f — 3t_w$ for rolled box sections, etc.

b_e Effective width of flange, in

b_f Flange width, in

d Overall depth of member, in

f_a Axial stress, either in compression or in tension, ksi

f_b Normal stress in bending, ksi

f_{b33}, f_{b22} Normal stress in major and minor direction bending, ksi

f_v Shear stress, ksi

f_{v2}, f_{v3} Shear stress in major and minor direction bending, ksi

h Clear distance between flanges for I shaped sections ($d - 2t_f$), in

h_e Effective distance between flanges, less fillets, in

k Distance from outer face of flange to web toes of fillet, in

k_c Parameter used for classification of sections,

$$\frac{4.05}{\left[h/t_w\right]^{0.46}} \quad \textit{if } h/t_w > 70,$$

$$1 \quad \textit{if } h/t_w \le 70$$

l_{33}, l_{22} Major and minor direction unbraced member length, in

l_c Critical length, in

r Radius of gyration, in

r_{33}, r_{22} Radii of gyration in the major and minor directions, in

r_z Minimum radius of gyration for angles, in

t Thickness of a plate in I, box, channel, angle, and T sections, in

t_f Flange thickness, in

t_w	Web thickness, in
β_w	Special section property for angles, in

STEEL FRAME DESIGN AISC-ASD89

Technical Note 34
Preferences

This Technical Note describes the items in the Preferences form.

General

The steel frame design preferences in this program are basic assignments that apply to all steel frame elements. Use the **Options menu > Preferences > Steel Frame Design** command to access the Preferences form where you can view and revise the steel frame design preferences.

Default values are provided for all steel frame design preference items. Thus, it is not required that you specify or change any of the preferences. You should, however, at least review the default values for the preference items to make sure they are acceptable to you.

Using the Preferences Form

To view preferences, select the **Options menu > Preferences > Steel Frame Design.** The Preferences form will display. The preference options are displayed in a two-column spreadsheet. The left column of the spreadsheet displays the preference item name. The right column of the spreadsheet displays the preference item value.

To change a preference item, left click the desired preference item in either the left or right column of the spreadsheet. This activates a drop-down box or highlights the current preference value. If the drop-down box appears, select a new value. If the cell is highlighted, type in the desired value. The preference value will update accordingly. You cannot overwrite values in the drop-down boxes.

When you have finished making changes to the composite beam preferences, click the **OK** button to close the form. You must click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the preferences are ignored and the form is closed.

Preferences

For purposes of explanation, the preference items are presented in Table 1. The column headings in the table are described as follows:

- **Item:** The name of the preference item as it appears in the cells at the left side of the Preferences form.
- **Possible Values:** The possible values that the associated preference item can have.
- **Default Value:** The built-in default value that the program assumes for the associated preference item.
- **Description:** A description of the associated preference item.

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	AISC-ASD89	Design code used for design of steel frame elements.
Time History Design	Envelopes, Step-by-Step	Envelopes	Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has *more than one* time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.
Frame Type	Moment Frame, Braced Frame	Moment Frame	
Stress Ratio Limit	>0	0.95	Program will select members from the auto select list with stress ratios less than or equal to this value.
Maximum Auto Iteration	≥1	1	Sets the number of iterations of the analysis-design cycle that the program will complete automatically assuming that the frame elements have been assigned as auto select sections.

STEEL FRAME DESIGN AISC-ASD89

Technical Note 35
Overwrites

General

The steel frame design overwrites are basic assignments that apply only to those elements to which they are assigned. This Technical Note describes steel frame design overwrites for AISC-ASD89. To access the overwrites, select an element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

Default values are provided for all overwrite items. Thus, you do not need to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned; that is, to the elements that are selected when the overwrites are changed.

Overwrites

For explanation purposes in this Technical Note, the overwrites are presented in Table 1. The column headings in the table are described as follows.

- **Item:** The name of the overwrite item as it appears in the program. To save space in the forms, these names are generally short.

- **Possible Values:** The possible values that the associated overwrite item can have.

- **Default Value:** The default value that the program assumes for the associated overwrite item. If the default value is given in the table with an associated note "Program Calculated," the value is shown by the program before the design is performed. After design, the values are calculated by the program and the default is modified by the program-calculated value.

- **Description:** A description of the associated overwrite item.

An explanation of how to change an overwrite is provided at the end of this Technical Note.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Current Design Section			Indicates selected member size used in current design.
Element Type	Moment Frame, Braced Frame	From Preferences	
Live Load Reduction Factor	≥0	1	Live load is multiplied by this factor.
Horizontal Earthquake Factor	≥0	1	Earthquake loads are multiplied by this factor.
Unbraced Length Ratio (Major)	≥0	1	Ratio of unbraced length divided by total length.
Unbraced Length Ratio (Minor, LTB)	≥0	1	Ratio of unbraced length divided by total length.
Effective Length Factor (K Major)	≥0	1	As defined in AISC-ASD Table C-C2.1, page 5-135.
Effective Length Factor (K Minor)	≥0	1	As defined in AISC-ASD Table C-C2.1, page 5-135.
Moment Coefficient (Cm Major)	≥0	0.85	As defined in AISC-ASD, page 5-55.
Moment Coefficient (Cm Minor)	≥0	0.85	As defined in AISC-ASD, page 5-55.
Bending Coefficient (Cb)	≥0	1	As defined in AISC-ASD, page 5-47.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Yield stress, Fy	≥0	0	If zero, yield stress defined for material property data used.
Compressive stress, Fa	≥0	0	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Tensile stress, Ft	≥0	0	If zero, as defined for material property data used and AISC-ASD Chapter D.
Major Bending stress, Fb3	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Minor Bending stress, Fb2	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Major Shear stress, Fv2	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Minor Shear stress, Fv3	≥0	0	If zero, as defined for material property data used and AISC-ASD specification Chapter F.

Making Changes in the Overwrites Form

To access the steel frame overwrites, select a frame element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

The overwrites are displayed in the form with a column of check boxes and a two-column spreadsheet. The left column of the spreadsheet contains the name of the overwrite item. The right column of the spreadsheet contains the overwrites values.

Initially, the check boxes in the Steel Frame Design Overwrites form are all unchecked and all of the cells in the spreadsheet have a gray background to indicate that they are inactive and the items in the cells cannot be changed.

The names of the overwrite items are displayed in the first column of the spreadsheet. The values of the overwrite items are visible in the second column of the spreadsheet if only one frame element was selected before the overwrites form was accessed. If multiple elements were selected, no values show for the overwrite items in the second column of the spreadsheet.

After selecting one or multiple elements, check the box to the left of an overwrite item to change it. Then left click in either column of the spreadsheet to activate a drop-down box or highlight the contents in the cell in the right column of the spreadsheet. If the drop-down box appears, select a value from the box. If the cell contents is highlighted, type in the desired value. The overwrite will reflect the change. You cannot change the values of the drop-down boxes.

When changes to the overwrites have been completed, click the **OK** button to close the form. The program then changes all of the overwrite items whose associated check boxes are checked for the selected members. You *must* click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the overwrites are ignored and the form is closed.

Resetting Steel Frame Overwrites to Default Values

Use the **Design menu > Steel Frame Design > Reset All Overwrites** command to reset all of the steel frame overwrites. All current design results will be deleted when this command is executed.

Important note about resetting overwrites: The program defaults for the overwrite items are built into the program. The steel frame overwrite values that were in a .edb file that you used to initialize your model may be different from the built-in program default values. When you reset overwrites, the program resets the overwrite values to its built-in values, not to the values that were in the .edb file used to initialize the model.

STEEL FRAME DESIGN AISC-ASD89

Technical Note 36
Design Load Combinations

This Technical Note describes the default design load combinations in the program when the AISC-ASD89 code is selected.

The design load combinations are the various combinations of the load cases for which the structure needs to be checked. For the AISC-ASD89 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, the following load combinations may need to be defined (ASD A4):

DL	(ASD A4.1)
DL + LL	(ASD A4.1)
DL ± WL	(ASD A4.1)
DL + LL ± WL	(ASD A4.1)
DL ± EL	(ASD A4.1)
DL + LL ± EL	(ASD A4.1)

These are also the default design load combinations in the program when the AISC-ASD89 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

When designing for combinations involving earthquake and wind loads, allowable stresses are increased by a factor of 4/3 of the regular allowable value (ASD A5.2).

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading. See AISC-ASD89 Steel Frame Design Technical Note 35 Overwrites for more information.

STEEL FRAME DESIGN AISC-ASD89

Technical Note 37
Classification of Sections

This Technical Note explains the classification of sections when the user selects the AISC-ASD89 design code.

The allowable stresses for axial compression and flexure are dependent upon the classification of sections as either Compact, Noncompact, Slender, or Too Slender. The program classifies the individual members according to the limiting width/thickness ratios given in Table 1 (ASD B5.1, F3.1, F5, G1, A-B5-2). The definition of the section properties required in this table is given in Figure 1 and AISC-ASD89 Steel Frame Design Technical Note 33 General and Notation.

If the section dimensions satisfy the limits shown in the table, the section is classified as either Compact, Noncompact, or Slender. If the section satisfies the criteria for Compact sections, the section is classified as a Compact section. If the section does not satisfy the criteria for Compact sections but satisfies the criteria for Noncompact sections, the section is classified as a Noncompact section. If the section does not satisfy the criteria for Compact and Noncompact sections but satisfies the criteria for Slender sections, the section is classified as a Slender section. If the limits for Slender sections are not met, the section is classified as Too Slender. **Stress check of "Too Slender" sections is beyond the scope of this program.**

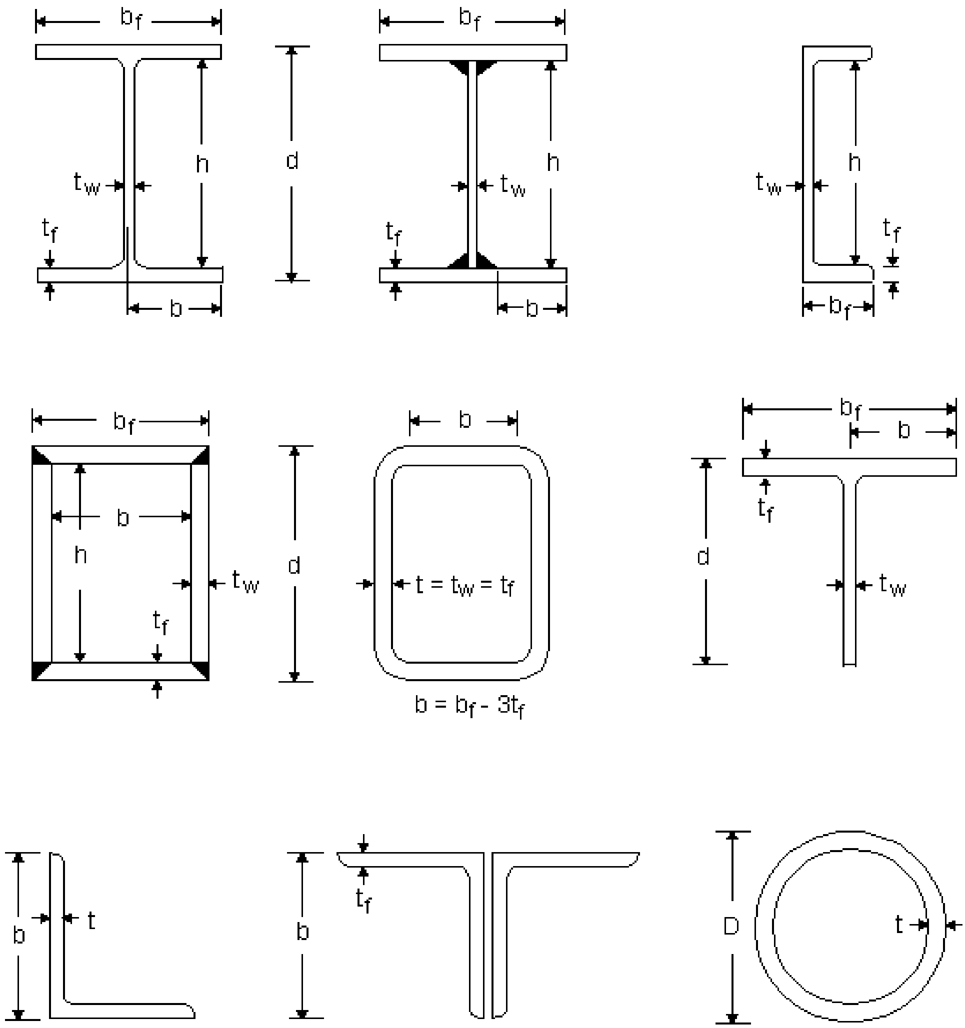
In classifying web slenderness of I-shapes, Box, and Channel sections, it is assumed that there are no intermediate stiffeners (ASD F5, G1). Double angles are conservatively assumed to be separated.

Table 1 Limiting Width-Thickness Ratios for Classification of Sections Based on AISC-ASD

<table>
<tr><th>Section Description</th><th>Ratio Check</th><th>Compact Section</th><th>Noncompact Section</th><th>Slender Section</th></tr>
<tr><td rowspan="4">I-SHAPE</td><td>$b_f / 2t_f$ (rolled)</td><td>$\leq 65 / \sqrt{F_y}$</td><td>$\leq 95 / \sqrt{F_y}$</td><td>No limit</td></tr>
<tr><td>$b_f / 2t_f$ (welded)</td><td>$\leq 65 / \sqrt{F_y}$</td><td>$\leq 95 / \sqrt{F_y / k_c}$</td><td>No limit</td></tr>
<tr><td>d / t_w</td><td>For $f_a / F_y \leq 0.16$
$\leq \frac{640}{\sqrt{F_y}}(1-3.74\frac{f_a}{F_y}),$
For $f_a / F_y > 0.16$
$\leq 257 / \sqrt{F_y}$</td><td>No limit</td><td>No limit</td></tr>
<tr><td>h / t_w</td><td>No limit</td><td>If compression only,
$\leq 253 / \sqrt{F_y}$
otherwise
$\leq 760 / \sqrt{F_b}$</td><td>If compression only,
$\leq \frac{14{,}000}{\sqrt{F_y(F_y + 16.5)}}$
≤ 260</td></tr>
<tr><td rowspan="4">BOX</td><td>b / t_f</td><td>$\leq 190 / \sqrt{F_y}$</td><td>$\leq 238 / \sqrt{F_y}$</td><td>No limit</td></tr>
<tr><td>d / t_w</td><td>As for I-shapes</td><td>No limit</td><td>No limit</td></tr>
<tr><td>h / t_w</td><td>No limit</td><td>As for I-shapes</td><td>As for I-shapes</td></tr>
<tr><td>Other</td><td>$t_w \geq t_f /2$, $d_w \leq 6b_f$</td><td>None</td><td>None</td></tr>
<tr><td rowspan="4">CHANNEL</td><td>b / t_f</td><td>As for I-shapes</td><td>As for I-shapes</td><td>No limit</td></tr>
<tr><td>d / t_w</td><td>As for I-shapes</td><td>No limit</td><td>No limit</td></tr>
<tr><td>h / t_w</td><td>No limit</td><td>As for I-shapes</td><td>As for I-shapes</td></tr>
<tr><td>Other</td><td>No limit</td><td>No limit</td><td>If welded
$b_f / d_w \leq 0.25$,
$t_f / t_w \leq 3.0$
If rolled
$b_f / d_w \leq 0.5$,
$t_f / t_w \leq 2.0$</td></tr>
</table>

Table 1 Limiting Width-Thickness Ratios for Classification of Sections Based on AISC-ASD *(continued)*

Section Description	Ratio Check	Compact Section	Noncompact Section	Slender Section
T-SHAPE	$b_f / 2t_f$	$\leq 65 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y}$	No limit
	d / t_w	Not applicable	$\leq 127 / \sqrt{F_y}$	No limit
	Other	No limit	No limit	If welded $b_f / d_w \geq 0.5$, $t_f / t_w \geq 1.25$ If rolled $b_f / d_w \geq 0.5$, $t_f / t_w \geq 1.10$
DOUBLE ANGLES	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
ANGLE	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
PIPE	D / t	$\leq 3{,}300 / F_y$	$\leq 3{,}300 / F_y$	$\leq 3{,}300 / F_y$ (Compression only) No limit for flexure
ROUND BAR	—	Assumed Compact		
RECTANGLE	—	Assumed Noncompact		
GENERAL	—	Assumed Noncompact		



AISC-ASD89 : Axes Conventions

3-3 is the cross-section axis parallel to the flanges or the smaller leg in angle sections.

