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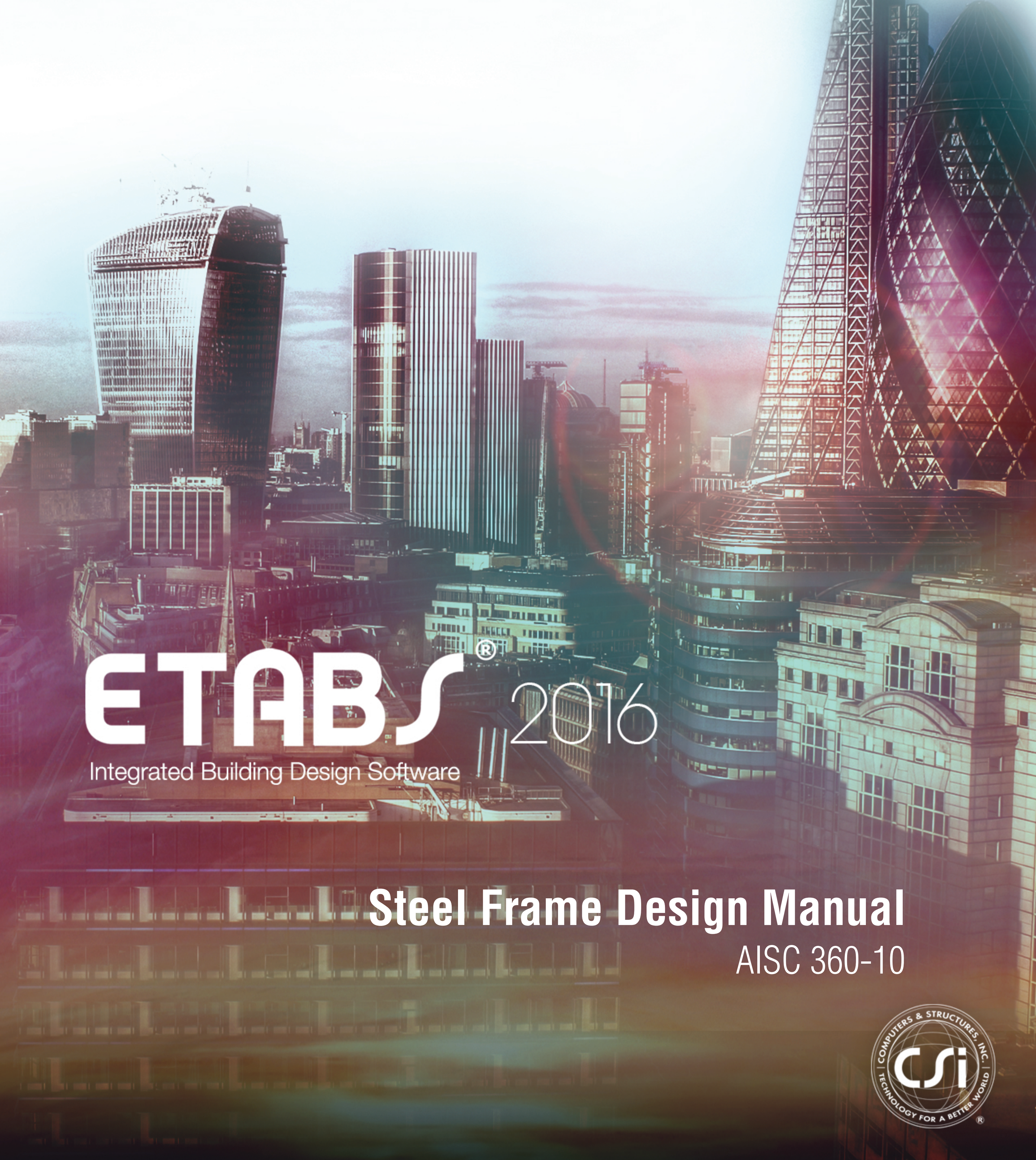

ETABS® 2016

Integrated Building Design Software

Steel Frame Design Manual

AISC 360-10

Steel Frame Design Manual

AISC 360-10

For ETABS® 2016

ISO ETA122815M9 Rev. 1
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Chapter 1
Introduction

The design/check of steel frames is seamlessly integrated within the program. Initiation of the design process, along with control of various design parameters, is accomplished using the Design menu. Automated design at the object level is available for any one of a number of user-selected design codes, as long as the structures have first been modeled and analyzed by the program. Model and analysis data, such as material properties and member forces, are recovered directly from the model database, and are used in the design process in accordance with the user defined or default design settings. As with all design applications, the user should carefully review all of the user options and default settings to ensure that the design process is consistent with the user's expectations. The AISC 360-10 steel frame design options include the use of the Direct Analysis Method. The software is well suited to make use of the Direct Analysis Method because it can capture the second-order P-Delta and P-δ effects, provided the user specifies that a nonlinear P-Delta analysis be performed.

Chapter 2 addresses prerequisites related to modeling and analysis for a successful design in accordance with "AISC 360-10." Chapter 3 provides detailed descriptions of the specific requirements as implemented in "AISC 360-10." Chapter 4 provides detailed descriptions of the specific requirements for seismic loading as required by the specification in ANSI/AISC 341-10 code. The appendices provide details on various topics referenced in this manual. The user also should review the *AISC Direct Analysis Method Practical Guide*.

1.1 Load Combinations and Notional Loads

The design is based on a set of user-specified loading combinations. However, the program provides default load combinations for each supported design code. If the default load combinations are acceptable, no definition of additional load combinations is required. The Direct Analysis Method requires that a notional load, $N = 0.002Y_i$, where Y_i is the gravity load acting at level i, be applied to account for the destabilizing effects associated with the initial imperfections and other conditions that may induce sway not explicitly modeled in the structure. The user must be aware that notional loads must be defined and assigned by the user. Currently, the software creates design combinations that include notional loads and gravity loads only. If the user needs notional loads that include combinations containing lateral loads, the user must define such combinations manually. The automation of combinations, including notional loads, is currently limited to gravity loads only. Design load combinations of notional loads acting together with lateral loads currently are *NOT* automated by the software.

1.2 Stress Check

Steel frame design/check consists of calculating the flexural, axial, and shear forces or stresses at several locations along the length of a member, and then comparing those calculated values with acceptable limits. That comparison produces a demand/capacity ratio, which typically should not exceed a value of one if code requirements are to be satisfied. The program follows the same review procedures whether it is checking a user-specified shape or a shape selected by the program from a predefined list. The program also checks the requirements for the beam-column capacity ratio, checks the capacity of the panel zone, and calculates the doubler plate and continuity plate thickness, if needed. The program does not do the connection design. However, it calculates the design basis forces for connection design.

Program output can be presented graphically on the model, in tables for both input and output data, or in calculation sheets prepared for each member. For each presentation method, the output is in a format that allows the engineer to quickly study the stress conditions that exist in the structure, and in the event the member is not adequate, aid the engineer in taking appropriate remedial measures, including altering the design member without re-running the entire analysis.

The program supports a wide range of steel frame design codes, including many national building codes. This manual is dedicated to the use of the menu option “AISC 360-10.” This option covers the “ANSI/AISC 360-10 Specification for Structural Steel Buildings” (AISC 2010a, b), and the “ANSI/ AISC 341-10 Seismic Provisions for Structural Steel Buildings” (AISC 2010c) codes.

The implementation covers loading and load combinations from “ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures” (ASCE 2010), and also special requirements from “IBC 2012 International Building Code” (IBC 2012). Both LRFD (Load and Resistance Factor Design) and ASD (Allowable Strength Design) codes are included in this implementation under the same AISC 360-10 code name. The LRFD and ASD are available as two options in the program’s preferences feature. In both cases, the strengths are calculated in the nominal levels. The phi (LRFD) and Omega (ADS) factors are applied during calculation of demand/capacity ratios only. The design codes supported under “AISC 360-10” are written in kip-inch units. All the associated equations and requirements have been implemented in the program in kip-in units. The program has been enabled with unit conversion capability. This allows the users to enjoy the flexibility of choosing any set of consistent units during creating and editing models, exporting and importing the model components, and reviewing the design results.