2-2 is the cross-section axis perpendicular to the flanges or the smaller leg in angle sections.

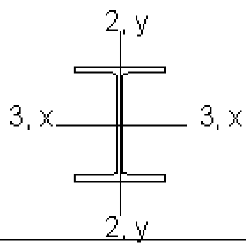


Figure 1 AISC-ASD Definition of Geometric Properties

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Technical Note 38
Calculation of Stresses

This Technical Note explains how the program calculates the stresses at each defined station. The member stresses for non-slender sections that are calculated for each load combination area, in general, based on the gross cross-sectional properties, as follows:

$$f_a = P/A$$
$$f_{b33} = M_{33}/S_{33}$$
$$f_{b22} = M_{22}/S_{22}$$
$$f_{v2} = V_2/A_{v2}$$
$$f_{v3} = V_3/A_{v3}$$

If the section is slender with slender stiffened elements, such as a slender web in I, Channel, and Box sections or slender flanges in Box sections, the program uses effective section moduli based on reduced web and reduced flange dimensions in calculating stresses, as follows:

$$f_a = P/A \qquad \text{(ASD A-B5.2d)}$$
$$f_{b33} = M_{33}/S_{eff,33} \qquad \text{(ASD A-B5.2d)}$$
$$f_{b22} = M_{22}/S_{eff,22} \qquad \text{(ASD A-B5.2d)}$$
$$f_{v2} = V_2/A_{v2} \qquad \text{(ASD A-B5.2d)}$$
$$f_{v3} = V_3/A_{v3} \qquad \text{(ASD A-B5.2d)}$$

The flexural stresses are calculated based on the properties about the principal axes. For I, Box, Channel, T, Double-angle, Pipe, Circular and Rectangular sections, the principal axes coincide with the geometric axes. For Single-angle sections, the design considers the principal properties. For general sections, it is assumed that all section properties are given in terms of the principal directions.

For Single-angle sections, the shear stresses are calculated for directions along the geometric axes. For all other sections, the program calculates the shear stresses along the geometric and principle axes.

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Technical Note 39
Calculation of Allowable Stresses

This Technical Note explains how the program calculates the allowable stresses in compression, tension, bending, and shear for Compact, Noncompact, and Slender sections. The allowable flexural stresses for all shapes of sections are calculated based on their principal axes of bending. For the I, Box, Channel, Circular, Pipe, T, Double-angle and Rectangular sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations related to flexural stresses are based on that.

If the user specifies nonzero allowable stresses for one or more elements in the Steel Frame Design Overwrites form (display using the **Design menu > Steel Frame Design > Review/Revise Overwrites** command), the nonzero values **will be used rather than the calculated values for those elements.** The specified allowable stresses should be based on the principal axes of bending.

Allowable Stress in Tension

The allowable axial tensile stress value F_a is assumed to be 0.60 F_y.

$$F_a = 0.6\ F_y \qquad \text{(ASD D1, ASD SAM 2)}$$

It should be noted that net section checks are not made. For members in tension, if l/r is greater than 300, a message to that effect is printed (ASD B7, ASD SAM 2). For single angles, the minimum radius of gyration, r_z is used instead of r_{22} and r_{33} in computing l/r.

Allowable Stress in Compression

The allowable axial compressive stress is the minimum value obtained from flexural buckling and flexural-torsional buckling. The allowable compressive stresses are determined according to the following subsections.

For members in compression, if Kl/r is greater than 200, a warning message is printed (ASD B7, ASD SAM 4). For single angles, the minimum radius of gyration, r_z, is used instead of r_{22} and r_{33} in computing Kl/r.

Flexural Buckling

The allowable axial compressive stress value, F_a, depends on the slenderness ratio Kl/r based on gross section properties and a corresponding critical value, C_c, where

$$\frac{Kl}{r} = \max\left\{\frac{K_{33}l_{33}}{r_{33}}, \frac{K_{22}l_{22}}{r_{22}}\right\}, \quad \text{and}$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}. \qquad \text{(ASD E2, ASD SAM 4)}$$

For single angles, the minimum radius of gyration, r_z, is used instead of r_{22} and r_{33} in computing Kl/r.

For Compact or Noncompact sections, F_a is evaluated as follows:

$$F_a = \frac{\left\{1.0 - \frac{(Kl/r)^2}{2C_c^2}\right\}F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}}, \qquad \text{if } \frac{Kl}{r} \le C_c, \qquad \text{(ASD E2-1, SAM 4-1)}$$

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2}, \qquad \text{if } \frac{Kl}{r} > C_c. \qquad \text{(ASD E2-2, SAM 4-2)}$$

If Kl/r is greater than 200, the calculated value of F_a is taken not to exceed the value of F_a, calculated by using the equation ASD E2-2 for Compact and Noncompact sections (ASD E1, B7).

For Slender sections, except slender Pipe sections, F_a is evaluated as follows:

$$F_a = Q\frac{\left\{1.0 - \frac{(Kl/r)^2}{2C'^2_c}\right\}F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C'_c} - \frac{(Kl/r)^3}{8C'^3_c}}, \quad \text{if } \frac{Kl}{r} \le C'_c \qquad \text{(ASD A-B5-11, SAM 4-1)}$$

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2}\ , \qquad \text{if } \frac{Kl}{r} > C'_c. \qquad \text{(ASD A-B5-12, SAM 4-2)}$$

where,

$$C'_c = \sqrt{\frac{2\pi^2 E}{QF_y}}\ . \qquad \text{(ASD A-B5.2c, ASD SAM 4)}$$

For slender sections, if *Kl/r* is greater than 200, the calculated value of F_a is taken not to exceed its value calculated by using the equation ASD A-B5-12 (ASD B7, E1).

For slender Pipe sections, F_a is evaluated as follows:

$$F_a = \frac{662}{D/t} + 0.40F_y \qquad \text{(ASD A-B5-9)}$$

The reduction factor, Q, for all compact and noncompact sections is taken as 1. For slender sections, Q is computed as follows:

$Q = Q_s Q_a$, where (ASD A-B5.2.c, SAM 4)

Q_s = reduction factor for unstiffened slender elements, and(ASD A-B5.2.a)

Q_a = reduction factor for stiffened slender elements. (ASD A-B5.2.c)

The Q_s factors for slender sections are calculated as described in Table 1 (ASD A-B5.2a, ASD SAM 4). The Q_a factors for slender sections are calculated as the ratio of effective cross-sectional area and the gross cross-sectional area.

$$Q_a = \frac{A_e}{A_g} \qquad \text{(ASD A-B5-10)}$$

The effective cross-sectional area is computed based on effective width as follows:

$$A_e = A_g - \sum (b - b_e)t$$

where

b_e for unstiffened elements is taken equal to b, and b_e for stiffened elements is taken equal to or less than b, as given in Table 2 (ASD A-B5.2b). For webs in I, box, and Channel sections, h_e is used as b_e and h is used as b in the above equation.

Flexural-Torsional Buckling

The allowable axial compressive stress value, F_a, determined by the limit states of torsional and flexural-torsional buckling, is determined as follows (ASD E3, C-E3):

$$F_a = Q \frac{\left\{1.0 - \frac{(Kl/r)_e{}^2}{2C'^2_c}\right\} F_y}{\frac{5}{3} + \frac{3(Kl/r)_e}{8C'_c} - \frac{(Kl/r)_e{}^3}{8C'^3_c}}, \quad \text{if } (Kl/r)_e \le C'_c \qquad \text{(E2-1, A-B5-11)}$$

$$F_a = \frac{12\pi^2 E}{23(Kl/r)_e{}^2}, \qquad \text{if } (Kl/r)_e > C'_c. \qquad \text{(E2-2, A-B5-12)}$$

where,

$$C'_c = \sqrt{\frac{2\pi^2 E}{QF_y}}, \text{ and} \qquad \text{(ASD E2, A-B5.2c, SAM 4)}$$

$$(Kl/r)_e = \sqrt{\frac{\pi^{2E}}{F_e}}. \qquad \text{(ASD C-E2-2, SAM 4-4)}$$

Table 1 Reduction Factor for Unstiffened Slender Elements, Q_s

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0 & \text{if } b_f/2t_f \leq 95/\sqrt{F_y/k_c}, \\ 1{,}293 - 0.00309[b_f/2t_f]\sqrt{F_y/k_c} & \text{if } 95/\sqrt{F_y/k_c} < b_f/2t_f < 195/\sqrt{F_y/k_c}, \\ 26{,}200k_c / \{[b_f/2t_f]^2 F_y\} & \text{if } b_f/2t_f \geq 195/\sqrt{F_y/k_c}. \end{cases}$	ASD A-B5-3, ASD A-B5-4
BOX	$Q_s = 1$	ASD A-B5.2c
CHANNEL	As for I-shapes with $b_f/2t_f$ replaced by b_f/t_f	ASD A-B5-3, ASD A-B5-4
T-SHAPE	For flanges, as for flanges in I-shapes. For web, see below. $Q_s = \begin{cases} 1.0 & \text{if } d/t_w \leq 127/\sqrt{F_y}, \\ 1.908 - 0.00715[d/tw]\sqrt{F_y} & \text{if } 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}, \\ 20{,}000 / \{[d/tw]^2 Fy\} & \text{if } d/t_w \geq 176/\sqrt{F_y}. \end{cases}$	ASD A-B5-3, ASD A-B5-4, ASD A-B5-5, ASD A-B5-6
DOUBLE-ANGLE	$Q_s = \begin{cases} 1.0 & \text{if } b/t \leq 76/\sqrt{F_y}, \\ 1.340 - 0.00447[b/t]\sqrt{F_y} & \text{if } 76/\sqrt{F_y} < d/t < 155/\sqrt{F_y}, \\ 15{,}500 / \{[b/t]^2 Fy\} & \text{if } d/t \geq 155/\sqrt{F_y}. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
ANGLE	$Q_s = \begin{cases} 1.0 & \text{if } b/t \leq 76/\sqrt{F_y}, \\ 1.340 - 0.00447[d/t]\sqrt{F_y} & \text{if } 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500 / \{[d/t]^2 Fy\} & \text{if } b/t \geq 155/\sqrt{F_y}. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
PIPE	$Q_s = 1$	ASD A-B5.2c
ROUND BAR	$Q_s = 1$	ASD A-B5.2c
RECTANGULAR	$Q_s = 1$	ASD A-B5.2c
GENERAL	$Q_s = 1$	ASD A-B5.2c

Table 2 Effective Width for Stiffened Sections

Section Type	Effective Width for Stiffened Sections	Equation Reference
I-SHAPE	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253 t_w}{\sqrt{f}}\left[1 - \dfrac{44.3}{(h/\ t_w)\sqrt{f}}\right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only $f = \dfrac{P}{A_g}$)	ASD A-B5-8
BOX	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253 t_w}{\sqrt{f}}\left[1 - \dfrac{44.3}{(h/\ t_w)\sqrt{f}}\right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only $f = \dfrac{P}{A_g}$) $b_e = \begin{cases} b, & \text{if } \dfrac{b}{t_f} \le \dfrac{183.74}{\sqrt{f}}, \\ \dfrac{253 t_w}{\sqrt{f}}\left[1 - \dfrac{50.3}{(h/\ t_w)\sqrt{f}}\right], & \text{if } \dfrac{b}{t} > \dfrac{183.74}{\sqrt{f}}. \end{cases}$ (compr. flexure $f = 0.6F_y$)	ASD A-B5-8 ASD A-B5-7
CHANNEL	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253 t_w}{\sqrt{f}}\left[1 - \dfrac{44.3}{(h/\ t_w)\sqrt{f}}\right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only $f = \dfrac{P}{A_g}$)	ASD A-B5-8
T-SHAPE	$b_e = b$	ASD A-B5.2c
DOUBLE-ANGLE	$b_e = b$	ASD A-B5.2c
ANGLE	$b_e = b$	ASD A-B5.2c
PIPE	$Q_a = 1$, (However, special expression for allowable axial stress is given)	ASD A-B5-9
ROUND BAR	Not applicable	—
RECTAN-GULAR	$b_e = b$	ASD A-B5.2C
GENERAL	Not applicable	—

Note: A reduction factor of 3/4 is applied on *f* for axial-compression-only cases and if the load combination includes any wind load or seismic load (ASD A-B5.2b).

ASD Commentary (ASD C-E3) refers to the 1986 version of the AISC-LRFD code for the calculation of F_e. The 1993 version of the AISC-LRFD code is the same as the 1986 version in this respect. F_e is calculated in the program as follows:

- For Rectangular, I, Box, and Pipe sections:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z l_z)^2} + GJ \right] \frac{1}{I_{22} + I_{33}} \qquad \text{(LRFD A-E3-5)}$$

- For T-sections and Double-angles:

$$F_e = \left(\frac{F_{e22} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4 F_{e22} F_{ez} H}{(F_{e22} + F_{ez})^2}} \right] \qquad \text{(LRFD A-E3-6)}$$

- For Channels:

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4 F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \right] \qquad \text{(LRFD A-E3-6)}$$

- For Single-angle sections with equal legs:

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4 F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \right] \qquad \text{(ASD SAM C-C4-1)}$$

- For Single-angle sections with unequal legs, F_e is calculated as the minimum real root of the following cubic equation (ASD SAM C-C4-2, LRFD A-E3-7):

$$(F_e\text{-}F_{e33})(F_3\text{-}F_{e22})(F_e\text{-}F_{ez})\text{-}F_e^{\,2}(F_e\text{-}F_{e22}),\frac{x_o^2}{r_o^2}\text{-}F_e^{\,2}(F_e\text{-}F_{e33})\frac{y_o^2}{r_o^2}=0,$$

where,

x_o, y_o are the coordinates of the shear center with respect to the centroid, x_o = 0 for double-angle and T-shaped members (y-axis of symmetry),

$$r_o = \sqrt{x_o^2 + y_o^2 + \frac{I_{22} + I_{33}}{A_g}}$$ = polar radius of gyration about the shear center,

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{r_o^2} \right), \quad \text{(LRFD A-E3-9)}$$

$$F_{e33} = \frac{\pi^2 E}{(K_{33} l_{33} / r_{33})^2}, \quad \text{(LRFD A-E3-10)}$$

$$F_{e22} = \frac{\pi^2 E}{(K_{22} l_{22} / r_{22})^2}, \quad \text{(LRFD A-E3-11)}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(K_z l_z)^2} + GJ \right] \frac{1}{A r_o^2}, \quad \text{(LRFD A-E3-12)}$$

K_{22}, K_{33} are effective length factors in minor and major directions,

K_z is the effective length factor for torsional buckling, and it is taken equal to K_{22} in the program,

l_{22}, l_{33} are effective lengths in the minor and major directions,

l_z is the effective length for torsional buckling, and it is taken equal to l_{22}.

For angle sections, the principal moment of inertia and radii of gyration are used for computing F_e (ASD SAM 4). Also, the maximum value of Kl, i.e, max($K_{22}l_{22}$, $K_{33}l_{33}$) , is used in place of $K_{22}l_{22}$ or $K_{33}l_{33}$ in calculating F_{e22} and F_{e33} in this case.

Allowable Stress in Bending

The allowable bending stress depends on the following criteria: the geometric shape of the cross-section; the axis of bending; the compactness of the section; and a length parameter.

I-Sections

For I-sections the length parameter is taken as the laterally unbraced length, l_{22}, which is compared to a critical length, l_c. The critical length is defined as

$$l_c = \min\left\{\frac{76 b_f}{\sqrt{F_y}}, \frac{20{,}000 A_f}{d F_y}\right\} \text{, where} \qquad \text{(ASD F1-2)}$$

A_f is the area of compression flange.

Major Axis of Bending

If l_{22} is less than l_c, the major allowable bending stress for Compact and Non-compact sections is taken depending on whether the section is welded or rolled and whether f_y is less than or equal to 65 ksi or greater than 65 ksi.

For Compact sections:

F_{b33} = 0.66 F_y if $f_y \leq$ 65 ksi, (ASD F1-1)

F_{b33} = 0.60 F_y if f_y > 65 ksi. (ASD F1-5)

For Noncompact sections:

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{2t_f}\sqrt{F_y}\right) F_y \quad \text{if rolled and } f_y \leq 65 \text{ ksi,} \qquad \text{(ASD F1-3)}$$

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{2t_f}\sqrt{\frac{F_y}{k_c}}\right) F_y \quad \text{if welded and } f_y \leq 65 \text{ ksi,} \qquad \text{(ASDF1-4)}$$

F_{b33} = 0.60 F_y if f_y > 65 ksi (ASD F1-5)

If the unbraced length l_{22} is greater than l_c, then for both Compact and Non-compact I-sections the allowable bending stress depends on the l_{22}/r_T ratio.

For $\dfrac{l_{22}}{r_T} \leq \sqrt{\dfrac{102{,}000 C_b}{F_y}}$,

F_{b33} = 0.60 F_y, (ASD F1-6)

for $\sqrt{\dfrac{102{,}000 C_b}{F_y}} < \dfrac{l_{22}}{r_T} \leq \sqrt{\dfrac{510{,}000 C_b}{F_y}}$,

$$F_{b33} = \left[\frac{2}{3} - \frac{F_y (l_{22} / r_T)^2}{1{,}530{,}000 C_b}\right] F_y \leq 0.60\ F_y \text{ , and} \qquad \text{(ASD F1-6)}$$

for $\dfrac{l_{22}}{r_T} > \sqrt{\dfrac{510{,}000 C_b}{F_y}}$,

$$F_{b33} = \left[\frac{170{,}000 C_b}{(l_{22} / r_T)^2}\right] \leq 0.60\ F_y, \qquad \text{(ASD F1-7)}$$

and F_{b33} is taken not to be less than that given by the following formula:

$$F_{b33} = \frac{12{,}000 C_b}{l_{22}(d / A_f)} \leq 0.60\ F_y \qquad \text{(ASD F1-8)}$$

where,

r_T is the radius of gyration of a section comprising the compression flange and 1/3 the compression web taken about an axis in the plane of the web,

$$C_b = 1.75 + 1.05\left(\frac{M_a}{M_b}\right) + 0.3\left(\frac{M_a}{M_b}\right)^2 \leq 2.3 \text{ , where} \qquad \text{(ASD F1.3)}$$

M_a and M_b are the end moments of any unbraced segment of the member and M_a is numerically less than M_b; M_a / M_b being positive for double curvature bending and negative for single curvature bending. Also, if any moment within the segment is greater than M_b, C_b is taken as 1.0. Also, C_b is taken as 1.0 for cantilevers and frames braced against joint translation (ASD F1.3). The program defaults C_b to 1.0 if the unbraced length, l_{22}, of the member is redefined by the user (i.e., it is not equal to the length of the member). The user can overwrite the value of C_b for any member by specifying it.

The allowable bending stress for Slender sections bent about their major axis is determined in the same way as for a Noncompact section. Then the following additional considerations are taken into account.

If the web is slender, the previously computed allowable bending stress is reduced as follows:

$$F'_{b33} = R_{PG} R_e F_{b33}, \text{ where} \qquad \text{(ASD G2-1)}$$

$$R_{PG} = 1.0 - 0.0005\ \frac{A_w}{A_f}\left[\frac{h}{t} - \frac{760}{\sqrt{F_{b33}}}\right] \leq 1.0, \qquad \text{(ASD G2)}$$

$$R_e = \frac{12 + (3\alpha - \alpha^3)\dfrac{A_w}{A_f}}{12 + 2\dfrac{A_w}{A_f}} \leq 1.0, \quad \text{(hybrid girders)} \qquad \text{(ASD G2)}$$

$R_e = 1.0,$ (non-hybrid girders) (ASD G2)

A_w = Area of web, in^2,

A_f = Area of compression flange, in^2,

$$\alpha = \frac{0.6F_y}{F_{b33}} \leq 1.0 \qquad \text{(ASD G2)}$$

F_{b33}=Allowable bending stress assuming the section is non-compact, and

F'_{b33}=Allowable bending stress after considering web slenderness.

In the above expressions, R_e is taken as 1, because currently the program deals with only non-hybrid girders.

If the flange is slender, the previously computed allowable bending stress is taken to be limited, as follows.

$F'_{b33} \leq Q_s\,(0.6\ F_y)$, where (ASD A-B5.2a, A-B5.2d)

Q_s is defined earlier.

Minor Axis of Bending

The minor direction allowable bending stress F_{b22} is taken as follows:

For Compact sections:

$F_{b22} = 0.75\ F_y$ if $f_y \leq 65$ ksi, (ASD F2-1)

$F_{b22} = 0.60\ F_y$ if $f_y > 65$ ksi. (ASD F2-2)

For Noncompact and Slender sections:

$$F_{b22} = \left(1.075 - 0.005 \frac{b_f}{2t_f} \sqrt{F_y}\right) F_y, \quad \text{if } f_y \leq 65 \text{ ksi}, \qquad \text{(ASD F2-3)}$$

$$F_{b22} = 0.60\, F_y \qquad \text{if } f_y > 65 \text{ ksi}. \qquad \text{(ASD F2-2)}$$

Channel Sections

For Channel sections, the length parameter is taken as the laterally unbraced length, l_{22}, which is compared to a critical length, l_c. The critical length is defined as

$$l_c = \min\left\{\frac{76 b_f}{\sqrt{F_y}}, \frac{20{,}000 A_f}{d F_y}\right\}, \text{ where} \qquad \text{(ASD F1-2)}$$

A_f is the area of compression flange.

Major Axis of Bending

If l_{22} is less than l_c, the major allowable bending stress for Compact and Noncompact sections is taken depending on whether the section is welded or rolled and whether f_y is greater than 65 ksi or not.

For Compact sections:

$$F_{b33} = 0.66\, F_y \qquad \text{if } f_y \leq 65 \text{ ksi}, \qquad \text{(ASD F1-1)}$$

$$F_{b33} = 0.60\, F_y \qquad \text{if } f_y > 65 \text{ ksi}. \qquad \text{(ASD F1-5)}$$

For Noncompact sections:

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{t_f} \sqrt{F_y}\right) F_y, \quad \text{if rolled and } f_y \leq 65 \text{ ksi}, \qquad \text{(ASD F1-3)}$$

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{t_f} \sqrt{\frac{F_y}{k_c}}\right) F_y, \quad \text{if welded and } f_y \leq 65 \text{ ksi}, \qquad \text{(ASD F1-4)}$$

$$F_{b33} = 0.60\, F_y \qquad \text{if } f_y > 65 \text{ ksi}. \qquad \text{(ASD F1-5)}$$

If the unbraced length l_{22} is greater than l_c, then for both Compact and Noncompact Channel sections the allowable bending stress is taken as follows:

$$F_{b33} = \frac{12{,}000 C_b}{l_{22}(d / A_f)} \le 0.60\ F_y \qquad \text{(ASD F1-8)}$$

The allowable bending stress for Slender sections bent about their major axis is determined in the same way as for a Noncompact section. Then the following additional considerations are taken into account.

If the web is slender, the previously computed allowable bending stress is reduced as follows:

$$F'_{b33} = R_e R_{PG} F_{b33} \qquad \text{(ASD G2-1)}$$

If the flange is slender, the previously computed allowable bending stress is taken to be limited as follows:

$$F'_{b33} = Q_s\ (0.60\ F_y) \qquad \text{(ASD A-B5.2a, A-B5.2d)}$$

The definitions for r_T, C_b, A_f, A_w, R_e, R_{PG}, Q_s, F_{b33}, and F'_{b33} are given earlier.

Minor Axis of Bending

The minor direction allowable bending stress F_{b22} is taken as follows:

$$F_{b22} = 0.60\ F_y \qquad \text{(ASD F2-2)}$$

T Sections and Double Angles

For T sections and Double angles, the allowable bending stress for both major and minor axes bending is taken as,

$$F_b = 0.60\ F_y$$

Box Sections and Rectangular Tubes

For all Box sections and Rectangular tubes, the length parameter is taken as the laterally unbraced length, l_{22}, measured compared to a critical length, l_c. The critical length is defined as

$$l_c = \max \left\{ (1{,}950 + 1{,}200 M_a / M_b) \frac{b}{F_y}, \frac{1{,}200 b}{F_y} \right\} \qquad \text{(ASD F3-2)}$$

where M_a and M_b have the same definition as noted earlier in the formula for C_b. If l_{22} is specified by the user, l_c is *taken* as $\frac{1{,}200b}{F_y}$ in the program.

Major Axis of Bending

If l_{22} is less than l_c, the allowable bending stress in the major direction of bending is taken as:

$F_{b33} = 0.66\ F_y$ (for Compact sections) (ASD F3-1)

$F_{b33} = 0.60\ F_y$ (for Noncompact sections) (ASD F3-3)

If l_{22} exceeds l_c, the allowable bending stress in the major direction of bending for both Compact and Noncompact sections is taken as:

$F_{b33} = 0.60\ F_y$ (ASD F3-3)

The major direction allowable bending stress for Slender sections is determined in the same way as for a Noncompact section. Then the following additional consideration is taken into account. If the web is slender, the previously computed allowable bending stress is reduced as follows:

$F'_{b33} = R_e R_{PG} F_{b33}$ (ASD G2-1)

The definitions for R_e, R_{PG}, F_{b33} and F'_{b33} are given earlier.

If the flange is slender, no additional consideration is needed in computing allowable bending stress. However, effective section dimensions are calculated and the section modulus is modified according to its slenderness.

Minor Axis of Bending

If l_{22} is less than l_c, the allowable bending stress in the minor direction of bending is taken as:

$F_{b22} = 0.66\ F_y$ (for Compact sections) (ASD F3-1)

$F_{b22} = 0.60\ F_y$ (for Noncompact and Slender sections) (ASD F3-3)

If l_{22} exceeds l_c, the allowable bending stress in the minor direction of bending is taken, irrespective of compactness, as:

$F_{b22} = 0.60\ F_y$ (ASD F3-3)

Pipe Sections

For Pipe sections, the allowable bending stress for both major and minor axes of bending is taken as

$F_b = 0.66\ F_y$ (for Compact sections), and (ASD F3-1)

$F_b = 0.60\ F_y$ (for Noncompact and Slender sections). (ASD F3-3)

Round Bars

The allowable stress for both the major and minor axis of bending of round bars is taken as,

$F_b = 0.75\ F_y$. (ASD F2-1)

Rectangular and Square Bars

The allowable stress for both the major and minor axis of bending of solid square bars is taken as,

$F_b = 0.75\ F_y$. (ASD F2-1)

For solid rectangular bars bent about their major axes, the allowable stress is given by

$F_b = 0.60\ F_y$, and

the allowable stress for minor axis bending of rectangular bars is taken as

$F_b = 0.75\ F_y$. (ASD F2-1)

Single-Angle Sections

The allowable flexural stresses for Single-angles are calculated based on their principal axes of bending (ASD SAM 5.3).

Major Axis of Bending

The allowable stress for major axis bending is the minimum considering the limit state of lateral-torsional buckling and local buckling (ASD SAM 5.1).

The allowable major bending stress for Single-angles for the limit state of lateral-torsional buckling is given as follows (ASD SAM 5.1.3):

$$F_{b,major} = \left[0.55 - 0.10\frac{F_{ob}}{F_y}\right]F_{ob}, \quad \text{if } F_{ob} \le F_y \qquad \text{(ASD SAM 5-3a)}$$

$$F_{b,major} = \left[0.95 - 0.50\sqrt{\frac{F}{F_{ob}}}\right]F_y, \le 0.66\ F_y \quad \text{if } F_{ob} > F_y \qquad \text{(ASD SAM 5-3b)}$$

where, F_{ob} is the elastic lateral-torsional buckling stress as calculated below.

The elastic lateral-torsional buckling stress, F_{ob}, for equal-leg angles is taken as

$$F_{ob} = C_b \, \frac{28{,}250}{l \,/\, t} \qquad \text{(ASD SAM 5-5)}$$

and for unequal-leg angles, F_{ob} is calculated as

$$F_{ob} = 143{,}100 C_b \frac{I_{\min}}{S_{major} l^2}\left[\sqrt{\beta_w^2 + 0.052(lt \,/\, r_{\min})^2} + \beta_w\right], \qquad \text{(ASD SAM 5-6)}$$

where,

t = $\min(t_w, t_f)$,

l = $\max(l_{22}, l_{33})$,

I_{min} = minor principal moment of inertia,

I_{max} = major principal moment of inertia,

S_{major} = major section modulus for compression at the tip of one leg,

r_{min} = radius of gyration for minor principal axis,

$$\beta_w = \left[\frac{1}{I_{\max}} \int_A z(w^2 + z^2)dA\right] - 2z_o, \qquad \text{(ASD SAM 5.3.2)}$$

z = coordinate along the major principal axis,

w = coordinate along the minor principal axis, and

z_o = coordinate of the shear center along the major principal axis with respect to the centroid.

β_w is a special section property for angles. It is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles (ASD SAM 5.3.2). However, for conservative design in the program, it is always taken as negative for unequal-leg angles.

In the previous expressions, C_b is calculated in the same way as is done for I sections, with the exception that the upper limit of C_b is taken here as 1.5 instead of 2.3.

$$C_b = 1.75 + 1.05 \left(\frac{M_a}{M_b}\right) + 0.3 \left(\frac{M_a}{M_b}\right)^2 \leq 1.5 \qquad \text{(ASD F1.3, SAM 5.2.2)}$$

The allowable major bending stress for Single-angles for the limit state of local buckling is given as follows (ASD SAM 5.1.1):

$$F_{b,major} = 0.66\, F_y \qquad \text{if} \qquad \frac{b}{t} \leq \frac{65}{\sqrt{F_y}}, \qquad \text{(ASD SAM 5-1a)}$$

$$F_{b,major} = 0.60\, F_y \qquad \text{if} \qquad \frac{65}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{76}{\sqrt{F_y}} \qquad \text{(ASD SAM 5-1b)}$$

$$F_{b,major} = Q(0.60\, F_y) \qquad \text{if} \qquad \frac{b}{t} > \frac{76}{\sqrt{F_y}} \qquad \text{(ASD SAM 5-1c)}$$

where,

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = slenderness reduction factor for local buckling.(ASD A-B5-2, SAM 4)

In calculating the allowable bending stress for Single-angles for the limit state of local buckling, the allowable stresses are calculated considering the fact that either of the two tips can be under compression. The minimum allowable stress is considered.

Minor Axis of Bending

The allowable minor bending stress for Single-angles is given as follows (ASD SAM 5.1.1, 5.3.1b, 5.3.2b):

$$F_{b,minor} = 0.66\, F_y \qquad \text{if} \qquad \frac{b}{t} \leq \frac{65}{\sqrt{F_y}}, \qquad \text{(ASD SAM 5-1a)}$$

$F_{b,minor} = 0.60\ F_y$ if $\frac{65}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{76}{\sqrt{F_y}}$ (ASD SAM 5-1b)

$F_{b,minor} = Q(0.60\ F_y)$ if $\frac{b}{t} > \frac{76}{\sqrt{F_y}}$ (ASD SAM 5-1c)

In calculating the allowable bending stress for Single-angles, it is assumed that the sign of the moment is such that both the tips are under compression. The minimum allowable stress is considered.

General Sections

For General sections, the allowable bending stress for both major and minor axes bending is taken as,

$F_b = 0.60\ F_y$.

Allowable Stress in Shear

The allowable shear stress is calculated along the geometric axes for all sections. For I, Box, Channel, T, Double angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes do not coincide with the geometric axes.

Major Axis of Bending

The allowable shear stress for all sections except I, Box and Channel sections is taken in the program as:

$F_v = 0.40\ F_y$ (ASD F4-1, SAM 3-1)

The allowable shear stress for major direction shears in I-shapes, boxes and channels is evaluated as follows:

$F_v = 0.40\ F_y$, if $\frac{h}{t_w} \leq \frac{380}{\sqrt{F_y}}$, and (ASD F4-1)

$F_v = \frac{C_v}{2.89} F_y \leq 0.40 F_y$, if $\frac{380}{\sqrt{F_y}} < \frac{h}{t_w} \leq 260$. (ASD F4-2)

where,

$$C_v = \begin{cases} \dfrac{45{,}000 k_v}{F_y (h / t_w)^2} & \text{if} \quad \dfrac{h}{t_w} \geq 56{,}250 \dfrac{k_v}{F_y} \\ \dfrac{190}{h/t_w} \sqrt{\dfrac{k_v}{F_y}} & \text{if} \quad \dfrac{h}{t_w} < 56{,}250 \dfrac{k_v}{F_y} \end{cases} \qquad \text{(ASD F4)}$$

$$k_v = \begin{cases} 4.00 + \dfrac{5.34}{(a / h)^2} & \text{if} \quad \dfrac{a}{h} \leq 1 \\ 5.34 + \dfrac{4.00}{(a / h)^2} & \text{if} \quad \dfrac{a}{h} > 1 \end{cases} \qquad \text{(ASD F4)}$$

t_w = Thickness of the web,

a = Clear distance between transverse stiffeners, in. Currently it is taken conservatively as the length, l_{22}, of the member in the program,

h = Clear distance between flanges at the section, in.