1.3 Direct Analysis Method vs. Effective Length Method

The Direct Analysis Method described in AISC 360-10, Chapter C, is substantially different from previous design methods supported by AISC. The user should be knowledgeable about the Design for Stability (Chapter C) requirements and the requirements pertaining to consideration of the geometric imperfections, stiffness reductions, and the P-Δ and P-δ effects. Several methods for consideration of the second-order effects are available to the users. Each of these are described in detail in a subsequent section (see User Options in this chapter) and in the Steel Frame Design Preferences, Appendix B of this manual. Alternatively, if the user desires to use a more traditional design method, the Effective Length method can be specified using the Design Preferences.

1.3.1 Effective Length Method

For structures exhibiting small second-order effects, the effective length method may be suitable. The effective length approach relies on two main assumptions, namely, that the structural response is elastic and that all columns buckle simultaneously. The effective length method also relies on a calibrated approach to account for the differences between the actual member response and the 2nd-order elastic analysis results. The calibration is necessary because the 2nd-order elastic analysis does not account for the effects of distributed yielding and geometric imperfections. Since the interaction equations used in the effective length approach rely on the calibration corresponding to a 2nd-order elastic analysis of an idealized structure, the results are not likely representative of the actual behavior of the structure. However, the results are generally conservative. In the AISC 360-10 code, the effective length method is allowed provided the member demands are determined using a second-order analysis (either explicit or by amplified first-order analysis) and notional loads are included in all gravity load combinations (AISC Appendix 7). K-factors must be calculated to account for buckling (except for braced frames, or where $\Delta_2 / \Delta_1 \leq 1.5$, K = 1.0) (AISC App. 7.2).

1.3.2 Direct Analysis Method

The Direct Analysis Method is expected to more accurately determine the internal forces of the structure, provided care is used in the selection of the appropriate methods used to determine the second-order effects, notional load effects and appropriate stiffness reduction factors as defined in AISC C2. Additionally, the Direct Analysis Method does not use an effective length factor other than K = 1.0. The rational behind the use of K = 1.0 is that proper consideration of the second-order effects (P-Δ and P-δ), geometric imperfections (using notional loads) and inelastic effects (applying stiffness reductions) better accounts for the stability effects of a structure than the earlier Effective Length methods.

1.4 User Options

In addition to offering ASD and LRFD design, the Design Options menu provides seven analysis methods for design, as follows:

- General Second Order Elastic Analysis (AISC C1.2)
- Second Order Analysis by Amplified First Order Analysis (AISC C1.2, App. 7.2, App. 8.2)
- Limited First Order Elastic Analysis (AISC C1.2, App. 7.3)
- Direct Analysis Method with General Second Order Analysis and Variable Factor Stiffness Reduction (AISC C1, C2)
- Direct Analysis Method with General Second Order Analysis and Fixed Factor Stiffness Reduction (AISC C1, C2)
- Direct Analysis Method with Amplified First Order Analysis and Variable Factor Stiffness Reduction (AISC C1, C2)
- Direct Analysis Method with Amplified First Order Analysis and Fixed Factor Stiffness Reduction (AISC C1, C2)

These options are explained in greater detail in Chapter 2. The first three options make use of the effective length approach to determine the effective length factors, K. The four options available for the Direct Design Method differ in the use of a variable or fixed stiffness reduction factor and the method used to capture the second-order effects. All four Direct Analysis Methods options use an effective length factor, $K = 1.0$.

1.5 Non-Automated Items in Steel Frame Design

Currently, the software does not automate the following:

- Notional loads combinations that include lateral wind and quake loads
- The validity of the analysis method. The user must verify the suitability of the specified analysis method used under the User Options described in the

preceding sections. The AISC code requires, for instance, that the Direct Analysis Method be used when a ratio of the second order displacements to the first order displacements exceeds 1.5 (AISC C1.2, App. 7.2.1(2), App. 7.3.1(2)). This check currently must be performed by the user.

- P-Δ analysis. Since many different codes are supported by the software and not all require a P-Δ analysis, the user must specify that a P-Δ analysis be performed during the analysis phase so that the proper member forces are available for use in the design phase. See the *AISC Direct Analysis Method Practical Guide* for additional information.

Chapter 2
Design Algorithms

This chapter provides an overview of the basic assumptions, design preconditions, and some of the design parameters that affect the design of steel frames.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- Reference to the ANSI/AISC 360-10 code is identified with the prefix "**AISC**."
- Reference to the ANSI/AISC 341-10 code is identified with the prefix "**AISC SEISMIC**" or sometimes "**SEISMIC**" only.
- Reference to the ASCE/SEI 7-10 code is identified with the prefix "**ASCE**."
- Reference to the IBC 2012 code is identified with the prefix "**IBC**."

2.1 Check and Design Capability

The program has the ability to check adequacy of a section (shape) in accordance with the requirements of the selected design code. Also the program can automatically choose (i.e., design) the optimal (i.e., least weight) sections from a predefined list that satisfies the design requirements.

To check adequacy of a section, the program checks the demand/capacity (D/C) ratios at a predefined number of stations for each design load combination. It calculates the envelope of the D/C ratios. It also checks the other requirements on a pass or fail basis. If the capacity ratio remains less than or equal to the D/C ratio limit, which is a number close to 1.0, and if the section passes all the special requirements, the section is considered to be adequate, else the section is considered to be failed. The D/C ratio limit is taken as 0.95 by default. However, this value can be overwritten in the Preferences (see Chapter 3).

To choose (design) the optional section from a predefined list, the program first orders the list of sections in increasing order of weight per unit length. Then it starts checking each section from the ordered list, starting with the one with least weight. The procedure of checking each section in this list is exactly the same as described in the preceding paragraph. The program will evaluate each section in the list until it finds the least weight section that passes the code checks. If no section in the list is acceptable, the program will use the heaviest section but flag it as being overstressed.

To check adequacy of an individual section, the user must assign the section using the **Assign** menu. In that case, both the analysis and design sections will be changed.

To choose the optimal section, the user must first define a list of steel sections, the *Auto Select* sections list. The user must next assign this list, in the same manner as any other section assignment, to the frame members to be optimized. The program will use the median section by weight when doing the initial analysis. Check the program Help for more information about defining and assigning Auto Select Section lists.

2.2 Design and Check Stations

For each design combination, steel frame members (beams, columns, and braces) are designed (optimized) or checked at a number of locations (stations) along the length of the object. The stations are located at equally spaced segments along the clear length of the object. By default, at least three stations will be located in a column or brace member, and the stations in a beam will be spaced at most 2 feet apart (0.5 m if the model has been created in metric units). The user can overwrite the number of stations in an object before the analysis is run and refine the design along the length of a member by requesting more stations. Refer to the program Help for more information about specifying the number of stations in an object.

2.3 Demand/Capacity Ratios

Determination of the controlling demand/capacity (D/C) ratios for each steel frame member indicates the acceptability of the member for the given loading conditions. The steps for calculating the D/C ratios are as follows:

- The factored forces are calculated for axial, flexural, and shear at each defined station for each design combination. The bending moments are calculated about the principal axes. For I-Shape, Box, Channel, T-Shape, 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 forces are calculated for directions along the geometric axes. For all other sections, the program calculates the shear forces along the geometric and principal axes.

- The nominal strengths are calculated for compression, tension, bending and shear based on the equations provided later in this manual. For flexure, the nominal strengths are calculated based on the principal axes of bending. For the I-Shape, Box, Channel, Circular, Pipe, T-Shape, 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 nominal strength for shear is calculated along the geometric axes for all sections. For I-Shape, Box, Channel, T-Shape, 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.

- Factored forces are compared to nominal strengths to determine D/C ratios. In either case, design codes typically require that the ratios not exceed a value of one. A capacity ratio greater than one indicates a member that has exceeded a limit state.

2.4 Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs to be checked. The program creates a number of default design load combinations for steel frame design. Users can add their own design combinations as well as modify or delete the program default design 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. The scale factors are applied to the forces and moments from the load cases to form the factored design forces and moments for each design load combination.

For normal loading conditions involving static dead load (DL), live load (LL), roof live load (RL), snow load (SL), wind load (WL), earthquake load (EL), notional load (NL), and dynamic response spectrum load (EL), the program has built-in default design combinations for the design code. These are based on the code recommendations.

The default design combinations assume all load cases declared as dead or live to be additive. However, each load case declared as wind, earthquake, or response spectrum cases, is assumed to be non-additive with other loads and produces multiple lateral combinations. Also static wind, earthquake and notional load responses produce separate design combinations with the sense (positive or negative) reversed. The notional load patterns are added to load combinations involving gravity loads only. The user is free to modify the default design preferences to include the notional loads for combinations involving lateral loads.