Minor Axis of Bending

The allowable shear stress for minor direction shears is taken as:

$$F_v = 0.40\ F_y \qquad \text{(ASD F4-1, SAM 3-1)}$$

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Technical Note 40
Calculation of Stress Ratios

This Technical Note describes how the program calculates stress ratios. In the calculation of the axial and bending stress ratios, first, for each station along the length of the member, the actual stresses are calculated for each load combination. Then the corresponding allowable stresses are calculated. Then, the stress ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling stress ratio is then obtained, along with the associated station and load combination. A stress ratio greater than 1.0 indicates an overstress.

During the design, the effect of the presence of bolts or welds is not considered.

Axial and Bending Stresses

With the computed allowable axial and bending stress values and the factored axial and bending member stresses at each station, an interaction stress ratio is produced for each of the load combinations as follows (ASD H1, H2, SAM 6):

- If f_a is compressive and f_a / F_a, > 0.15, the combined stress ratio is given by the larger of

$$\frac{f_a}{F_a} + \frac{C_{m33} f_{b33}}{\left(1 - \frac{f_a}{F'_{e33}}\right) F_{b33}} + \frac{C_{m22} f_{b22}}{\left(1 - \frac{f_a}{F'_{e22}}\right) F_{b22}}, \text{ and} \qquad \text{(ASD H1-1, SAM 6.1)}$$

$$\frac{f_a}{Q(0.60 F_y)} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{where} \qquad \text{(ASD H1-2, SAM 6.1)}$$

f_a = axial stress

f_{b33} = bending stress about the local 3-axis

f_{b22} = bending stress about the local 2-axis

F_a = allowable axial stress

F_{b33} = allowable bending stress about the local 3-axis

F_{b22} = allowable bending stress about the local 2-axis

C_{m33} and C_{m22} are coefficients representing distribution of moment along the member length.

$$C_m = \begin{cases} 1.00 & \text{if length is overwritten,} \\ 1.00 & \text{if tension member,} \\ 0.85 & \text{if sway frame,} \\ 0.6\text{-}0.4\,\dfrac{M_a}{M_b}, & \text{if nonsway, no transverse loading} \\ 0.85 & \text{if nonsway, trans. load, end restrained,} \\ 1.00 & \text{if nonsway, trans. load, end unrestrained} \end{cases} \qquad \text{(ASD H1)}$$

For sway frame, C_m = 0.85; for nonsway frame without transverse load, C_m = 0.6 - 0.4 M_a / M_b; for nonsway frame with transverse load and end restrained compression member, C_m = 0.85; and for nonsway frame with transverse load and end unrestrained compression member, C_m = 1.00 (ASD H1). In these cases, M_a / M_b is the ratio of the smaller to the larger moment at the ends of the member, M_a / M_b being positive for double curvature bending and negative for single curvature bending. When M_b is zero, C_m is taken as 1.0. The program defaults C_m to 1.0 if the unbraced length factor, l, of the member is redefined by either the user or the program, i.e., if the unbraced length is not equal to the length of the member. The user can overwrite the value of C_m for any member. C_m assumes two values, C_{m22} and C_{m33}, associated with the major and minor directions.

F'_e is given by

$$F'_e = \frac{12\pi^2 E}{23(Kl\,/\,r)^2}\,. \qquad \text{(ASD H1)}$$

A factor of 4/3 is applied on F'_e and $0.6F_y$ if the load combination includes any wind load or seismic load (ASD H1, ASD A5.2).

- If f_a is compressive and f_a / $F_a \leq 0.15$, a relatively simplified formula is used for the combined stress ratio.

$$\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}} \qquad \text{(ASD H1-3, SAM 6.1)}$$

- If f_a is tensile or zero, the combined stress ratio is given by the larger of

$$\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \text{ and} \qquad \text{(ASD H2-1, SAM 6.2)}$$

$$\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \text{ where}$$

f_a, f_{b33}, f_{b22}, F_a, F_{b33}, and F_{b22} are as defined earlier in this Technical Note. However, either F_{b33} or F_{b22} need not be less than $0.6F_y$ in the first equation (ASD H2-1). The second equation considers flexural buckling without any beneficial effect from axial compression.

For circular and pipe sections, an SRSS combination is first made of the two bending components before adding the axial load component, instead of the simple addition implied by the above formulae.

For Single-angle sections, the combined stress ratio is calculated based on the properties about the principal axis (ASD SAM 5.3, 6.1.5). For I, Box, Channel, T, Double-angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes are determined in the program. For general sections, no effort is made to determine the principal directions.

When designing for combinations involving earthquake and wind loads, allowable stresses are increased by a factor of 4/3 of the regular allowable value (ASD A5.2).

Shear Stresses

From the allowable shear stress values and the factored shear stress values at each station, shear stress ratios for major and minor directions are computed for each of the load combinations as follows:

$$\frac{f_{v2}}{F_v}, \quad \text{and}$$

$$\frac{f_{v3}}{F_v}.$$

For Single-angle sections, the shear stress ratio is calculated for directions along the geometric axis. For all other sections, the shear stress is calculated along the principle axes that coincide with the geometric axes.

When designing for combinations involving earthquake and wind loads, allowable shear stresses are increased by a factor of 4/3 of the regular allowable value (ASD A5.2).

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Technical Note 41
Input Data

This Technical Note describes the steel frame design input data for AISC-ASD89. The input can be printed to a printer or to a text file when you click the **File menu > Print Tables > Steel Frame Design** command. A printout of the input data provides the user with the opportunity to carefully review the parameters that have been input into the program and upon which program design is based. Further information about using the Print Design Tables Form is provided at the end of this Technical Note.

Input Data

The program provides the printout of the input data in a series of tables. The column headings for input data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Material Property Data	
Material Name	Steel, concrete or other.
Material Type	Isotropic or orthotropic.
Design Type	Concrete, steel or none. Postprocessor available if steel is specified.
Material Dir/Plane	"All" for isotropic materials; specify axis properties define for orthotropic.
Modulus of Elasticity	
Poisson's Ratio	
Thermal Coeff	
Shear Modulus	
Material Property Mass and Weight	
Material Name	Steel, concrete or other.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Mass Per Unit Vol	Used to calculate self mass of the structure.
Weight Per Unit Vol	Used to calculate the self weight of the structure.
Material Design Data for Steel Materials	
Material Name	Steel.
Steel FY	Minimum yield stress of steel.
Steel FU	Maximum tensile stress of steel.
Steel Cost ($)	Cost per unit weight used in composite beam design if optimum beam size specified to be determined by cost.
Material Design Data for Concrete Materials	
Material Name	Concrete.
Lightweight Concrete	Check this box if this is a lightweight concrete material.
Concrete FC	Concrete compressive strength.
Rebar FY	Bending reinforcing yield stress.
Rebar FYS	Shear reinforcing yield stress.
Lightwt Reduc Fact	Define reduction factor if lightweight concrete box checked. Usually between 0.75 ad 0.85.
Frame Section Property Data	
Frame Section Name	User specified or auto selected member name.
Material Name	Steel, concrete or none.
Section Shape Name or Name in Section Database File	Name of section as defined in database files.
Section Depth	Depth of the section.
Flange Width Top	Width of top flange per AISC database.
Flange Thick Top	Thickness of top flange per AISC database.
Web Thick	Web thickness per AISC database.
Flange Width Bot	Width of bottom flange per AISC database.
Flange Thick Bot	Thickness of bottom flange per AISC database.
Section Area	

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Torsional Constant	
Moments of Inertia	I33, I22
Shear Areas	A2, A3
Section Moduli	S33, S22
Plastic Moduli	Z33, Z22
Radius of Gyration	R33, R22
Load Combination Multipliers	
Combo	Load combination name.
Type	Additive, envelope, absolute, or SRSS as defined in **Define > Load Combination**.
Case	Name(s) of case(s) to be included in this load combination.
Case Type	Static, response spectrum, time history, static nonlinear, sequential construction.
Factor	Scale factor to be applied to each load case.
Beam Steel Stress Check Element Information	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member section assigned.
Framing Type	Moment frame or braced frame.
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Beam Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
CM Major	As defined in AISC-ASD, page 5-55.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
CM Minor	As defined in AISC-ASD, page 5-55.
Cb Factor	As defined in AISC-ASD, page 5-47.
Beam Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Fa	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Ft	If zero, as defined for material property data used and AISC-ASD Chapter D.
Fb Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fb Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Beam Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
CM Major	As defined in AISC-ASD, page 5-55.
CM Minor	As defined in AISC-ASD, page 5-55.
Cb Factor	As defined in AISC-ASD, page 5-47.
Column Steel Stress Check Element Information	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Framing Type	Moment Frame or Braced Frame
RLLF Factor	Live load reduction factor.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Column Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
CM Major	As defined in AISC-ASD, page 5-55.
CM Minor	As defined in AISC-ASD, page 5-55.
Cb Factor	As defined in AISC-ASD, page 5-47.
Column Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
Fa	If zero, yield stress defined for material property data used and AISC-ASD specification Chapter E.
Ft	If zero, as defined for material property data used and AISC-ASD Chapter D.
Fb Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fb Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Major	If zero, as defined for material property data used and AISC-ASD specification Chapter F.
Fv Minor	If zero, as defined for material property data used and AISC-ASD specification Chapter F.

Using the Print Design Tables Form

To print steel frame design input data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Input Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design input data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

STEEL FRAME DESIGN AISC-ASD89

Technical Note 42

Output Details

This Technical Note describes the steel frame design output for AISC-ASD89 that can be printed to a printer or to a text file. The design output is printed when you click the **File menu > Print Tables > Steel Frame Design** command and select Output Summary on the Print Design Tables form. Further information about using the Print Design Tables form is provided at the end of this Technical Note.

The program provides the output data in a table. The column headings for output data and a description of what is included in the columns of the table are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Beam Steel Stress Check Output	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
Axl	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Column Steel Stress Check Output	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
AXL	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.

Using the Print Design Tables Form

To print steel frame design output data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Output Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design output data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename>>** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

Technical Note 43
General and Notation

Introduction to the AISC-LRFD93 Series of Technical Notes

The AISC-LRFD93 Steel Frame Design series of Technical Notes describes the details of the structural steel design and stress check algorithms used by this program when the user selects the AISC-LRFD93 design code. The various notations used in this series are described herein.

The design is based on user-specified loading combinations. To facilitate use, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures. See AISC-LRFD93 Steel Frame Design Technical Note 46 Design Load Combinations for more information.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first, the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then, the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this Technical Note. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ratio is also calculated separately. Algorithms for completing these calculations are described in AISC-LRFD93 Steel Frame Design Technical Note 48 Calculation of Factored Forces and Moments, Technical Note 49 Calculation of Nominal Strengths, and Technical Note 50 Calculation of Capacity Ratios.

Further information is available from AISC-LRFD93 Steel Frame Design Technical Note 47 Classification of Sections.

The program uses preferences and overwrites, which are described in AISC-LRFD93 Steel Frame Design Technical Note 44 Preferences and Technical Note 45 Overwrites. It also provides input and output data summaries, which are described in AISC-LRFD93 Steel Frame Design Technical Note 51 Input Data and Technical Note 52 Output Details.

Notation

A	Cross-sectional area, in^2
A_e	Effective cross-sectional area for slender sections, in^2
A_g	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	Major and minor shear areas, in^2
A_w	Shear area, equal dt_w per web, in^2
B_1	Moment magnification factor for moments not causing sidesway
B_2	Moment magnification factor for moments causing sidesway
C_b	Bending coefficient
C_m	Moment coefficient
C_w	Warping constant, in^6
D	Outside diameter of pipes, in
E	Modulus of elasticity, ksi
F_{cr}	Critical compressive stress, ksi
F_r	Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi
F_y	Yield stress of material, ksi
G	Shear modulus, ksi
I_{22}	Minor moment of inertia, in^4
I_{33}	Major moment of inertia, in^4
J	Torsional constant for the section, in^4

K	Effective length factor
K_{33},K_{22}	Effective length K-factors in the major and minor directions
L_b	Laterally unbraced length of member, in
L_p	Limiting laterally unbraced length for full plastic capacity, in
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, in
M_{cr}	Elastic buckling moment, kip-in
M_{lt}	Factored moments causing sidesway, kip-in
M_{nt}	Factored moments not causing sidesway, kip-in
M_{n33},M_{n22}	Nominal bending strength in major and minor directions, kip-in
M_{ob}	Elastic lateral-torsional buckling moment for angle sections, kip-in
M_{r33}, M_{r22}	Major and minor limiting buckling moments, kip-in
M_u	Factored moment in member, kip-in
M_{u33}, M_{u22}	Factored major and minor moments in member, kip-in
P_e	Euler buckling load, kips
P_n	Nominal axial load strength, kip
P_u	Factored axial force in member, kips
P_y	A_gF_y, kips
Q	Reduction factor for slender section, = Q_aQ_s
Q_a	Reduction factor for stiffened slender elements
Q_s	Reduction factor for unstiffened slender elements
S	Section modulus, in^3

S_{33}, S_{22}	Major and minor section moduli, in^3
$S_{eff,33}, S_{eff,22}$	Effective major and minor section moduli for slender sections, in^3
S_c	Section modulus for compression in an angle section, in^3
V_{n2}, V_{n3}	Nominal major and minor shear strengths, kips
V_{u2}, V_{v3}	Factored major and minor shear loads, kips
Z	Plastic modulus, in^3
Z_{33}, Z_{22}	Major and minor plastic moduli, in^3
b	Nominal dimension of plate in a section, in longer leg of angle sections, $b_f - 2t_w$ for welded and $b_f - 3t_w$ for rolled box sections, etc.
b_e	Effective width of flange, in
b_f	Flange width, in
d	Overall depth of member, in
d_e	Effective depth of web, in
h_c	Clear distance between flanges less fillets, in assumed $d - 2k$ for rolled sections, and $d - 2t_f$ for welded sections
k	Distance from outer face of flange to web toe of fillet, in
k_c	Parameter used for section classification, $4 / \sqrt{h/t_w}$, $0.35 \le k_c \le 0.763$
l_{33}, l_{22}	Major and minor directions unbraced member lengths, in
r	Radius of gyration, in
r_{33}, r_{22}	Radii of gyration in the major and minor directions, in
t	Thickness, in

t_f	Flange thickness, in
t_w	Thickness of web, in
β_w	Special section property for angles, in
λ	Slenderness parameter
λ_c, λ_e	Column slenderness parameters
λ_p	Limiting slenderness parameter for compact element
λ_r	Limiting slenderness parameter for non-compact element
λ_s	Limiting slenderness parameter for seismic element
$\lambda_{slender}$	Limiting slenderness parameter for slender element
φ_b	Resistance factor for bending, 0.9
φ_c	Resistance factor for compression, 0.85
φ_t	Resistance factor for tension, 0.9
φ_v	Resistance factor for shear, 0.9

STEEL FRAME DESIGN AISC-LRFD93

Technical Note 44
Preferences

This Technical Note describes the items in the Preferences form.

General

The steel frame design preferences in this program are basic assignments that apply to all steel frame elements. Use the **Options menu > Preferences > Steel Frame Design** command to access the Preferences form where you can view and revise the steel frame design preferences.

Default values are provided for all steel frame design preference items. Thus, it is not required that you specify or change any of the preferences. You should, however, at least review the default values for the preference items to make sure they are acceptable to you.

Using the Preferences Form

To view preferences, select the **Options menu > Preferences > Steel Frame Design.** The Preferences form will display. The preference options are displayed in a two-column spreadsheet. The left column of the spreadsheet displays the preference item name. The right column of the spreadsheet displays the preference item value.

To change a preference item, left click the desired preference item in either the left or right column of the spreadsheet. This activates a drop-down box or highlights the current preference value. If the drop-down box appears, select a new value. If the cell is highlighted, type in the desired value. The preference value will update accordingly. You cannot overwrite values in the drop-down boxes.

When you have finished making changes to the composite beam preferences, click the **OK** button to close the form. You must click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the preferences are ignored and the form is closed.

Preferences

For purposes of explanation, the preference items are presented in Table 1. The column headings in the table are described as follows:

- **Item:** The name of the preference item as it appears in the cells at the left side of the Preferences form.
- **Possible Values:** The possible values that the associated preference item can have.
- **Default Value:** The built-in default value that the program assumes for the associated preference item.
- **Description:** A description of the associated preference item.

Table 1: Steel Frame Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	AISC-ASD89	Design code used for design of steel frame elements.
Time History Design	Envelopes, Step-by-Step	Envelopes	Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has *more than one* time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.
Frame Type	Moment Frame, Braced Frame	Moment Frame	
Stress Ratio Limit	>0	0.95	Program will select members from the auto select list with stress ratios less than or equal to this value.
Maximum Auto Iteration	≥1	1	Sets the number of iterations of the analysis-design cycle that the program will complete automatically assuming that the frame elements have been assigned as auto select sections.

Technical Note 45
Overwrites

General

The steel frame design overwrites are basic assignments that apply only to those elements to which they are assigned. This Technical Note describes steel frame design overwrites for AISC-LRFD93. To access the overwrites, select an element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

Default values are provided for all overwrite items. Thus, you do not need to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned; that is, to the elements that are selected when the overwrites are changed.

Overwrites

For explanation purposes in this Technical Note, the overwrites are presented in Table 1. The column headings in the table are described as follows.

- **Item:** The name of the overwrite item as it appears in the program. To save space in the forms, these names are generally short.

- **Possible Values:** The possible values that the associated overwrite item can have.

- **Default Value:** The default value that the program assumes for the associated overwrite item. If the default value is given in the table with an associated note "Program Calculated," the value is shown by the program before the design is performed. After design, the values are calculated by the program and the default is modified by the program-calculated value.

- **Description:** A description of the associated overwrite item.

An explanation of how to change an overwrite is provided at the end of this Technical Note.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Current Design Section			Indicates selected member size used in current design.
Element Type	Moment Frame, Braced Frame	From Preferences	
Live Load Reduction Factor	≥0	1	Live load is multiplied by this factor.
Horizontal Earthquake Factor	≥0	1	Earthquake loads are multiplied by this factor.
Unbraced Length Ratio (Major)	≥0	1	Ratio of unbraced length divided by total length.
Unbraced Length Ratio (Minor, LTB)	≥0	1	Ratio of unbraced length divided by total length.
Effective Length Factor (K Major)	≥0	1	As defined in AISC-LRFD Table C-C2.1, page 6-184.
Effective Length Factor (K Minor)	≥0	1	As defined in AISC-LRFD Table C-C2.1, page 6-184.
Moment Coefficient (Cm Major)	≥0	0.85	As defined in AISC-LRFD specification Chapter C.
Moment Coefficient (Cm Minor)	≥0	0.85	As defined in AISC-LRFD specification Chapter C.
Bending Coefficient (Cb)	≥0	1	As defined in AISC-LRFD specification Chapter F.

Table 1 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
NonSway Moment Factor (B1 Major)	≥0	1	As defined in AISC-LRFD specification Chapter C.
NonSway Moment Factor (B1 Minor)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Sway Moment Factor (B2 Major)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Sway Moment Factor (B2 Minor)	≥0	1	As defined in AISC-LRFD specification Chapter C.
Yield stress, Fy	≥0	0	If zero, yield stress defined for material property data used.
Compressive Capacity, phi*Pnc	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
Tensile Capacity, phi*Pnt	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
Major Bending Capacity, phi*Mn3	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
Minor Bending Capacity, phi*Mn2	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
Major Shear Capacity, phi*Vn2	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
Minor Shear Capacity, phi*Vn3	≥0	0	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.

Making Changes in the Overwrites Form

To access the steel frame overwrites, select a frame element and click the **Design menu > Steel Frame Design > View/Revise Overwrites** command.

The overwrites are displayed in the form with a column of check boxes and a two-column spreadsheet. The left column of the spreadsheet contains the name of the overwrite item. The right column of the spreadsheet contains the overwrites values.

Initially, the check boxes in the Steel Frame Design Overwrites form are all unchecked and all of the cells in the spreadsheet have a gray background to indicate that they are inactive and the items in the cells cannot be changed. The names of the overwrite items are displayed in the first column of the spreadsheet. The values of the overwrite items are visible in the second column of the spreadsheet if only one frame element was selected before the overwrites form was accessed. If multiple elements were selected, no values show for the overwrite items in the second column of the spreadsheet.

After selecting one or multiple elements, check the box to the left of an overwrite item to change it. Then left click in either column of the spreadsheet to activate a drop-down box or highlight the contents in the cell in the right column of the spreadsheet. If the drop-down box appears, select a value from the box. If the cell contents is highlighted, type in the desired value. The overwrite will reflect the change. You cannot change the values of the drop-down boxes.

When changes to the overwrites have been completed, click the **OK** button to close the form. The program then changes all of the overwrite items whose associated check boxes are checked for the selected members. You *must* click the **OK** button for the changes to be accepted by the program. If you click the **Cancel** button to exit the form, any changes made to the overwrites are ignored and the form is closed.

Resetting Steel Frame Overwrites to Default Values

Use the **Design menu > Steel Frame Design > Reset All Overwrites** command to reset all of the steel frame overwrites. All current design results will be deleted when this command is executed.

Important note about resetting overwrites: The program defaults for the overwrite items are built into the program. The steel frame overwrite values that were in a .edb file that you used to initialize your model may be different from the built-in program default values. When you reset overwrites, the program resets the overwrite values to its built-in values, not to the values that were in the .edb file used to initialize the model.

STEEL FRAME DESIGN AISC-LRFD93

Technical Note 46
Design Load Combinations

The design load combinations are the various combinations of the load cases for which the structure needs to be checked. For the AISC-LRFD93 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, the following load combinations may need to be defined (LRFDA4.1):

1.4DL	(LRFD A4-1)
1.2DL + 1.6LL	(LRFD A4-2)
0.9DL ± 1.3WL	(LRFD A4-6)
1.2DL ± 1.3WL	(LRFD A4-4)
1.2DL + 0.5LL ± 1.3WL	(LRFD A4-4)
0.9DL ± 1.0 EL	(LRFD A4-6)
1.2DL ± 1.0 EL	(LRFD A4-4)
1.2DL + 0.5LL ± EL	(LRFD A4-4)

These are also the default design load combinations in the program whenever the AISC-LRFD93 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading. See AISC-LRFD93 Steel Frame Design Technical Note 45 Overwrites for more information.

When using the AISC-LRFD93 code, the program design assumes that a P-delta analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-delta analysis be performed at the factored load level of 1.2DL plus 0.5LL (White and Hajjar 1991).

Reference

White, D.W. and J.F. Hajjar. 1991. Application of Second-Order Elastic Analysis in LRFD: Research to Practice. *Engineering Journal*. American Institute of Steel Construction, Inc. Vol. 28. No. 4.

STEEL FRAME DESIGN AISC-LRFD93

Technical Note 47
Classification of Sections

This Technical Note explains the classification of sections when the user selects the AISC-LRFD93 design code.

The nominal strengths for axial compression and flexure are dependent on the classification of the section as Compact, Noncompact, Slender, or Too Slender. The program classifies individual members according to the limiting width/thickness ratios given in Table 1 and Table 2 (LRFD B5.1, A-G1, Table A-F1.1). The definition of the section properties required in these tables is given in Figure 1 and AISC-LRFD93 Steel Frame Design Technical Note 43 General and Notation. Moreover, special considerations are required regarding the limits of width-thickness ratios for Compact sections in Seismic zones and Noncompact sections with compressive force as given in Table 2. If the limits for Slender sections are not met, the section is classified as Too Slender. **Stress check of Too Slender sections is beyond the scope of this program.**

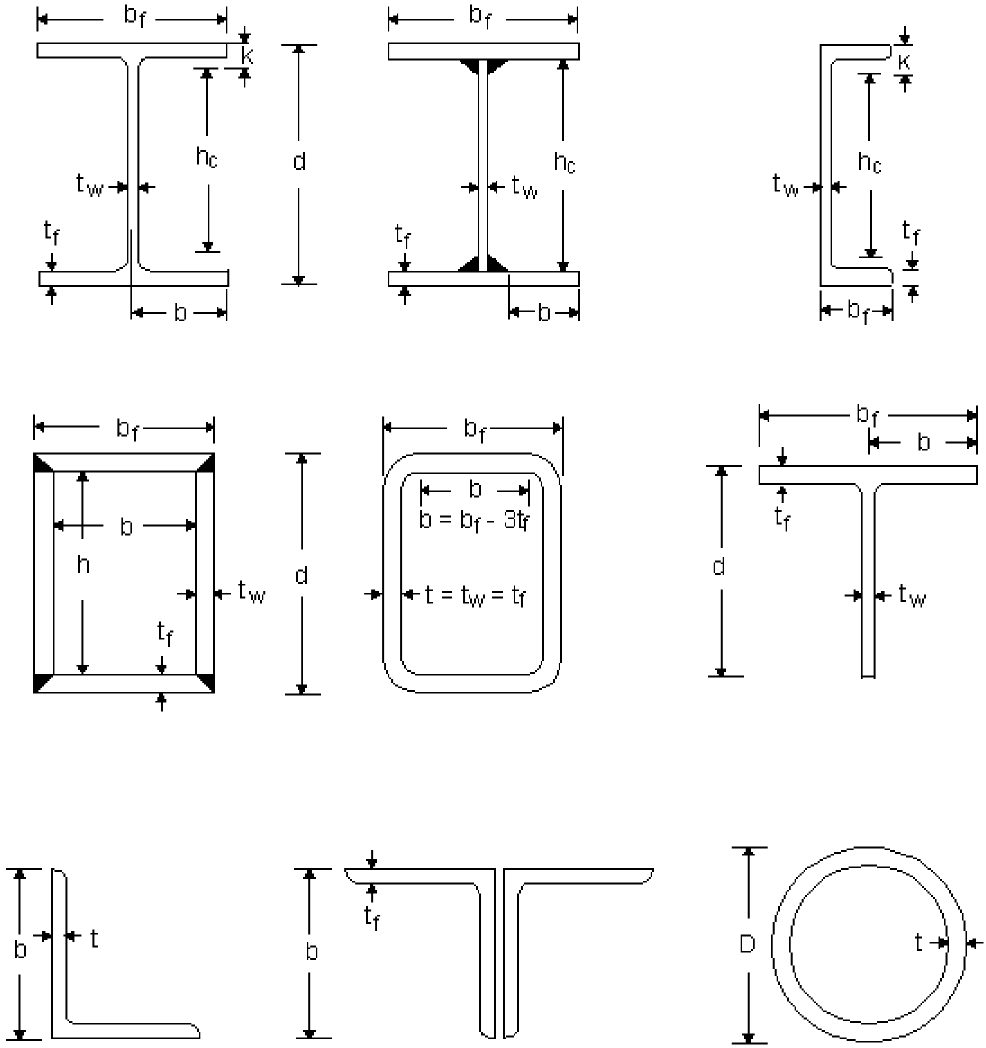
In classifying web slenderness of I-shapes, Box, and Channel sections, it is assumed that there are no intermediate stiffeners. Double angles are conservatively assumed to be separated.

Table 1 Limiting Width-Thickness Ratios for Classification of Sections in Flexure Based on AISC-LRFD

Description of Section	Check λ	COMPACT (λ_p)	NONCOMPACT λ_r	SLENDER $(\lambda_{slender})$
I-SHAPE	$b_f / 2t_f$ (rolled)	$\leq 65 / \sqrt{F_y}$	$\leq 141 / \sqrt{F_y - 10.0}$	No limit
	$b_f / 2t_f$ (welded)	$\leq 65 / \sqrt{F_y}$	$\leq 162 / \sqrt{\dfrac{F_y - 16.5}{k_c}}$	No limit
	h_c / t_w	For $P_u / \varphi_b P_y \leq 0.125$, $\leq \dfrac{640}{\sqrt{F_y}}\left(1 - \dfrac{2.75 P_u}{\varphi_b P_y}\right)$ For $P_u / \varphi_b P_y > 0.125$, $\leq \begin{cases} \dfrac{191}{\sqrt{F_y}}\left(2.33 - \dfrac{P_u}{\varphi_b P_y}\right) \\ \geq \dfrac{253}{\sqrt{F_y}} \end{cases}$	$\leq \dfrac{970}{\sqrt{F}}\left[1 - 0.74 \dfrac{P_u}{\varphi_b P_y}\right]$	$\leq \begin{cases} \dfrac{14{,}000}{\sqrt{F_y (F_y + 16.5)}} \\ \leq 260 \end{cases}$
BOX	b / t_f h_c / t_w	$\leq 190 / \sqrt{F_y}$ As for I-shapes	$\leq 238 / \sqrt{F_y}$ As for I-shapes	No limit $\leq 970 / \sqrt{F_y}$
CHANNEL	b_f / t_f h_c / t_w	As for I-shapes As for I-shapes	As for I-shapes As for I-shapes	No limit As for I-shapes
T-SHAPE	$b_f / 2t_f$ d / t_w	As for I-shapes Not applicable	As for I-shapes $\leq 127 / \sqrt{F_y}$	No limit No limit
ANGLE	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
DOUBLE-ANGLE (Separated)	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
PIPE	D / t	$\leq 2{,}070 / F_y$	$\leq 8{,}970 / \sqrt{F_y}$	$\leq 13{,}000 / F_y$ (Compression only) No limit for flexure
ROUND BAR	—	Assumed Compact		
RECTAN-GULAR	—	Assumed Compact		
GENERAL	—	Assumed Compact		

Table 2 Limiting Width-Thickness Ratios for Classification of Sections (Special Cases) Based on AISC-LRFD

Description of Section	Width-Thickness Ratio λ	NONCOMPACT (Uniform Compression) ($M_{22} \approx M_{33} \approx 0$) ($\lambda_r$)
I-SHAPE	$b_f / 2t_f$ (rolled)	$\leq 95 / \sqrt{F_y}$
	$b_f / 2t_f$ (welded)	$\leq 95 / \sqrt{F_y}$
	h_c / t_w	$\leq 253 / \sqrt{F_y}$
BOX	b / t_f h_c / t_w	$\leq 238 / \sqrt{F_y}$ $\leq 253 / \sqrt{F_y}$
CHANNEL	b_f / t_f h_c / t_w	As for I-shapes As for I-shapes
T-SHAPE	$b_f / 2t_f$ d / t_w	As for I-shapes $\leq 127 / \sqrt{F_y}$
ANGLE	b / t	$\leq 76 / \sqrt{F_y}$
DOUBLE-ANGLE (Separated)	b / t	$\leq 76 / \sqrt{F_y}$
PIPE	D / t	$\leq 3{,}300 / \sqrt{F_y}$
ROUND BAR	—	Assumed Compact
RECTANGULAR	—	Assumed Noncompact
GENERAL	—	Assumed Noncompact



AISC-LRFD93: Axes Conventions

2-2 is the cross section axis parallel to the webs, the longer dimension of tubes, the longer leg of single angles, or the side by side legs of double angles. This is the same as the y-y axis.

3-3 is orthogonal to 2-2. This is the same as the x-x axis.

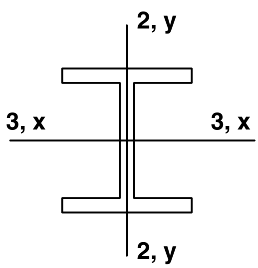


Figure 1 AISC-LRFD Definition of Geometric Properties

STEEL FRAME DESIGN AISC-LRFD93

Technical Note 48
Calculation of Factored Forces and Moments

This Technical Note describes how the program calculates factored forces and moments.

The factored member loads that are calculated for each load combination are P_u, M_{u33}, M_{u22}, V_{u2}, and V_{u3}, corresponding to factored values of the axial load, the major moment, the minor moment, the major direction shear force and the minor direction force, respectively. These factored loads are calculated at each of the previously defined stations.

For loading combinations that cause compression in the member, the factored moment M_u (M_{u33} and M_{u22} in corresponding directions) is magnified to consider second order effects. The magnified moment in a particular direction is given by:

$$M_u = B_1 M_{nt} + B_2 M_{lt}, \text{ where} \qquad \text{(LRFD C1-1, SAM 6)}$$

B_1 = Moment magnification factor for non-sidesway moments,
B_2 = Moment magnification factor for sidesway moments,
M_{nt} = Factored moments not causing sidesway, and
M_{lt} = Factored moments causing sidesway.