For other loading conditions involving moving load, time history, pattern live load, separate consideration of roof live load, snow load, and the like, the user must define the design load combinations in lieu of or in addition to the default design load combinations. If notional loads are to be combined with other load combinations involving wind or earthquake loads, the design load combinations need to be defined in lieu of or in addition to the default design load combinations.

For multi-valued design combinations, such as those involving response spectrum, time history, moving loads and envelopes, where any correspondence between forces is lost, the program automatically produces sub-combinations using the maxima/minima values of the interacting forces. Separate combinations with negative factors for response spectrum load cases are not required because the program automatically takes the minima to be the negative of the maxima response when preparing the sub-combinations described previously.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

2.5 Second Order P-Delta Effects

The AISC 360-10 steel frame design options include the use of the Direct Analysis Method. The software is well suited to make us of the Direct Analysis Method because each program can capture the second-order P-Δ and P-δ effects, provided the user specifies that a nonlinear P-Delta analysis be performed.

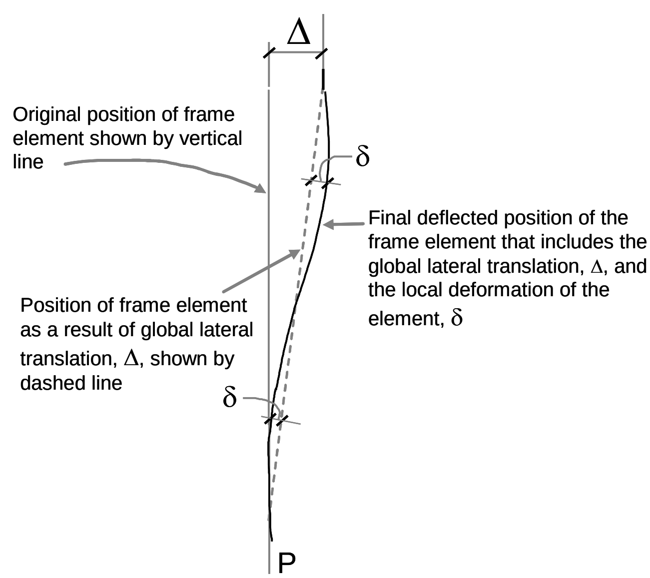


Figure 2-1 *System sway and element order effects*

For more details about the program capabilities and limitations, see Appendix A.

2.6 Analysis Methods

The code requires that stability shall be provided for the structure as a whole and for each of the elements. Any method of analysis that considers the influence of second order effects of $P\text{-}\Delta$ and $P\text{-}\delta$, geometric imperfections, out-of-plumbness, and member stiffness reduction due to residual stresses are permitted by the code. The effects of geometric imperfection and out-of-plumbness generally are captured by the use of notional loads. The effect of axial, shear and flexural deformations and the effects of residual stresses on the member stiffness reduction has been considered in a specialized method called "Direct Analysis Method." This method can come in different incarnations (formats) according to the choice of the engineer as allowed in the code.

The program offers the user seven analysis options for design:

Direct Analysis Method

- General Second Order Elastic Analysis with
  - τ_b variable (user option 1, Default)
  - τ_b fixed (user option 2)
- Amplified First Order Elastic Analysis with
  - τ_b variable (user option 3)
  - τ_b fixed (user option 4)

Equivalent Length Method

- General Second Order Elastic Analysis (AISC C1.2, App. 7.2) (user option 5)
- Amplified First Order Elastic Analysis (AISC C1.2, App. 8.2) (user option 6)

Limited First-Order Analysis (AISC C1.2, App. 7.3) (user option 7)

A summary of all of the user options and requirements is provided in Table 2-1. The main difference between the various options concerns the use of the Direct Analysis Method or the Equivalent Length Method. Within each of the categories, the user can choose the method to calculate the second-order effects, namely, by a *General Second Order Analysis* or an *Amplified First-Order Analysis*. When the amplified first-order analysis is used, the force amplification factors, B_1 and B_2 (AISC App. 8.2), are needed. The B_1 factor is calculated by the program; however, the B_2 factor is not. The user will need to provide this value using the overwrite options that are described in Appendix B.

When the user selects one of the options available under the Direct Analysis Method, the user must further choose how the stiffness reduction factors for EI and AE are to be considered. For options 1 and 3, Table 2-1, the stiffness reduction factors (τ_b) are variable because they are functions of the axial force in the members, while for methods 2 and 4, the stiffness reduction factors are fixed (0.8), and not a function of axial force. If the user desires, the stiffness reduction factors (τ_b) can be overwritten. When options 2 and 4 are used, a higher notional load coefficient (0.003) must be used compared to methods 1 and 3 for which the notional load coefficient is 0.002. Also, all the direct analysis methods (methods 1 through 4) allow use of K-factors for sway condition (K_2) to be equal to 1, which is a drastic simplification over the other effective length method.

The AISC requirements to include notional loads are also summarized in Table 2-1. The notional load coefficients (AISC C2.2b) are summarized as well. The program automates creation of notional load combinations for all gravity loads but does not automate the creation of notional load combinations that include lateral wind or seismic loads. Combinations for notional loads with lateral loads are required for the Direct Analysis Method when the $\Delta_{2nd}/\Delta_{1st}$ exceeds 1.7 (AISC E2.2b(4)). Additionally, combinations for notional loads with lateral loads are required if the Limited First Order Analysis, option 7, is used (AISC App. 7.3.2).

The Limited First Order Analysis, option 7, does not include the secondary $P\text{-}\Delta$ and $P\text{-}\delta$ effects. This method has very limited applicability and might be appropriate only when the axial forces in the columns are very small compared to their Euler buckling capacities.

When using the LRFD provision, the actual load combinations are used for second order P-Δ effects. When using the ASD provision, the load combinations are first amplified by 1.6 before the P-Δ analysis and then the results are reduced by a factor of $(1/1.6)$ (AISC C2.1(4)).

Table 2-1 The Essentials and Limitations of the Design Analysis Methods

Direct Analysis Method			
Option	**Variable**	**Limitation or Applicability**	**Essentials of the Method**
General Second Order Analysis	Variable Factor Stiffness Reduction	No limitation	2nd Order Analysis Reduced stiffness $EI^* = 0.8\tau_b EI$ $EA^* = 0.8EA$ $\tau_b = \begin{cases} 1.0 \text{ for } \frac{\alpha P_r}{P_y} \leq 0.5 \\ 4\left(\frac{\alpha P_r}{P_y}\right)\left(1-\frac{\alpha P_r}{P_y}\right) \text{ for } \frac{\alpha P_r}{P_y} \geq 0.5 \end{cases}$ B_1 and B_2 not used $K_2 = 1$ (used for P_n) Notional load with all combos, except for $\Delta_{2nd}/\Delta_{1st} \leq 1.7$ for which notional load with gravity combos only Notional load coefficient = 0.002 (typically)
	Fixed Factor Stiffness Reduction	No limitation	2nd Order Analysis Reduced stiffness $EI^* = 0.8\tau_b EI$ $EA^* = 0.8EA$ $\tau_b = 1.0$ B_1 and B_2 not used $K_2 = 1$ (used for P_n) Notional load with all combos, except for $\Delta_{2nd}/\Delta_{1st} \leq 1.7$ for which notional load with gravity combos only Notional load coefficient = 0.003 (typically)
Amplified First Order Analysis	Variable Factor Stiffness Reduction	No limitation	1st Order Analysis Reduced Stiffness $EI^* = 0.8\tau_b EI$ $EA^* = 0.8EA$ $\tau_b = \begin{cases} 1.0 \text{ for } \frac{\alpha P_r}{P_y} \leq 0.5 \\ 4\left(\frac{\alpha P_r}{P_y}\right)\left(1-\frac{\alpha P_r}{P_y}\right) \text{ for } \frac{\alpha P_r}{P_y} \geq 0.5 \end{cases}$ $K_1 = 1$ for B_1 $K_2 = 1$ for P_n and B_2 Notional load with all combos, except for $\Delta_{2nd}/\Delta_{1st} \leq 1.7$ for which notional load with gravity combos only Notional load coefficient = 0.002 (typically)