The moment magnification factors are associated with corresponding directions. The moment magnification factor B_1 for moments not causing sidesway is given by

$$B_1 = \frac{C_m}{(1 - P_u / P_e)} \geq 1.0, \text{ where} \qquad \text{(LRFD C1-2, SAM 6-2)}$$

P_e is the Euler buckling load ($P_e = \dfrac{A_g F_y}{\lambda^2}$, with $\lambda = \dfrac{Kl}{r\pi}\sqrt{\dfrac{F_y}{E}}$), and

C_{m33} and C_{m22} are coefficients representing distribution of moment along the member length.

$$C_m = \begin{cases} 1.00 & \text{if length is overwritten,} \\ 1.00 & \text{if tension member,} \\ 1.00 & \text{if end unrestrained,} \\ 0.6\text{-}0.4\,\dfrac{M_a}{M_b} & \text{if no transverse loading} \\ 0.85 & \text{if trans. load, end restrained} \\ 1.00 & \text{if trans. load, end unrestrained} \end{cases} \qquad \text{(LRFD C1-3)}$$

M_a / M_b is the ratio of the smaller to the larger moment at the ends of the member; M_a / M_b being positive for double curvature bending and negative for single curvature bending. For tension members, C_m is assumed as 1.0. For compression members with transverse load on the member, C_m is assumed as 1.0 for members with any unrestrained end and as 0.85 for members with two unrestrained ends. When M_b is zero, C_m is taken as 1.0. The program defaults C_m to 1.0 if the unbraced length factor, l, of the member is redefined by either the user or the program, i.e., if the unbraced length is not equal to the length of the member. The user can overwrite the value of C_m for any member. C_m assumes two values, C_{m22} and C_{m33}, associated with the major and minor directions.

The magnification factor B_1 must be a positive number. Therefore P_u must be less than P_e. If P_u is found to be greater than or equal to P_e, a failure condition is declared.

The program design assumes the analysis includes P-delta effects; therefore, B_2 is taken as unity for bending in both directions. It is suggested that the P-delta analysis be performed at the factored load level of 1.2 DL plus 0.5 LL (LRFD C2.2). See also White an Hajjar (1991).

For single angles, where the principal axes of bending are not coincident with the geometric axes (2-2 and 3-3), the program conservatively uses the maximum of $K_{22}l_{22}$ and $K_{33}l_{33}$ for determining the major and minor direction Euler buckling capacity.

If the program assumptions are not satisfactory for a particular structural model or member, the user has a choice of explicitly specifying the values of B_1 and B_2 for any member.

Reference

White, D.W. and J. F. Hajjar. 1991. Application of Second-Order Elastic Analysis in LRFD: Research to Practice. *Engineering Journal.* American Institute of Steel Construction, Inc. Vol. 28, No. 4.

Technical Note 49
Calculation of Nominal Strengths

This Technical Note describes how the program calculates nominal strengths in compression, tension, bending, and shear for Compact, Noncompact, and Slender sections.

Overview

The nominal strengths in compression, tension, bending, and shear are computed for Compact, Noncompact, and Slender sections according to the following subsections. The nominal flexural strengths for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I, Box, Channel, Circular, Pipe, T, and Double-angle sections, the principal axes coincide with their geometric axes. For the Angle Sections, the principal axes are determined and all computations except shear are based on that.

For Single-angle sections, the nominal shear strengths are calculated for directions along the geometric axes. For all other sections, the shear stresses are calculated along their geometric and principal axes.

The strength reduction factor, φ, is taken as follows (LRFD A5.3):

φ_t = Resistance factor for tension, 0.9 (LRFD D1, H1, SAM 2, 6)
φ_c = Resistance factor for compression, 0.85 (LRFD E2, E3, H1)
φ_c = Resistance factor for compression in angles, 0.90 (LRFD SAM 4,6)
φ_b = Resistance factor for bending, 0.9 (LRFD F1, H1, A-F1, A-G2, SAM 5)
φ_v = Resistance factor for shear, 09 (LRFD F2, A-F2, A-G3, SAM 3)

If the user specifies nonzero factored strengths for one or more of the elements on the Steel Frame Overwrites form, these values will override the calculated values for those elements. The specified factored strengths should be based on the principal axes of bending.

Compression Capacity

The nominal compression strength is the minimum value obtained from flexural buckling, torsional buckling and flexural-torsional buckling. The strengths are determined according to the following subsections.

For members in compression, if Kl/r is greater than 200, a message to that effect is printed (LRFD B7, SAM 4). For single angles, the minimum radius of gyration, r_z, is used instead of r_{22} and r_{33} in computing Kl/r.

Flexural Buckling

The nominal axial compression strength, P_n, depends on the slenderness ratio, Kl/r, and its critical value, λ_c, where

$$\frac{Kl}{r} = \max\left\{\frac{K_{33}l_{33}}{r_{33}}, \frac{K_{22}l_{22}}{r_{22}}\right\}, \text{ and}$$

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}. \qquad \text{(LRFD E2-4, SAM 4)}$$

For single angles, the minimum radius of gyration, r_z, is used instead of r_{22} and r_{33} in computing Kl/r.

P_n for Compact or Noncompact sections is evaluated for flexural buckling as follows:

$$P_n = A_g F_{cr}, \text{ where} \qquad \text{(LRFD E2-1)}$$

$$F_{cr} = (0.658^{\lambda_c^2})F_y, \quad \text{for } \lambda_c \leq 1.5, \text{ and} \qquad \text{(LRFD E2-2)}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y, \quad \text{for } \lambda_c > 1.5 \qquad \text{(LRFD E2-3)}$$

P_n for Slender sections is evaluated for flexural buckling as follows:

$$P_n = A_g F_{cr}, \text{ where} \qquad \text{(LRFD A-B3d, SAM 4)}$$

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_y, \text{ for } \lambda_c\sqrt{Q} \leq 1.5, \text{ and} \qquad \text{(LRFD A-B5-15, SAM 4-1)}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y, \qquad \text{for } \lambda_c \sqrt{Q} > 1.5 \qquad \text{(LRFD E2-3)}$$

The reduction factor, Q, for all compact and noncompact sections is taken as 1. For slender sections, Q is computed as follows:

$Q = Q_s\, Q_a$, where (LRFD A-B5-17, SAM 4)

Q_s = reduction factor for unstiffened slender elements, and(LRFD A-B5.3a)

Q_a = reduction factor for stiffened slender elements. (LRFD A-B5.3c)

The Q_s factors for slender sections are calculated as described in Table 1 (LRFD A-B5.3a). The Q_a factors for slender sections are calculated as the ratio of effective cross-sectional area and the gross cross-sectional area (LRFD A-B5.3c).

$$Q_a = \frac{A_e}{A_s} \qquad \text{(LRFD A-B5-14)}$$

The effective cross-sectional area is computed based on effective width as follows:

$$A_e = A_g - \Sigma(b - b_e)t$$

b_e for unstiffened elements is taken equal to b, and b_e for stiffened elements is taken equal to or less than b as given in Table 2 (LRFD A-B5.3b). For webs in I, Box, and Channel sections, h_e is used as b_e and h is used as b in the above equation.

Flexural-Torsonal Buckling

P_n for flexural-torsional buckling of Double-angle and T-shaped compression members whose elements have width-thickness ratios less than λ_r is given by

$P_n = A_g F_{crft}$, where (LRFD E3-1)

$$F_{crft} = \left(\frac{F_{cr2} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cr2}F_{crz}H}{(F_{cr2} + F_{crz})^2}}\right], \text{ where} \qquad \text{(LRFD E3-1)}$$

Table 1 Reduction Factor for Unstiffened Slender Elements, Q_s

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0 & \text{if} \quad b_f/2t_f \le 95/\sqrt{F_y}, \\ 1.415 - 0.00437[b_f/2t_f]\sqrt{F_y} & \text{if} \quad 95/\sqrt{F_y} < b_f/2t_f < 176/\sqrt{F_y}, \\ 20{,}000 / \{[b_f/2t_f]^2 F_y\} & \text{if} \quad b_f/2t_f \ge 176/\sqrt{F_y}. \end{cases}$	LRFD A-B5-5, LRFD A-B5-6
	$Q_s = \begin{cases} 1.0 & \text{if} \quad b_f/2t_f \le 109/\sqrt{F_y/k_c} \\ 1.415 - 0.00381[b_f/2t_f]\sqrt{F_y/k_c} & \text{if} \quad 109/\sqrt{F_y/k_c} < b_f/2t_f < 200/\sqrt{F_y/k_c} \\ 26{,}200 k_c / \{[b_f/2t_f]^2 F_y\} & \text{if} \quad b_f/2t_f \ge 200/\sqrt{F_y/k_c}. \end{cases}$	LRFD A-B5-7, LRFD A-B5-8
BOX	$Q_s = 1$	LRFD A-B5.3d
CHANNEL	As for I-shapes with $b_f / 2t_f$ replaced by b_f / t_f	LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8
T-SHAPE	*For flanges, as for flanges in I-shapes. For web, see below.* $Q_s = \begin{cases} 1.0 & \text{if} \quad d/t_w \le 127/\sqrt{F_y}, \\ 1.908 - 0.00715[d/t_w]\sqrt{F_y} & \text{if} \quad 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}, \\ 20{,}000 / \{[d/t_w]^2 F_y\} & \text{if} \quad d/t_w \ge 176/\sqrt{F_y}. \end{cases}$	LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8, LRFD A-B5-9, LRFD A-B5-10
DOUBLE-ANGLE (Separated)	$Q_s = \begin{cases} 1.0 & \text{if} \quad b/t \le 76/\sqrt{F_y}, \\ 1.340 - 0.00447[b/t]\sqrt{F_y} & \text{if} \quad 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500 / \{[b/t]^2 F_y\} & \text{if} \quad b/t \ge 155/\sqrt{F_y}. \end{cases}$	LRFD A-B5-3 LRFD A-B5-4
ANGLE	$Q_s = \begin{cases} 1.0 & \text{if} \quad b/t \le 0.446/\sqrt{F_y/E}, \\ 1.34 - 0.761[b/t]\sqrt{F_y/E} & \text{if} \quad 0.446\sqrt{F_y/E} < b/t < 0.910/\sqrt{F_y/E}, \\ 0.534 / \{[b/t]^2 [F_y/E]\} & \text{if} \quad b/t \ge 0.910/\sqrt{F_y/E}. \end{cases}$	LRFD SAM4-3
PIPE	$Q_s = 1$	LRFD A-B5.3d
ROUND BAR	$Q_s = 1$	LRFD A-B5.3d
RECTANGULAR	$Q_s = 1$	LRFD A-B5.3d
GENERAL	$Q_s = 1$	LRFD A-B5.3d

Table 2 Effective Width for Stiffened Sections

Section Type	Effective Width for Stiffened Sections	Equation Reference
I-SHAPE	$h_e = \begin{cases} h & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right] & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}} \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
BOX	$h_e = \begin{cases} h & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right] & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}} \end{cases}$ (compression only, $f = \frac{P}{A_g}$) $b_e = \begin{cases} b, & \text{if } \frac{h}{t_w} \le \frac{238}{\sqrt{f}}, \\ \frac{326t_f}{\sqrt{f}}\left[1 - \frac{64.9}{(b/t_f)\sqrt{f}}\right] & \text{if } \frac{b}{t_f} > \frac{238}{\sqrt{f}} \end{cases}$ (compr. or flexure, $f = F_y$)	LRFD A-B5-12 LRFD A-B5-11
CHANNEL	$h_e = \begin{cases} h & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right] & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}} \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
T-SHAPE	b_e - b	LRFD A-B5.3B
DOUBLE-ANGLE (Separated)	b_e - b	LRFD A-B5.3B
ANGLE	b_e - b	LRFD A-B5.3B
PIPE	$Q_a = \begin{cases} 1, & \text{if } \frac{D}{t} \le \frac{3{,}300}{F_y} \\ \frac{1{,}100}{(D/t)F_y} + \frac{2}{3} & \text{if } \frac{D}{t} > \frac{3{,}300}{F_y} \end{cases}$ (compression only)	LRFD A-B5-13
ROUND BAR	Not applicable	—
RECTANGULAR	b_e - b	LRFD A-B5.3b
GENERAL	Not applicable	—

$$F_{crz} = \frac{GJ}{Ar_o^2}$$

$$H = 1-\left(\frac{x_o^2 + y_o^2}{r_o^2}\right),$$

r_o = Polar radius of gyration about the shear center,

x_o, y_o are the coordinates of the shear center with respect to the centroid, x_o = 0 for double angle and T-shaped members (y-axis of symmetry),

F_{cr2} is determined according to the equation LRFD E2-1 for flexural buckling about the minor axis of symmetry for $\lambda_c = \frac{Kl}{\pi r_{22}}\sqrt{\frac{F_y}{E}}$.

Torsional and Flexural-Torsional Buckling

The strength of a compression member, P_n, determined by the limit states of torsional and flexural-torsional buckling, is determined as follows:

$$P_n = A_g F_{cr}, \text{ where} \qquad \text{(LRFD A-E3-1)}$$

$$F_{cr} = Q(0.658^{Q\lambda_e^2})F_y, \quad \text{for } \lambda_e\sqrt{Q} \leq 1.5, \text{ and} \qquad \text{(LRFD A-E3-2)}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_e^2}\right]F_y, \quad \text{for } \lambda_e\sqrt{Q} > 1.5. \qquad \text{(LRFD A-E3-3)}$$

In the above equations, the slenderness parameter λ_e is calculated as

$$\lambda_e = \sqrt{\frac{F_y}{F_e}}, \qquad \text{(LRFD A-E3-4)}$$

where F_e is calculated as follows:

- For Rectangular, I, Box and Pipe sections:

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z l_z)^2} + GJ\right]\frac{1}{I_{22} + I_{33}} \qquad \text{(LRFD A-E3-5)}$$

- For T-sections and Double-angles:

$$F_e = \left(\frac{F_{e22} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e22}F_{ez}H}{(F_{e22} + F_{ez})^2}}\right] \quad \text{(LRFD A-E3-6)}$$

- For Channels:

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{(F_{e33} + F_{ez})^2}}\right] \quad \text{(LRFD A-E3-6)}$$

- For Single-angle sections with equal legs:

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{(F_{e33} + F_{ez})^2}}\right] \quad \text{(LRFD A-E3-6)}$$

- For Single-angle sections with unequal legs, F_e is calculated as the minimum real root of the following cubic equation (LRFD A-E3-7):

$$(F_e - F_{e33})(F_e - F_{e22})(F_e - F_{ez}) - F_e^2(F_e - F_{e22})\frac{x_o^2}{r_o^2} - F_e^2(F_e - F_{e33})\frac{y_o^2}{r_o^2} = 0$$

where

x_o, y_o are the coordinates of the shear center with respect to the centeroid, $x_o = 0$ for double-angle and T-shaped members (y-axis symmetry),

$r_o = \sqrt{x_o^2 + y_o^2 + \frac{I_{22} + I_{33}}{A_g}}$ = polar radius of gyration about the shear center,

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{r_o^2}\right), \quad \text{(LRFD A-E3-9)}$$

$$F_{e33} = \frac{\pi^2 E}{(K_{33}l_{33} / r_{33})^2} \quad \text{(LRFD A-E3-10)}$$

$$F_{e22} = \frac{\pi^2 E}{(K_{22} l_{22} / r_{22})^2} \qquad \text{(LRFD A-E3-11)}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(K_z l_z)^2} + GJ\right]\frac{1}{A r_0^2} \qquad \text{(LRFD A-E3-12)}$$

K_{22}, K_{33} are effective length factors in minor and major directions,

K_z is the effective length factor for torsional buckling, and it is taken equal to K_{22} in this program,

l_{22}, l_{33} are effective lengths in the minor and major directions,

l_z is the effective length for torsional buckling and it is taken equal to l_{22}.

For angle sections, the principal moment of inertia and radii of gyration are used for computing F_e. Also, the maximum value of Kl, i.e., max (K_{22},l_{22}, K_{33},l_{33}), is used in place of $K_{22}l_{22}$ or $K_{33}l_{33}$ in calculating F_{e22} and F_{e33} in this case.

Tension Capacity

The nominal axial tensile strength value P_n is based on the gross cross-sectional area and the yield stress.

$$P_n = A_g F_y \qquad \text{(LRFD D1-1)}$$

It should be noted that no net section checks are made. For members in tension, if l/r is greater than 300, a message to that effect is printed (LRFD B7, SAM 2). For single angles, the minimum radius gyration, r_z, is used instead of r_{22} and r_{33} in computing Kl/r.

Nominal Strength in Bending

The nominal bending strength depends on the following criteria: the geometric shape of the cross-section; the axis of bending; the compactness of the section; and a slenderness parameter for lateral-torsional buckling. The nominal strengths for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I, Box, Channel, Circular, Pipe, T, and Double-angle sections, the principal axes coincide with their geometric axes. For the Single-angle sections, the principal axes are determined, and all computations related to flexural strengths are based on that. The nominal bending strength is the minimum value obtained according to the limit states of yielding, lateral-torsional buckling, flange local buckling, and web local buckling, as follows:

Yielding

The flexural design strength of beams, determined by the limit state of yielding, is:

$$M_p = ZF_y \le 1.5\ S\ F_y \qquad \text{(LRFD F1-1)}$$

Lateral-Torsional Buckling

Doubly Symmetric Shapes and Channels

For I, Channel, Box, and Rectangular shaped members bent around the major axis, the moment capacity is given by the following equation (LRFD F1):

$$M_{n33} = \begin{cases} M_{p33} & \text{if} \quad L_b \le L_p \\ C_b \left[M_{p33} - (M_{p33} - M_{r33}) \left(\dfrac{L_b - L_p}{L_r - L_p} \right) \right] \le M_{p33} & \text{if} \quad L_p < L_b \le L_r \\ M_{cr33} \le \mathrm{M}_{p33} & \text{if} \quad L_b > L_r. \end{cases}$$

(LRFD F1-1, F1-2, F1-12)

where,

M_{n33} = Nominal major bending strength

M_{p33} = Major plastic moment, $Z_{33}F_y \le 1.5\ S_{33}F_y$, (LRFD F1.1)

M_{r33} = Major limiting buckling moment
$(F_y$ - $F_r)S_{33}$ for I-shapes and channels, (LRFD F1-7)
and $F_yS_{eff,33}$ for rectangular bars and boxes (LRFD F1-11)

M_{cr33} = Critical elastic moment,

$$\frac{C_b\pi}{L_b}\sqrt{EI_{22}GJ + \left(\frac{\pi E}{L_b}\right)^2 I_{22}C_w}$$ for I-shapes and

channels and (LRFD F1-13)

$$\frac{57{,}000 C_b \sqrt{JA}}{L_b / r_{22}}$$ for boxes and rectangular bars (LRFD F1-14)

L_b = Laterally unbraced length, l_{22}

L_p = Limiting laterally unbraced length for full plastic capacity,

$$\frac{300 r_{22}}{\sqrt{F_y}}$$ for I-shapes and channels, and (LRFD F1-4)

$$\frac{3{,}750 r_{22}}{M_{p33}} \sqrt{JA}$$ for boxes and rectangular bars, (LRFD F1-5)

L_r = Limiting laterally unbraced length for inelastic lateral-torsional buckling,

$$\frac{r_{22} X_1}{F_y - F_r} \left\{ 1 + \left[1 + X_2 \left(F_y - F_r \right)^2 \right]^{1/2} \right\}^{1/2}$$ for I-shapes and channels, and (LRFD F1-6)

$$\frac{57{,}000 r_{22} \sqrt{JA}}{M_{r33}}$$ for boxes and rectangular bars, (LRFD F1-10)

$$X_1 = \frac{\pi}{S_{33}} \sqrt{\frac{EGJA}{2}} \qquad \text{(LRFD F1-8)}$$

$$X_2 = 4 \frac{C_w}{I_{22}} \left(\frac{S_{33}}{GJ} \right)^2 \qquad \text{(LRFD F1-9)}$$

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3 M_A + 4 M_B + 3 M_c} \text{ and} \qquad \text{(LRFD F1-3)}$$

M_{max}, M_A, M_B, and M_c are absolute values of maximum moment, 1/4 point, center of span and 3/4 point major moments respectively, in the member. C_b should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. **The user should overwrite C_b for cantilevers.** The program also defaults C_b to 1.0 if the minor unbraced length, l_{22}, of the member is redefined by the user (i.e., it is not equal to the length of the member). The user can overwrite the value of C_b for any member.

For I, Channel, Box, and Rectangular shaped members bent about the minor axis, the moment capacity is given by the following equation:

$$M_{n22} = M_{p22} = Z_{22}F_y \leq 1.5S_{22}F_y \quad \text{(LRFD F1)}$$

For pipes and circular bars bent about any axis,

$$M_n = M_p = ZF_y \leq 1.5SF_y. \quad \text{(LRFD F1)}$$

T-Sections and Double-Angles

For T-shapes and Double-angles, the nominal major bending strength is given as,

$$M_{n33} = \frac{\pi\sqrt{EI_{22}GJ}}{L_b}\left[B + \sqrt{1 + B^2}\right], \text{ where} \quad \text{(LRFD F1-15)}$$

$$M_{n33} \leq 1.5F_yS_{33}, \text{ for positive moment, stem in tension} \quad \text{(LRFD F1.2c)}$$

$$M_{n33} \leq F_yS_{33}, \text{ for negative positive moment, stem in tension} \quad \text{(LRFD F1.2c)}$$

$$B = \pm\, 2.3\, \frac{d}{L_b}\sqrt{\frac{I_{22}}{J}} \quad \text{(LRFD F1-16)}$$

The positive sign for B applies for tension in the stem of T-sections or the outstanding legs of double angles (positive moments) and the negative sign applies for compression in stem or legs (negative moments).

For T-shapes and double-angles the nominal minor bending strength is assumed as:

$$M_{n22} = S_{22}F_y.$$

Single Angles

The nominal strengths for Single-angles are calculated based on their principal axes of bending. The nominal major bending strength for Single-angles for the limit state of lateral-torsional buckling is given as follows (LRFD SAM 5.1.3):

$$M_{n,major} = \left[0.92 - 0.17\frac{M_{ob}}{M_{y,major}}\right]M_{ob} \leq 1.25\ M_{y,major},\ \text{if } M_{ob} \leq M_{y,major}$$

$$M_{n,major} = \left[1.58 - 0.83\sqrt{\frac{M_{y,major}}{M_{ob}}}\right]M_{y,major} \leq 1.25\ M_{y,major},\ \text{if } M_{ob} \leq M_{y,major}$$

where,

$M_{y,major}$ = yield moment about the major principal axis of bending, considering the possibility of yielding at the heel and both of the leg tips,

M_{ob} = elastic lateral-torsional buckling moment as calculated below.

The elastic lateral-torsional buckling moment, M_{ob}, for equal-leg angles is taken as

$$M_{ob} = C_b\frac{0.46Eb^2t^2}{l} \qquad \text{(LRFD SAM 5-5)}$$

and for unequal-leg angles, the M_{ob} is calculated as

$$M_{ob} = 4.9EC_b\frac{I_{\min}}{l^2}\left[\sqrt{\beta_w^2 + 0.052(lt / r_{\min})^2} + \beta_w\right] \qquad \text{(LRFD SAM 5-6)}$$

where,

t = min (t_w, t_f)

l = max (l_{22}, l_{33})

I_{min} = minor principal axis moment of inertia

I_{max} = major principal axis moment of inertia,

r_{min} = radius of gyration for minor principal axis,

$$\beta_w = \left[\frac{1}{I_{\max}}\int_A z(w^2 + z^2)dA\right] -2z_0, \qquad \text{(LRFD SAM 5.3.2)}$$

z = coordinate along the major principal axis

w = coordinate along the minor principal axis, and

z_0 = coordinate of the shear center along the major principal axis with respect to the centroid.

β_w is a special section property for angles. It is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles (LRFD SAM 5.3.2). However, for conservative design in this program, it is always taken as negative for unequal-leg angles.

General Sections

For General Sections the nominal major and minor direction bending strengths are assumed as

$$M_n = S\,F_y.$$

Flange Local Buckling

The flexual design strength, M_n, of Noncompact and Slender beams for the limit state of Flange Local Buckling is calculated as follows (LRFD A-F1):

For major direction bending,

$$M_{n33} = \begin{cases} M_{p33} & \text{if} \quad \lambda \le \lambda_p, \\ M_{p33} - (M_{p33} - M_{r33}) \left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) & \text{if} \quad \lambda_p < \lambda \le \lambda_r, \\ M_{cr33} \le M_{p33} & \text{if} \quad \lambda > \lambda_r \end{cases} \quad \text{(A-F1-3)}$$

and for minor direction bending,

$$M_{n22} = \begin{cases} M_{p22} & \text{if} \quad \lambda \le \lambda_p, \\ M_{p22} - (M_{p22} - M_{r22}) \left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) & \text{if} \quad \lambda_p < \lambda \le \lambda_r, \\ M_{cr22} \le M_{p22} & \text{if} \quad \lambda > \lambda_r. \end{cases} \quad \text{(A-F1-3)}$$

where,

M_{n33} = Nominal major bending strength,

M_{n22} = Nominal minor bending strength,

M_{p33} = Major plastic moment, $Z_{33},F_y \leq 1.5S_{33}F_y$,

M_{p22} = Major plastic moment, $Z_{22},F_y \leq 1.5S_{22}F_y$,

M_{r33} = Major limiting buckling moment,

M_{r22} = Minor limiting buckling moment,

M_{cr33} = Major buckling moment,

M_{cr22} = Minor buckling moment,

λ = Controlling slenderness parameter,

λ_p = Largest value of λ for which $M_n = M_p$ and

λ_r = Largest value of λ for which buckling is inelastic.

The parameters λ, λ_p, λ_r, M_{r33},M_{r22}, M_{cr33}, and M_{cr22} for flange local buckling for different types of shapes are given below:

I Shapes, Channels

$$\lambda = \frac{b_f}{2t_f}, \text{ (for I sections)} \qquad \text{(LRFD B5.1, Table A-F1.1)}$$

$$\lambda = \frac{b_f}{t_f}, \text{ (for Channel sections)} \qquad \text{(LRFD B5.1, Table A-F1.1)}$$

$$\lambda_p = \frac{65}{\sqrt{F_y}}, \qquad \text{(LRFD B5.1, Table A-F1.1)}$$

$$\lambda_r = \begin{cases} \dfrac{141}{\sqrt{F_y - F_r}}, & \text{For rolled shape,} \\[2ex] \dfrac{162}{\sqrt{(F_y - F_r)/k_c}}, & \text{For welded shape,} \end{cases} \qquad \text{(LRFD Table A-F1.1)}$$

M_{r33} = $(F_y - F_r)\ S_{33}$ (LRFD Table A-F1.1)

M_{r22} = $F_y\ S_{22}$ (LRFD Table A-F1.1)

$$M_{cr33} = \begin{cases} \dfrac{20{,}000}{\lambda^2} S_{33} & \text{For rolled shape} \\ \dfrac{26{,}200 k_c}{\lambda^2} S_{33} & \text{For welded shape} \end{cases}$$ (LRFD Table A-F1.1)

$$M_{cr22} = \begin{cases} \dfrac{20{,}000}{\lambda^2} S_{22} & \text{For rolled shape} \\ \dfrac{26{,}200 k_c}{\lambda^2} S_{22} & \text{For welded shape} \end{cases}$$ (LRFD Table A-F1.1)

$$F_r = \begin{cases} 10 \quad \text{ksi} & \text{For rolled shape} \\ 16.5 \text{ ksi} & \text{For welded shape} \end{cases}$$ (LRFD Table A-F1)

Boxes

$$\lambda = \begin{cases} \dfrac{b_f - 3t_w}{t_f}, & \text{For rolled shape,} \\ \dfrac{b_f - 2t_w}{t_f}, & \text{For welded shape,} \end{cases}$$ (LRFD B5.1, Table A-F1.1)

$$\lambda_p = \frac{190}{\sqrt{F_y}},$$ (LRFD B5.1, Table A-F1.1)

$$\lambda_r = \frac{238}{\sqrt{F_y}},$$ (LRFD B5.1, Table A-F1.1)

M_{r33} = $(F_y - F_r)\ S_{eff,33}$ (LRFD Table A-F1.1)

M_{r22} = $(F_y - F_r)\ S_{eff,22}$ (LRFD Table A-F1.1)

M_{cr33} = $F_y\ S_{eff,33}\ (S_{eff,33}/S_{33})$ (LRFD Table A-F1.1)

M_{cr22} = $F_y\ S_{eff,22}$ (LRFD Table A-F1.1)

$$F_r = \begin{cases} 10 \text{ ksi} & \text{For rolled shape} \\ 16.5 \text{ ksi} & \text{For welded shape} \end{cases} \qquad \text{(LRFD Table A-F1)}$$

$S_{eff,33}$ = effective major section modulus considering slenderness and

$S_{eff,22}$ = effective minor section modulus considering slenderness.

T-Sections and Double Angles

No local buckling is considered for T-sections and Double-angles in this program. If special consideration is required, the user is expected to analyze this separately.

Singles Angles

The nominal strengths for Single-angles are calculated based on their principal axes of bending. The nominal major and minor bending strengths for Single-angles for the limit state of flange local buckling are given as follows (LRFD SAM 5.1.1):

$$M_n = \begin{cases} 1.25 F_y S_c & \text{if} \quad \dfrac{b}{t} \le 0.382\sqrt{\dfrac{E}{F_y}}, \\ F_y S_c \left[1.25 - 1.49 \left(\dfrac{b/t}{0.382\sqrt{\dfrac{E}{F_y}}} - 1 \right) \right] & \text{if } 0.382\sqrt{\dfrac{E}{F_y}} < \dfrac{b}{t} \le 0.446\sqrt{\dfrac{E}{F_y}} \\ Q F_y S_c & \text{if} \quad \dfrac{b}{t} > 0.446\sqrt{\dfrac{E}{F_y}} \end{cases}$$

where,

S_c = section modulus for compression at the tip of one leg,

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = strength reduction factor due to local buckling.

In calculating the bending strengths for single-angles for the limit state of flange local buckling, the capacities are calculated for both the principal axes considering the fact that either of the two tips can be under compression. The minimum capacities are considered.

Pipe Sections

$$\lambda = \frac{D}{t} \qquad \text{(LRFD B Table A-F1.1)}$$

$$\lambda_p = \frac{2{,}070}{F_y}, \qquad \text{(LRFD Table A-F1.1)}$$

$$\lambda_r = \frac{8{,}970}{F_y}, \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{r33} = M_{r22} = \left(\frac{600}{D/t} + F_y\right)S \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{cr33} = M_{cr22} = \left(\frac{9{,}570}{D/t}\right)S \qquad \text{(LRFD Table A-F1.1)}$$

Circular, Rectangular, and General Sections

No consideration of local buckling is required for solid circular shapes or rectangular plates (LRFD Table A-F1.1). No local buckling is considered in the program for circular, rectangular, and general shapes. If special consideration is required, the user is expected to analyze this separately.

Web Local Buckling

The flexural design strengths are considered in the program for only the major axis bending (LRFD Table A-F1.1).