Table 2-1 The Essentials and Limitations of the Design Analysis Methods

Direct Analysis Method

Option	Variable	Limitation or Applicability	Essentials of the Method
Amplified First Order Analysis	Fixed Factor Stiffness Reduction	No limitation	2nd Order Analysis Reduced stiffness $EI^* = 0.8\tau_b EI$ $EA^* = 0.8EA$ $\tau_b = 1.0$ $K_2 = 1$ (used for P_n) Notional load with all combos, except for $\Delta_{2nd}/\Delta_{1st} \le 1.7$ for which notional load with gravity combos only Notional load coefficient = 0.003 (typically)

Effective Length Method

Option	Limitation or Applicability	Essentials of the Method
General Second Order Elastic Analysis	$\frac{\Delta_{2nd}}{\Delta_{1st}} \le 1.5$ (for all stories) $\frac{\alpha P_r}{P_y} = any$ (for all columns)	2nd Order Analysis Unreduced Stiffness $K = K_2$ (used for P_n) Notional load with gravity combos only Notional load coefficient = 0.002 (typically) $B_1 = 1$ $B_2 = 1$
Amplified First Order Analysis	$\frac{\Delta_{2nd}}{\Delta_{1st}} \le 1.5$ (for all stories) $\frac{\alpha P_r}{P_y} = any$ (for all columns)	1st Order Analysis Unreduced stiffness K_1 for B_1 K_2 for B_2 $K = K_2$ (used for P_n) Notional load with gravity combos only Notional load with coefficient = 0.002 (typically) Use of B_1 and B_2

Limited First Order Analysis

Limited First Order Elastic Analysis	$\frac{\Delta_{2nd}}{\Delta_{1st}} \le 1.5$ (for all stories) $\frac{\alpha P_r}{P_y} \le 0.5$ (for all columns)	1st Order Analysis Unreduced stiffness K_2 for P_n (not B_2) Notional load with all combos Notional load with coefficient = $(2)\alpha\left(\frac{\Delta}{L}\right) \ge 0.0042$

The program has several limitations that have been stated in Section 1.5 and the preceding paragraphs. Additionally, the user must be aware that it is possible to choose a design option that violates certain provisions of the AISC code that will not be identified by the program. The limitation for the use of the effective length method, namely, the requirement that $\frac{\Delta_{2nd}}{\Delta_{1st}} \leq 1.5$ and $\frac{\alpha P_r}{P_e}$ must be verified by the user. To assist users to in making validity checks, the ratio $\frac{\alpha P_r}{P_e}$ and τ are now reported in tabular form for each member.

2.7 Notional Load Patterns

Notional loads are lateral loads that are applied at each framing level and are specified as a percentage of the gravity loads applied at that level. They are intended to account for the destabilizing effects of out-of-plumbness, geometric imperfections, inelasticity in structural members, and any other effects that could induce sway and that are not explicitly considered in the analysis.

The program allows the user to create a Notional Load pattern as a percentage of the previously defined gravity load pattern to be applied in one of the global lateral directions: X or Y. The user can define more than one notional load pattern associated with one gravity load by considering different factors and different directions. In the ANSI/AISC 360-10 code, the notional loads are typically suggested to be 0.2% (or 0.002) (AISC C2.2b(3)), a factor referred to as the notional load coefficient in this document. The notional load coefficient can be 0.003 (AISC C2.3(3)). In some cases, it can be a function of second order effects measured by relative story sway (AISC App. 7.3(2)). The code also gives some flexibility to allow the engineer-of-record to apply judgment.

The notional load patterns should be considered in combination with appropriate factors, appropriate directions, and appropriate senses. Some of the design analysis methods need the notional loads to be considered only in gravity load combinations (AISC C2.2b(4)), and some of the methods need the notional loads to be considered in all the design load combinations (AISC C2.2b(4)). For a complete list, see Table 2-1 in the preceding “Second Order Effects and Analysis Methods” section of this chapter.

Currently, the notional loads are not automatically included in the default design load combinations that include lateral loads. However, the user is free to modify the default design load combinations to include the notional loads with appropriate factors and in appropriate load combinations.

2.8 Member Unsupported Lengths

The column unsupported lengths are required to account for column slenderness effects for flexural buckling and for lateral-torsional buckling. The program automatically determines the unsupported length ratios, which are specified as a fraction of the frame object length. These ratios times the frame object lengths give the unbraced lengths for the member. These ratios can also be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

Two unsupported lengths, l_{33} and l_{22}, as shown in Figure 2-2 are to be considered for flexural buckling. These are the lengths between support points of the member 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_{LTB}, not shown in the figure, 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 members, 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 member support points and evaluates the corresponding unsupported length.

It is possible for the unsupported length of a frame object to be evaluated by the program as greater than the corresponding member 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.

By default, the unsupported length for lateral-torsional buckling, l_{LTB}, is taken to be equal to the l_{22} factor. Similar to l_{22} and l_{33}, l_{LTB} can be overwritten.

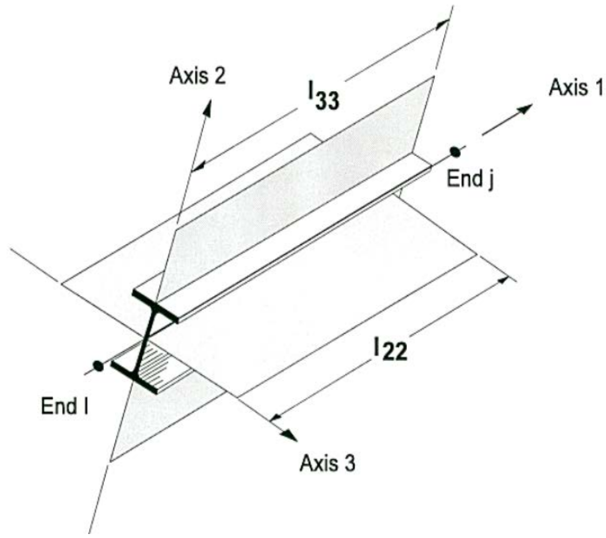


Figure 2-2 Unsupported lengths l_{33} ***and*** l_{22}

2.9 Effects of Breaking a Member into Multiple Elements

The preferred method is to model a beam, column or brace member as one single element. However, the user can request that the program break a member internally at framing intersections and at specified intervals. In this way, accuracy in modeling can be maintained, at the same time design/check specifications can be applied accurately. There is special emphasis on the end forces (moments in particular) for many different aspects of beam, column and brace design. If the member is manually meshed (broken) into segments, maintaining the integrity of the design algorithm becomes difficult.

Manually, breaking a column member into several elements can affect many things during design in the program.

1. The unbraced length: The unbraced length is really the unsupported length between braces. If there is no intermediate brace in the member, the unbraced length is typically calculated automatically by the program from the top of the flange of the beam framing the column at bottom to the bottom of the flange of the beam framing the column at the top. The automatically

calculated length factor typically becomes less than 1. If there are intermediate bracing points, the user should overwrite the unbraced length factor in the program. The user should choose the critical (larger) one. Even if the user breaks the element, the program typically picks up the unbraced length correctly, provided that there is no intermediate bracing point.

2. K-factor: Even if the user breaks the member into pieces, the program typically can pick up the K-factors correctly. However, sometimes it can not. The user should note the K-factors. All segments of the member should have the same K-factor and it should be calculated based on the entire member. If the calculated K-factor is not reasonable, the user can overwrite the K-factors for all the segments.

3. C_m factor: The C_m factor should be based on the end moments of unbraced lengths of each segment and should not be based on the end moments of the member. The program already calculates the C_m factors based on the end moments of unbraced lengths of each segment. If the break-up points are the brace points, no action is required by the user. If the broken segments do not represent the brace-to-brace unsupported length, the program calculated C_m factor is conservative. If this conservative value is acceptable, no action is required by the user. If it is not acceptable, the user can calculate the C_m factor manually for the critical combination and overwrite its value for that segment.

4. C_b factor: The logic is similar to that for the C_m factor.

5. B_1 factor: This factor amplifies the factored moments for the P-δ effect. In its expression, there are the C_m factor and the Euler Buckling capacity P_e. If the user keeps the unbraced length ratios (l_{33} and l_{22}) and the K-factors (K_{33} and K_{22}) correct, the B_1 factor would be correct. If the axial force is small, the B_1 factor can be 1 and have no effect with respect to modeling the single segment or multi-segment element.

6. B_2 factor: The program does not calculate the B_2 factor. The program assumes that the user turns on the P-Δ. In such cases, B_2 can be taken as equal to 1. That means the modeling with one or multiple segments has no effect on this factor.