I Shapes, Channels, and Boxes

The flexural design strength for the major axis bending, M_n, of Noncompact and Slender beams for the limit state of Web Local Buckling is calculated as follows (LRFD A-F1-1, A-F1-3, A-G2-2):

$$M_{n33} = \begin{cases} M_{p33} & \text{if} \quad \lambda \le \lambda_p \\ M_{p33} - (M_{p33} - M_{r33}) \left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) & \text{if} \quad \lambda_p < \lambda \le \lambda_r, \text{ (A-F1, A-G1)} \\ S_{33} R_{PG} R_e R_{cr} & \text{if} \quad \lambda > \lambda_r \end{cases}$$

where,

M_{n33} = Nominal major bending strength,

M_{p33} = Major plastic moment, $Z_{33}F_y \le 1.5 S_{33}F_y$ (LRFD F1.1)

M_{r33} = Major limiting buckling moment, $R_e S_{33} F_y$ (LRFD Table A-F1.1)

λ = Web slenderness parameter,

λ_p = Largest value of λ for which $M_n = M_p$

λ_r = Largest value of λ for which buckling in inelastic

R_{PG} = Plate girder bending strength reduction factor

R_e = Hybrid girder factor, and

F_{cr} = Critical compression flange stress, ksi

The web slenderness parameters are computed as follows, where the value of P_u is taken as positive for compression and zero for tension:

$$\lambda = \frac{h_c}{t_w}$$

$$\lambda_p = \begin{cases} \dfrac{640}{\sqrt{F_y}} \left(1 - 2.75 \dfrac{P_u}{\varphi_b P_y} \right) & \text{for } \dfrac{P_u}{\varphi_b P_y} \le 0.125, \\ \dfrac{191}{\sqrt{F_y}} \left(2.33 \dfrac{P_u}{\varphi_b P_y} \right) \ge \dfrac{253}{\sqrt{F_y}} & \text{for } \dfrac{P_u}{\varphi_b P_y} > 0.125, \text{ and} \end{cases}$$

$\lambda_r \qquad \dfrac{970}{\sqrt{F_y}}\left(1 - 0.74\,\dfrac{P_u}{\varphi_b P_y}\right).$

The parameters R_{PG}, R_e, and F_{cr} for slender web sections are calculated in the program as follows:

$$R_{PG} = 1 - \frac{a_r}{1{,}200 + 300a_r}\left(\frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}}\right) \leq 1.0 \qquad \text{(LRFD A-G2-3)}$$

$$R_e = \frac{12 + a_r(2m - m^3)}{12 + 12a_r} \leq 1.0 \text{ (for hybrid sections)} \qquad \text{(LRFD A-G2)}$$

$$R_e = 1.0 \qquad \text{(for non-hybrid section), where (LRFD A-G2)}$$

$$a_r = \frac{\text{web area}}{\text{compression flange area}} \leq 1.0\text{ , and} \qquad \text{(LRFD A-G2)}$$

$$m = \frac{F_y}{\min(F_{cr}, F_y)}\text{, taken as 1.0} \qquad \text{(LRFD A-G2)}$$

In the above expression, R_e is taken as 1, because currently the program deals with only non-hybrid girders.

The critical compression flange stress, F_{cr}, for slender web sections is calculated for limit states of lateral-torsional buckling and flange local buckling for the corresponding slenderness parameter η in the program as follows:

$$F_{cr} = \begin{cases} F_y & \text{if} \quad \eta \leq \eta_p \\ C_p F_y \left[1 - \dfrac{1}{2}\dfrac{\eta - \eta_p}{\eta_r - \eta_p}\right] \leq F_y & \text{if} \quad \eta_p < \eta \leq \eta_r, \text{ (LRFD A-G2-4, 5, 6)} \\ \dfrac{C_{PG}}{\eta^2} & \text{if} \quad \eta > \eta_r \end{cases}$$

The parameters η, η_p, η_r, and C_{PG} for lateral-torsional buckling for slender web I, Channel and Box sections are given as follows:

$$\eta = \frac{L_b}{r_T}, \qquad \text{(LRFD A-G2-7)}$$

η_p = $\dfrac{300}{\sqrt{F_y}}$, (LRFD A-G2-8)

η_r = $\dfrac{756}{\sqrt{F_y}}$, (LRFD A-G2-9)

C_{PG} = 286,000 C_b, and (LRFD A-G2-10)

r_T = radius of gyration of the compression flange plus one-third of the compression portion of the web, and it is taken as $b_f/\sqrt{12}$ in this program.

C_b = a factor that depends on span moment. It is calculated as follows:

$$\frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_c}$$ (LRFD F1-3)

The parameters η, η_p, η_r, and C_{PG} for flange local buckling for slender web I, Channel and Box sections are given as follows:

η = $\dfrac{b}{t}$, (LRFD A-G2-11)

η_p = $\dfrac{65}{\sqrt{F_y}}$, (LRFD A-G2-12)

η_r = $\dfrac{230}{\sqrt{F_y/k_c}}$, (LRFD A-G2-13)

C_{PG} = 26,200 k_c, and (LRFD A-G2-14)

C_b = 1. (LRFD A-G2-15)

T-Sections and Double-Angles

No local buckling is considered for T-sections and Double-angles in this program. If special consideration is required, the user is expected to analyze this separately.

Single Angles

The nominal major and minor bending strengths for Single angles for the limit state of web local buckling are the same as those given for flange local buckling (LRFD SAM 5.1.1). No additional check is considered in this program.

Pipe Sections

The nominal major and minor bending strengths for Pipe sections for the limit state of web local buckling are the same as those given for flange local buckling (LRFD Table A-F1.1). No additional check is considered in this program.

Circular, Rectangular, and General Sections

No web local buckling is required for solid circular shapes and rectangular plates (LRFD Table A-F1.1). No web local buckling is considered in the program for circular, rectangular, and general shapes. If special consideration is required, the user is expected to analyze them separately.

Shear Capacities

The nominal shear strengths are calculated for shears along the geometric axes for all sections. For I, Box, Channel, T, Double angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes do not coincide with their geometric axes.

Major Axis of Bending

The nominal shear strength, V_{n2}, for major direction shears in I-shapes, boxes and channels is evaluated as follows:

For $\frac{h}{t_w} \le \frac{418}{\sqrt{F_y}}$,

$$V_{n2} = 0.6\ F_y A_w, \qquad \text{(LRFD F2-1)}$$

for $\frac{418}{\sqrt{F_y}} < \frac{h}{t_w} \le \frac{523}{\sqrt{F_y}}$,

$$V_{n2} = 0.6\ Fy\ Aw\ \frac{418}{\sqrt{F_y}} / \frac{h}{t_w}, \text{ and} \qquad \text{(LRFD F2-2)}$$

for $\frac{523}{\sqrt{F_y}} < \frac{h}{t_w} \leq 260,,$

$$V_{n2} = 132{,}000 \, \frac{A_w}{\left[h / t_w\right]^2} \qquad \text{(LRFD F2-3 and A-F2-3)}$$

The nominal shear strength for all other sections is taken as:

$$V_{n2} = 0.6\, F_y A_{v2}.$$

Minor Axis of Bending

The nominal shear strength for minor direction shears is assumed as:

$$V_{n3} = 0.6\, F_y A_{v3}.$$

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STEEL FRAME DESIGN AISC-LRFD93

Technical Note 50
Calculation of Capacity Ratios

This Technical Note describes the calculation of capacity ratios when the user selects the AISC-LRFD93 code, including axial and bending stresses and shear stresses.

Overview

In the calculation of the axial force/biaxial moment capacity ratios, first, for each station along the length of the member, the actual member force/moment components are calculated for each load combination. Then the corresponding capacities are calculated. Then, the capacity ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling capacity ratio is then obtained, along with the associated station and load combination. A capacity ratio greater than 1.0 indicates exceeding a limit state.

During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.

Axial and Bending Stresses

The interaction ratio is determined based on the ratio $P_u/(\varphi P_n)$. If P_u is tensile, P_n is the nominal axial tensile strength and $\varphi = \varphi_t = 0.9$; and if Pu is compressive, P_n is the nominal axial compressive strength and $\varphi = \varphi_c = 0.85$, except for angle sections $\varphi = \varphi_c = 0.90$ (LRFD SAM 6). In addition, the resistance factor for bending, $\varphi_b = 0.9$.

For $\frac{P_u}{\varphi P_n} \geq 0.2$, the capacity ration if given as

$$\frac{P_u}{\varphi P_n} + \frac{8}{9}\left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}}\right) \qquad \text{(LRFD H1-1a, SAM 6-1a)}$$

For $\frac{P_u}{\varphi P_n} < 0.2$, the capacity ration if given as

$$\frac{P_u}{2\varphi P_n} + \left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}} \right) \qquad \text{(LRFD H1-1b, SAM 6-1a)}$$

For circular sections, an SRSS (Square Root of Sum of Squares) combination is first made of the two bending components before adding the axial load component instead of the simple algebraic addition implied by the above formulas.

For single-angle sections, the combined stress ratio is calculated based on the properties about the principal axis (LRFD SAM 5.3, 6). For I, Box, Channel, T, Double angle, Pipe, Circular, and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes are determined in the program. For general sections, it is assumed that the section properties are given in terms of principal directions.

Shear Stresses

Similar to the normal stresses, from the factored shear force values and the nominal shear strength values at each station for each of the load combinations, shear capacity ratios for major and minor directions are calculated as follows:

$$\frac{V_{u2}}{\varphi_v V_{n2}}, \text{ and}$$

$$\frac{V_{u3}}{\varphi_v V_{n3}},$$

where $\varphi_v = 0.9$.

For Single-angle sections, the shear stress ratio is calculated for directions along the geometric axis. For all other sections, the shear stress is calculated along the principal axes that coincides with the geometric axes.

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STEEL FRAME DESIGN AISC-LRFD93

Technical Note 51
Input Data

This Technical Note describes the steel frame design input data for AISC-LRFD93. The input can be printed to a printer or to a text file when you click the **File menu > Print Tables > Steel Frame Design** command. A printout of the input data provides the user with the opportunity to carefully review the parameters that have been input into the program and upon which program design is based. Further information about using the Print Design Tables Form is provided at the end of this Technical Note.

Input Data

The program provides the printout of the input data in a series of tables. The column headings for input data and a description of what is included in the columns of the tables are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Material Property Data	
Material Name	Steel, concrete or other.
Material Type	Isotropic or orthotropic.
Design Type	Concrete, steel or none. Postprocessor available if steel is specified.
Material Dir/Plane	"All" for isotropic materials; specify axis properties define for orthotropic.
Modulus of Elasticity	
Poisson's Ratio	
Thermal Coeff	
Shear Modulus	
Material Property Mass and Weight	
Material Name	Steel, concrete or other.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Mass Per Unit Vol	Used to calculate self mass of the structure.
Weight Per Unit Vol	Used to calculate the self weight of the structure.
Material Design Data for Steel Materials	
Material Name	Steel.
Steel FY	Minimum yield stress of steel.
Steel FU	Maximum tensile stress of steel.
Steel Cost ($)	Cost per unit weight used in composite beam design if optimum beam size specified to be determined by cost.
Material Design Data for Concrete Materials	
Material Name	Concrete.
Lightweight Concrete	Check this box if this is a lightweight concrete material.
Concrete FC	Concrete compressive strength.
Rebar FY	Bending reinforcing yield stress.
Rebar FYS	Shear reinforcing yield stress.
Lightwt Reduc Fact	Define reduction factor if lightweight concrete box checked. Usually between 0.75 ad 0.85.
Frame Section Property Data	
Frame Section Name	User specified or auto selected member name.
Material Name	Steel, concrete or none.
Section Shape Name or Name in Section Database File	Name of section as defined in database files.
Section Depth	Depth of the section.
Flange Width Top	Width of top flange per AISC database.
Flange Thick Top	Thickness of top flange per AISC database.
Web Thick	Web thickness per AISC database.
Flange Width Bot	Width of bottom flange per AISC database.
Flange Thick Bot	Thickness of bottom flange per AISC database.
Section Area	

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Torsional Constant	
Moments of Inertia	I33, I22
Shear Areas	A2, A3
Section Moduli	S33, S22
Plastic Moduli	Z33, Z22
Radius of Gyration	R33, R22
Load Combination Multipliers	
Combo	Load combination name.
Type	Additive, envelope, absolute, or SRSS as defined in **Define > Load Combination**.
Case	Name(s) of case(s) to be included in this load combination.
Case Type	Static, response spectrum, time history, static nonlinear, sequential construction.
Factor	Scale factor to be applied to each load case.
Code Preferences	
Phi_bending	Resistance factor for bending.
Phi_tension	Resistance factor for tension.
Phi_compression	Resistance factor for compression.
Phi_shear	Resistance factor for shear.
Beam Steel Stress Check Element Information	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member section assigned.
Framing Type	Moment frame or braced frame.
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ratio Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
K Minor	Effective length factor.
Beam Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
CM Major	As defined in AISC-LRFD specification Chapter C.
CM Minor	As defined in AISC-LRFD specification Chapter C.
Cb Factor	As defined in AISC-LRFD specification Chapter F.
B1 Major	As defined in AISC-LRFD specification Chapter C.
B1 Minor	As defined in AISC-LRFD specification Chapter C.
B2 Major	As defined in AISC-LRFD specification Chapter C.
B2 Minor	As defined in AISC-LRFD specification Chapter C.
Beam Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier
phi*Pnc	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
phi*Pnt	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
phi*Mn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Mn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Vn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
phi*Vn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
Column Steel Stress Check Element Information	
Story Level	Name of the story level.
Column Line	Column line identifier.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
Section ID	Name of member section assigned.
Framing Type	Moment frame or braced frame.
RLLF Factor	Live load reduction factor.
L_Ratio Major	Ratio of unbraced length divided by the total member length.
L_Ration Minor	Ratio of unbraced length divided by the total member length.
K Major	Effective length factor.
K Minor	Effective length factor.
Column Steel Moment Magnification Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
CM Major	As defined in AISC-LRFD specification Chapter C.
CM Minor	As defined in AISC-LRFD specification Chapter C.
Cb Factor	As defined in AISC-LRFD specification Chapter F.
B1 Major	As defined in AISC-LRFD specification Chapter C.
B1 Minor	As defined in AISC-LRFD specification Chapter C.
B2 Major	As defined in AISC-LRFD specification Chapter C.
B2 Minor	As defined in AISC-LRFD specification Chapter C.
Column Steel Allowables & Capacities Overwrites	
Story Level	Name of the story level.
Column Line	Column line identifier.
phi*Pnc	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter E.
phi*Pnt	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter D.
phi*Mn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.
phi*Mn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F and G.

Table 1 Steel Frame Design Input Data

COLUMN HEADING	DESCRIPTION
phi*Vn Major	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.
phi*Vn Minor	If zero, as defined for Material Property Data used and per AISC-LRFD specification Chapter F.

Using the Print Design Tables Form

To print steel frame design input data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Input Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design input data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.

Technical Note 52
Output Details

This Technical Note describes the steel frame design output for AISC-LRFD93 that can be printed to a printer or to a text file. The design output is printed when you click the **File menu > Print Tables > Steel Frame Design** command and select Output Summary on the Print Design Tables form. Further information about using the Print Design Tables form is provided at the end of this Technical Note.

The program provides the output data in a table. The column headings for output data and a description of what is included in the columns of the table are provided in Table 1 of this Technical Note.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Beam Steel Stress Check Output	
Story Level	Name of the story level.
Beam Bay	Beam bay identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces the maximum load/resistance ratio.
Ratio	Ratio of acting load to available resistance.
Axl	Ratio of acting axial load to available axial resistance.
B33	Ratio of acting bending moment to available bending resistance about the 33 axis.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
B22	Ratio of acting bending moment to available bending resistance about the 22 axis.
Shear22	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting shear divided by available shear resistance.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear divided by available shear resistance.
Column Steel Stress Check Output	
Story Level	Name of the story level.
Column Line	Column line identifier.
Section ID	Name of member sections assigned.
Moment Interaction Check	
Combo	Name of load combination that produces maximum stress ratio.
Ratio	Ratio of acting stress to allowable stress.
AXL	Ratio of acting axial stress to allowable axial stress.
B33	Ratio of acting bending stress to allowable bending stress about the 33 axis.
B22	Ratio of acting bending stress to allowable bending stress about the 22 axis.

Table 1 Steel Frame Design Output

COLUMN HEADING	DESCRIPTION
Shear22	
Combo	Load combination that produces the maximum shear parallel to the 22 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.
Shear33	
Combo	Load combination that produces the maximum shear parallel to the 33 axis.
Ratio	Ratio of acting shear stress divided by allowable shear stress.

Using the Print Design Tables Form

To print steel frame design output data directly to a printer, use the **File menu > Print Tables > Steel Frame Design** command and click the Output Summary check box on the Print Design Tables form. Click the **OK** button to send the print to your printer. Click the **Cancel** button rather than the **OK** button to cancel the print. Use the **File menu > Print Setup** command and the **Setup>>** button to change printers, if necessary.

To print steel frame design output data to a file, click the Print to File check box on the Print Design Tables form. Click the **Filename** button to change the path or filename. Use the appropriate file extension for the desired format (e.g., .txt, .xls, .doc). Click the **Save** buttons on the Open File for Printing Tables form and the Print Design Tables form to complete the request.

Note:

The **File menu > Display Input/Output Text Files** command is useful for displaying output that is printed to a text file.

The Append check box allows you to add data to an existing file. The path and filename of the current file is displayed in the box near the bottom of the Print Design Tables form. Data will be added to this file. Or use the **Filename** button to locate another file, and when the Open File for Printing Tables caution box appears, click Yes to replace the existing file.

If you select a specific frame element(s) before using the **File menu > Print Tables > Steel Frame Design** command, the Selection Only check box will be checked. The print will be for the selected beam(s) only.