If the user models a column with a single element and makes sure that the L-factors and K-factors are correct, the effect of B_1 and B_2 will be picked up correctly. The factors C_m and C_b will be picked up correctly if there is no intermediate bracing point. The calculated C_m and C_b factors will be slightly conservative if there are intermediate bracing points.

If the user models a column with multiple elements and makes sure that L-factors and K-factors are correct, the effect of B_1 and B_2 will be picked up correctly. The factors C_m and C_b will be picked up correctly if the member is broken at the bracing points. The calculated C_m and C_b factors will be conservative if the member is not broken at the bracing points.

2.10 Effective Length Factor (*K*)

The effective length method for calculating member axial compressive strength has been used in various forms in several stability based design codes. The method originates from calculating effective buckling lengths, KL, and is based on elastic/inelastic stability theory. The effective buckling length is used to calculate an axial compressive strength, P_n, through an empirical column curve that accounts for geometric imperfections, distributed yielding, and residual stresses present in the cross-section.

There are two types of K-factors in the ANSI/AISC 360-10 code. The first type of K-factor is used for calculating the Euler axial capacity assuming that all of the beam-column joints are held in place, i.e., no lateral translation is allowed. The resulting axial capacity is used in calculation of the B_1 factor. This K-factor is named as K_1 in the code. This K_1 factor is always less than 1 and is not calculated. By default the program uses the value of 1 for K_1. The program allows the user to overwrite K_1 on a member-by-member basis.

The other K-factor is used for calculating the Euler axial capacity assuming that all the beam-column joints are free to sway, i.e., lateral translation is allowed. The resulting axial capacity is used in calculating P_n. This K-factor is named as K_2 in the code. This K_2 is always greater than 1 if the frame is a sway frame. The program calculates the K_2 factor automatically based on sway condition. The program also allows the user to overwrite K_2 factors on a member-by-member basis. The same K_2 factor is supposed to be used in calculation of the B_2 factor. However the program does not calculate B_2 factors and relies on the overwritten values. If the frame is not really a sway frame, the user should overwrite the K_2 factors.

Both K_1 and K_2 have two values: one for major direction and the other for minor direction, K_{1minor}, K_{1major}, K_{2minor}, K_{2major}.

There is another K-factor. K_{ltb} for lateral torsional buckling. By default, K_{ltb} is taken as equal to K_{2minor}. However the user can overwrite this on a member-by-member basis.

The rest of this section is dedicated to the determination of K_2 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 frame for which the K-factor calculation is relatively complex. For the purpose of calculating K-factors, the objects are identified as columns, beam and braces. All frame objects parallel to the Z-axis are classified as columns. All objects parallel to the X-Y plane are classified as beams. The remainders are considered to be braces.

The beams and braces are assigned K-factors of unity. In the calculation of the K-factors for a column object, 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:

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

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

If a rotational release exists at a particular end (and direction) of an object, the corresponding value of G 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-factors 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 $K = \pi/\alpha$. 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 object that has a pin at the joint under consideration will not enter the stiffness summations calculated above. An object that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated *EI* value. Also, beam members 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 member, 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 object 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.

When a steel frame design is performed in accordance with ANSI/AISC 360-10 provision and the analysis method is chosen to be any of the four direct analysis methods, the K_2 factors are automatically taken as 1 (AISC C.3). The calculated K_2 factors and their overwritten values are not considered in design.

2.11 Supported Framing Types

The code (ANSI/AISC 341-10) recognizes the following types of framing systems.

Framing Type	**References**
OMF (Ordinary Moment Frame)	AISC SEISMIC E1
IMF (Intermediate Moment Frame)	AISC SEISMIC E2
SMF (Special Moment Frame)	AISC SEISMIC E3
STMF (Special Truss Moment Frame)	AISC SEISMIC E4
OCBF (Ordinary Concentrically Braced Frame)	AISC SEISMIC F1
SCBF (Special Concentrically Braced Frame)	AISC SEISMIC F2
EBF (Eccentrically Braced Frame)	AISC SEISMIC F3
BRBF (Buckling Restrained Braced Frame)	AISC SEISMIC F4
SPSW (Special Plate Shear Wall)	AISC SEISMIC F5

With regard to these framing types, the program has implemented specifications for all types of framing systems, except STMF, BRBF, and SPSW. Implementing those three types of framing require further information about modeling.

The program recognizes the OCBF framing in its two separate incarnations: OCBF for regular Ordinary Concentrically Braced Frames (AISC SEISMIC F1.1) and OCBFI for (base) Isolated Ordinary Concentrically Braced Frames (AISC SEISMIC F1.7).

See Chapter 4 Special Seismic Provisions (ANSI/AISC 341-10) for additional requirements.

2.12 Continuity Plates

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

- The steel beam frames in a direction parallel to the column major direction, i.e., the beam frames into the column flange.
- The steel beam frames in a direction parallel to the column minor direction, i.e., the beam frames into the column web.
- The steel beam frames in a direction that is at an angle to both of the principal axes.

To achieve a beam/column moment connection, continuity plates, such as shown in Figure 2-3, 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 connection conditions described in the last two bullet items, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam.

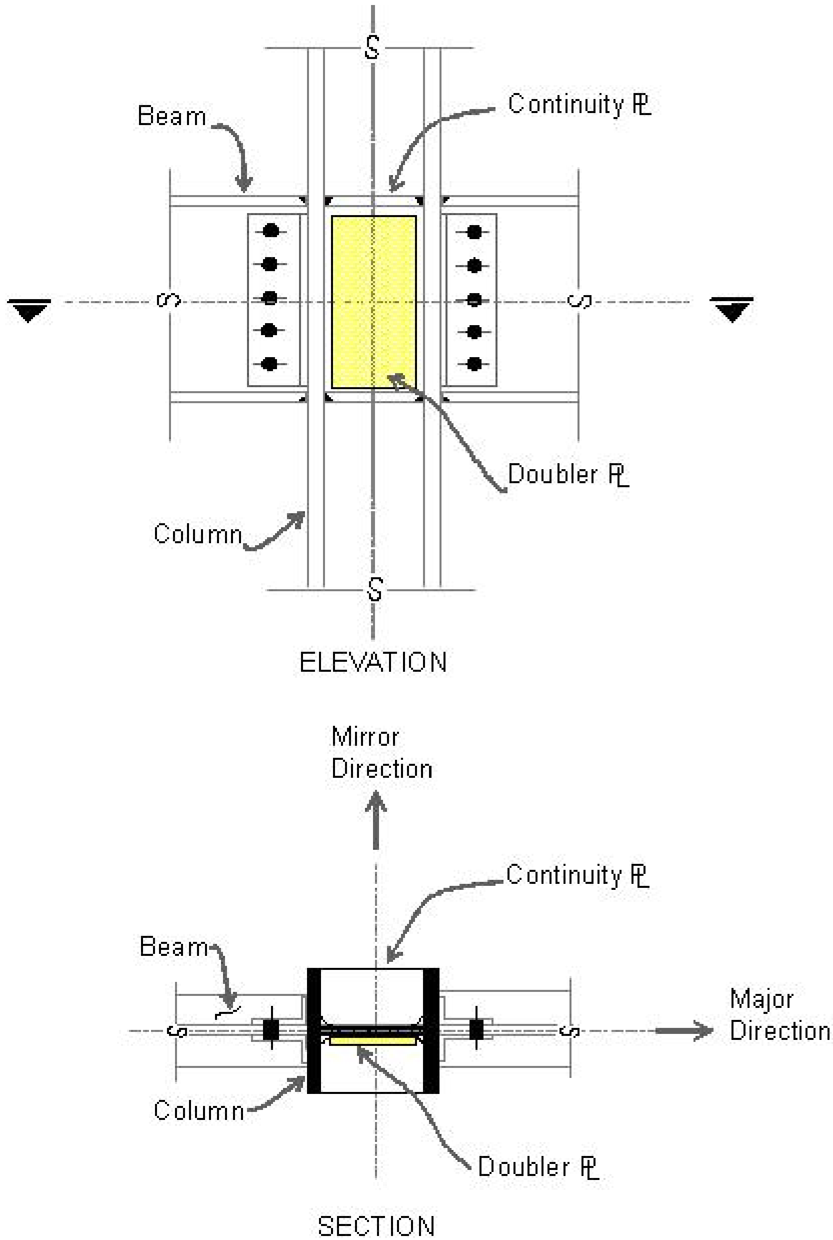


Figure 2-3 Doubler Plates and Continuity Plates

However, for the connection condition described by the first bullet item, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends upon the magnitude of the beam flange force and the properties of the column.

The program investigates whether the continuity plates are needed based on the requirements of the selected code. Columns of I-sections supporting beams of I-sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only.

2.13 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 web of the column resists the joint shear forces. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear stresses are seldom critical, and the program does therefore not investigate this condition.

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending upon 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 3-3. When code appropriate, the program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes and only supporting beams with I-shapes are investigated for doubler plate requirements. Also, doubler plate requirements are evaluated for moment frames only.

2.14 Choice of Units

English as well as SI and MKS metric units can be used for input. The codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. However, any system of units can be used to define and design a structure in the program.

The Display Unit preferences allow the user to specify the units.

Chapter 3
Design Using ANSI/AISC 360-10

This chapter provides a detailed description of the algorithms used by the programs in the design/check of structures in accordance with "ANSI/AISC 360-10 — Specifications for Structural Steel Building" (AISC 2010a, b). The menu option also covers the "ANSI/AISC 341-10 — Seismic Provisions for Structural Steel Building" (AISC 2010c), which is described in the next chapter. The implementation covers load combinations from "ASCE/SEI 7-10," which is described in the section "Design Loading Combinations" in this chapter. The loading based on "ASCE/SEI 7-10" has been described in a separate document entitled "CSI Lateral Load Manual" (CSI 2012). References also are made to IBC 2012 in this document.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- Reference to the ANSI/AISC 360-10 code is identified with the prefix "**AISC**."
- Reference to the ANSI/AISC 341-10 code is identified with the prefix "**AISC SEISMIC**" or sometimes "**SEISMIC**" only.
- Reference to the ASCE/SEI 7-10 code is identified with the prefix "**ASCE**."
- Reference to the IBC 2012 code is identified with the prefix "**IBC**."

3.1 Notations

The various notations used in this chapter are described herein.

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_1	Effective length factor for braced condition
K_2	Effective length factor for unbraced condition
K_{33},K_{22}	Effective length K-factors in the major and minor directions for appropriate braced (K_1) and unbraced (K_2) condition
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, and the like
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 $k_c = 4\big/\sqrt{h/t_w}$, $0.35 \le k_c \le 0.763$
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
λ_{slender}	Limiting slenderness parameter for slender element
φ_b	Resistance factor for bending
φ_c	Resistance factor for compression
φ_t	Resistance factor for tension yielding
φ_T	Resistance factor for torsion
φ_v	Resistance factor for shear
Ω_b	Safety factor for bending
Ω_c	Safety factor for compression
Ω_t	Safety factor for tension
Ω_T	Safety factor for torsion
Ω_v	Safety factor for shear

3.2 Design Loading Combinations

The structure is to be designed so that its design strength equals or exceeds the effects of factored loads stipulated by the applicable design code. The default design combinations are the various combinations of the already defined load cases, such as dead load (DL), live load (LL), roof live load (RL), snow load (SL), wind load (WL), and horizontal earthquake load (EL).

AISC 360-10 refers to the applicable building code for the loads and load combinations to be considered in the design, and to ASCE 7-10 in the absence of such a building code. Hence, the default design combinations used in the current version are the ones stipulated in ASCE 7-10:

For design in accordance with LRFD provisions:

1.4 DL	(ASCE 2.3.2-1)
1.2 DL + 1.6 LL + 0.5RL	(ASCE 2.3.2-2)
1.2 DL + 1.0 LL + 1.6RL	(ASCE 2.3.2-3)
1.2 DL + 1.6 LL + 0.5 SL	(ASCE 2.3.2-2)
1.2 DL + 1.0 LL + 1.6 SL	(ASCE 2.3.2-3)
0.9 DL ± 1.0WL	(ASCE 2.3.2-6)
1.2 DL + 1.6 RL± 0.5WL	(ASCE 2.3.2-3)
1.2 DL + 1.0LL+ 0.5RL± 1.0WL	(ASCE 2.3.2-4)
1.2 DL + 1.6 SL± 0.5 WL	(ASCE 2.3.2-3)
1.2 DL + 1.0LL+ 0.5SL± 1.0 WL	(ASCE 2.3.2-4)
0.9 DL ± 1.0 EL	(ASCE 2.3.2-7)
1.2 DL + 1.0 LL+ 0.2SL± 1.0EL	(ASCE 2.3.2-5)

For design in accordance with ASD provisions:

1.0 DL	(ASCE 2.4.1-1)
1.0 DL + 1.0 LL	(ASCE 2.4.1-2)
1.0 DL + 1.0 RL	(ASCE 2.4.1-3)
1.0 DL + 0.75 LL + 0.75 RL	(ASCE 2.3.2-4)
1.0 DL + 1.0 SL	(ASCE 2.4.1-3)
1.0 DL + 0.75 LL + 0.75 SL	(ASCE 2.3.2-4)

1.0 DL ± 0.6 WL (ASCE 2.4.1-5)
1.0 DL + 0.75 LL+ 0.75 RL± 0.75 (0.6WL) (ASCE 2.4.1-6a)
1.0 DL + 0.75 LL+ 0.75 SL± 0.75 (0.6WL) (ASCE 2.4.1-6a)
0.6 DL ± 0.6 WL (ASCE 2.4.1-7)

1.0 DL ± 0.7 EL (ASCE 2.4.1-5)
1.0 DL + 0.75 LL+ 0.75 SL± 0.75(0.7 EL) (ASCE 2.4.1-6b)
0.6 DL ± 0.7 EL (ASCE 2.4.1-8)

Most of the analysis methods recognized by the code are required to consider Notional Load in the design loading combinations for steel frame design. The program allows the user to define and create notional loads as individual load cases from a specified percentage of a given gravity load acting in a particular lateral direction. These notional load patterns should be considered in the combinations with appropriate factors, appropriate directions, and appropriate senses. Currently, the program automatically includes the notional loads in the default design load combinations for gravity combinations only. The user is free to modify the default design preferences to include the notional loads for combinations involving lateral loads. For further information, refer to the "Notional Load Patterns" section in Chapter 2.

The program automatically considers seismic load effects, including overstrength factors (ASCE 12.4.3), as special load combinations that are created automatically from each load combination, involving seismic loads. In that case, the horizontal component of the force is represented by E_{hm} and the vertical component of the force is represented by E_v, where

$$E_{hm} = \Omega_0 Q_E \quad \text{(ASCE 12.4.3.1)}$$

$$E_v = 0.2 S_{DS} D \quad \text{(ASCE 12.4.2.2)}$$

where, Ω_o is the overstrength factor and it is taken from ASCE 7-10 Table 12.2-1. The factor S_{DS} is described later in this section. Effectively, the special seismic combinations that are considered for the LRFD provision are

$$(1.2 + 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E \quad \text{(ASCE 2.3.2-5, 12.4.3.2)}$$

$$(1.2 + 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E + 1.0\text{LL} \quad \text{(ASCE 2.3.2-5, 12.4.3.2)}$$

$$(0.9 - 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E \quad \text{(ASCE 2.3.2-7, 12.4.3.2)}$$

and for the ASD provision the combinations are

$(1.0 + 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E$ (ASCE 2.4.1-5, 12.4.3.2)

$(1.0 + 0.105S_{DS})\text{DL} \pm 0.75(0.7\Omega_0)Q_E + 0.75\text{LL}$ (ASCE 2.4.1-6b,12.4.3.2)

$(0.6 - 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E$ (ASCE 2.4.1-8,12.4.3.2)

The program assumes that the defined earthquake load is really the strength level earthquake, which is equivalent to Q_E as defined in Section 12.4.2.1 of the ASCE 7-10 code. For regular earthquake, load is considered to have two components: horizontal, E_h and vertical E_v, which are taken as

$E_h = \rho Q_E$ (ASCE 12.4.2.1)

$E_v = 0.2S_{DS}D$ (ASCE 12.4.2.2)

where, ρ is the redundancy factor as defined in Section 12.3.4 of ASCE 7-10, and the S_{DS} is the design earthquake spectral response acceleration parameters at short periods, as defined in Section 11.4.4 of ASCE 7-10 code.

Effectively, the seismic load combination for the LRFD provision becomes:

$(1.2 + 0.2S_{DS})\text{DL} \pm \rho Q_E$ (ASCE 2.3.2-5, 12.4.2.3)

$(1.2 + 0.2S_{DS})\text{DL} \pm \rho Q_E + 1.0\text{LL}$ (ASCE 2.3.2-5, 12.4.2.3)

$(0.9 - 0.2S_{DS})\text{DL} \pm \rho Q_E$ (ASCE 2.3.2-7, 12.4.2.3)

The seismic load combinations for the ASD provision become:

$(1.0 + 0.14S_{DS})\text{DL} \pm 0.7\rho Q_E$ (ASCE 2.4.1-5, 12.4.2.3)

$(1.0 + 0.105S_{DS})\text{DL} \pm 0.75(0.7\rho)Q_E + 0.75\text{LL}$ (ASCE 2.4.1-6b, 12.4.2.3)

$(0.6 - 0.14S_{DS})\text{DL} \pm 0.7\rho Q_E$ (ASCE 2.4.1-8, 12.4.2.3)

The program assumes that the seismic loads defined as the strength level load is the program load case. Otherwise, the factors ρ, Ω_o, and S_{DS} will not be able to scale the load to the desired level.

The combinations described herein are the default loading combinations only. They can be deleted or edited as required by the design code or engineer-of-record.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

3.3 Classification of Sections for Local Buckling

The nominal strengths for flexure are dependent on the classification of the section as Seismically Compact, Compact, Noncompact, Slender, or Too Slender. Compact or Seismically Compact sections are capable of developing the full plastic strength before local buckling occurs. Non-compact sections can develop partial yielding in compression, and buckle inelastically before reaching to a fully plastic stress distribution. Slender sections buckle elastically before any of the elements yield under compression. Seismically Compact sections are capable of developing the full plastic strength before local buckling occurs when the section goes through low cycle fatigue and withstands reversal of load under seismic conditions.

Sections are classified as Compact, Noncompact, or Slender sections in accordance with Section B4 of the code (AISC B4). For a section to qualify as Compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table B4.1b of the code. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1, the section is Noncompact. If the width-thickness ratio of any element exceeds λ_r but does not exceed λ_s, the section is Slender. If the width-thickness ratio of any element exceedλ_s, the section is considered Too Slender. The expressions of λ_p, λ_r, andλ_s, as implemented in the program, are reported in Table 3-1 (AISC Table B4.1b, B4, F8, F13.2). In that table all expressions of λ_p and λ_r are taken from AISC section B4 and AISC Table B4.1. The limit demarcating Slender and Too Slender has been identified as λ_s in this document. The expressions of λ_s for I-Shape, Double Channel, Channel and T-Shape sections are taken from AISC section F13.2. The expression of λ_s for Pipe Sections is taken from AISC section F8. The expression of λ_p for Angle and Double Angle sections is taken from AISC Seismic code ANSI/AISC 341-10 Table D1.1.

For compression, sections are classified as nonslender element or slender element sections as reported in Table 3-2 (AISC B4.1, Table B4.1a). For a nonslender element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table 3-2. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender element section.

The table uses the variables k_c, F_L, h, h_p, h_c, b_f, t_f, t_w, b, t, D, d, and so on. The variables b, d, D and t are explained in the respective figures inside the table. The variables b_f, t_f, h, h_p, h_c, and t_w are explained in Figure 3-1. For Doubly Symmetric I-Shapes, h, h_p, and h_c are all equal to each other.

For unstiffened elements supported along only one edge parallel to the direction of compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width b is one-half the full-flange width, b_f.

(b) For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.

(c) For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, d is taken as the full nominal depth of the section.

Refer to Table 3-1 (AISC Table B4.1) for the graphic representation of unstiffened element dimensions.

For stiffness elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

(a) For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

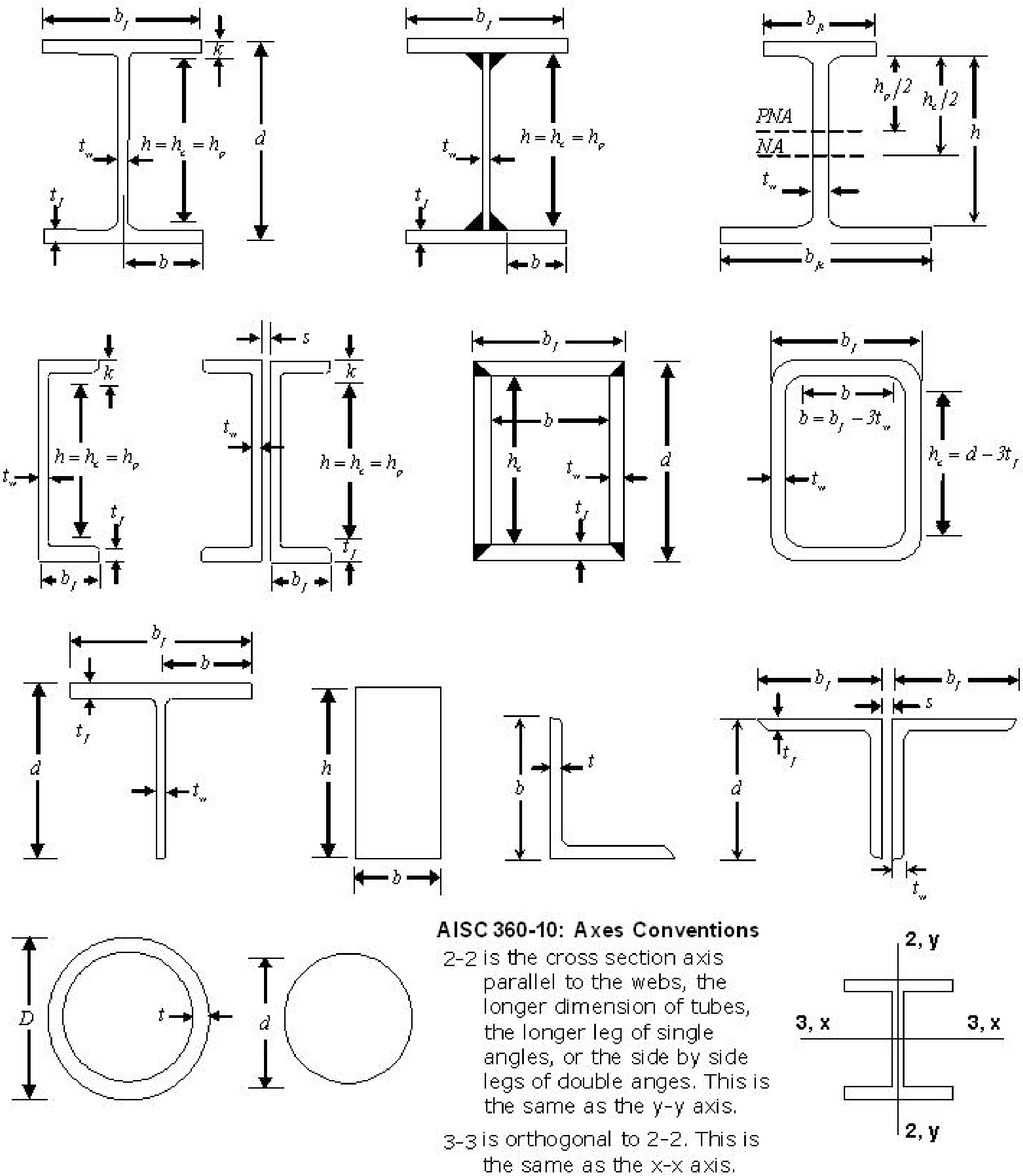


Figure 3-1 AISC-360-10 Definition of Geometric Properties

(b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centorid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.

Table 3-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections – Members Subjected to Flexure With or Without Axial Force

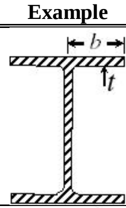
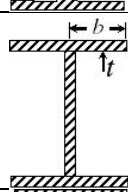
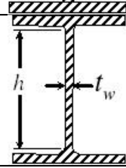
Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element		
					Compact (λ_p)	NonCompact (λ_r)	Slender (λ_s)
Doubly Symmetric I-Shape	Flexural compression of flanges of rolled I-Shapes		10	$b_f/2t_f$	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	No Limit
	Flexural compression in flanges of built-up I-Shapes		11	$b_f/2t_f$	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$	No Limit
	Flexure in web		15	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	$\min\{0.40\,E/F_y, 260\}$ (beams) No limit for columns and braces

Table 3-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections – Members Subjected to Flexure With or Without Axial Force

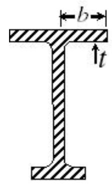
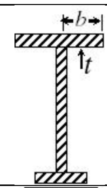
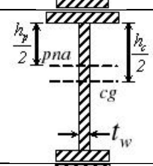
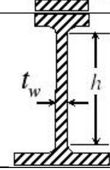
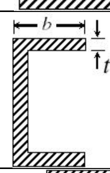
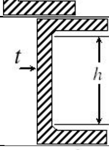
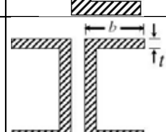
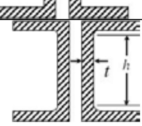
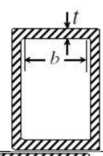
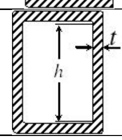
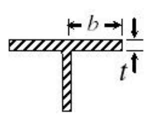
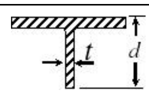
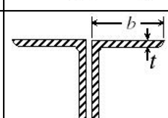
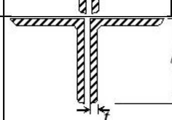
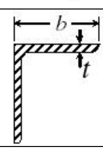
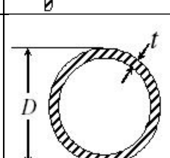
Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element: Compact (λ_p)	NonCompact (λ_r)	Slender (λ_s)
Singly Symmetric I-Shapes	Flexural Compression of flanges of rolled I-Shapes		10	$b_f/2t_f$	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	No Limit
	Flexural Compression in flanges of built-up I-Shapes		11	$b_f/2t_f$	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$	No Limit
	Flexure in Web		16	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)} \le \lambda_r$	$5.70\sqrt{E/F_y}$	No Limit
	Flexure in Web			h/t_w	NA	NA	$\min\{0.40\,E/F_y, 260\}$ (beams) No limit for columns and braces
Channel	Flexural compression in flanges		10	b_f/t_f	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	No Limit
	Flexure in web		15	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	$\min\{0.40\,E/F_y, 260\}$ (beams) No limit for columns and braces
Double Channel	Flexural compression in flanges		10	b_f/t_f	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	No Limit
	Flexure in web		15	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	$\min\{0.40\,E/F_y, 260\}$ (beams) No limit for columns and braces

Table 3-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections – Members Subjected to Flexure With or Without Axial Force

Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element: Compact (λ_p)	NonCompact (λ_r)	Slender (λ_s)
Box	Flexural or axial compression of flanges under major axis bending		17	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	No Limit
Box	Flexure in web		19	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	No Limit
T-Shape	Flexural or axial compression in flanges		10	$b_f/2t_f$	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	No Limit
T-Shape	Compression in stems		14	d/t_w	$0.84\sqrt{E/F_y}$	$1.03\sqrt{E/F_y}$	No Limit
Double Angle	Any type of compression in leg		12	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	No Limit
Double Angle	Any type of compression in leg		12	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	No Limit
Angle	Flexural compression in any leg		12	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	No Limit
Pipe	Flexural compression		20	D/t	$0.07\sqrt{E/F_y}$	$0.31E/F_y$	$0.45E/F_y$
Round Bar	———	———	——	———	Assumed Noncompact		
Rectangular	———	———	——	———	Assumed Noncompact		
General	———	———	——	———	Assumed Noncompact		
SD Section	———	———	——	———	Assumed Noncompact		

(d) For flanges of rectangular hollow structural sections (HSS), the width *b* is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, *h* is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, *b* and *h* shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, *t*, shall be taken as the design wall thickness, in accordance with AISC Section B3.12.

Refer to Table 3-1 (AISC Table B4.1) for the graphic representation of stiffened element dimensions.

Table 3-2 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections Subjected to Axial Compression

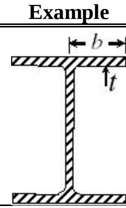
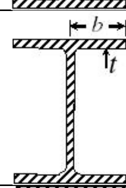
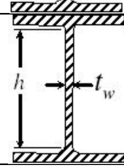
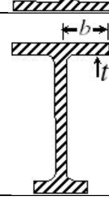
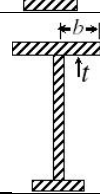
Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element NonCompact (λ_r)
Doubly Symmetric I-Shape	Axial only compression in flanges of rolled I-Shapes		1	$b_f/2t_f$	$0.56\sqrt{E/F_y}$
	Axial only compression in flanges of built-up I-Shapes		2	$b_f/2t_f$	$0.64\sqrt{k_cE/F_L}$
	Web in axial only compression		5	h/t_w	$1.49\sqrt{E/F_y}$
Singly Symmetric I-Shapes	Axial only compression in flanges of rolled I-Shapes		1	$b_f/2t_f$	$0.56\sqrt{E/F_y}$
	Axial only compression in flanges of built-up I-Shapes		2	$b_f/2t_f$	$0.64\sqrt{k_cE/F_L}$

Table 3-2 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections Subjected to Axial Compression

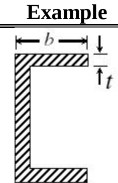
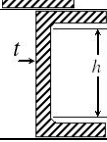
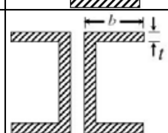
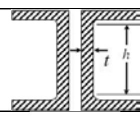
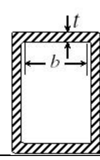
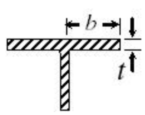
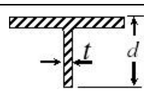
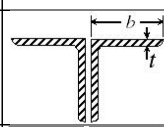
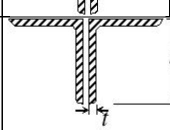
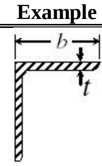
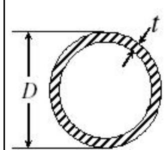
Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element NonCompact (λ_r)
Channel	Axial only compression in flanges		1	b_f/t_f	$0.56\sqrt{E/F_y}$
	Web in axial only compression		5	h/t_w	$1.49\sqrt{E/F_y}$
Double Channel	Axial only compression in flanges		1	b_f/t_f	$0.56\sqrt{E/F_y}$
	Web in axial only compression		5	h/t_w	$1.49\sqrt{E/F_y}$
Box	Axial compression		6	b/t	$1.40\sqrt{E/F_y}$
T-Shape	Axial compression in flanges		2	$b_f/2t_f$	$0.56\sqrt{E/F_y}$
	Compression in stems		4	d/t_w	$0.75\sqrt{E/F_y}$
Double Angle	Any type of compression in leg		3	b/t	$0.45\sqrt{E/F_y}$
	Any type of compression in leg		3	b/t	$0.45\sqrt{E/F_y}$

Table 3-2 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections Subjected to Axial Compression

Section Type	Description of Element	Example	AISC Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element NonCompact (λ_r)
Angle	Axial only compression in any leg		3	b/t	$0.45\sqrt{E/F_y}$
Pipe	Axial only compression		9	D/t	$0.11E/F_y$
Round Bar	———	———	——	———	Assumed Noncompact
Rectangular	———	———	——	———	Assumed Noncompact
General	———	———	——	———	Assumed Noncompact
SD Section	———	———	——	———	Assumed Noncompact

The design wall thickness, t, for hollow structural sections, such as Box and Pipe sections, is modified for the welding process (AISC B4.2). If the welding process is ERW (Electric-Resistance Welding), the thickness is reduced by a factor of 0.93. However, if the welding process is SAW (Submerged Arc Welded), the thickness is not reduced. The Overwrites can be used to choose if the thickness of HSS sections should be reduced for ERW on a member-by-member basis. The Overwrites can also be used to change the reduction factor.

The variable k_c can be expressed as follows:

$$k_c = \frac{4}{\sqrt{h/t_w}}, \qquad \text{(AISC Table B4.1b Note a)}$$

$$0.35 \le k_c \le 0.76. \qquad \text{(AISC Table B4.1b Note a)}$$

For Doubly Symmetric I-Shapes, Channels, and Double Channels, F_L can be expressed as follows:

$$F_L = 0.7F_y, \qquad \text{(AISC Table B4.1b Note b)}$$

and for Singly Symmetric I-Shape sections, F_L can be expressed as follows:

$F_L = \frac{S_{xt}}{S_{xc}} F_y$, where (AISC Table B4.1b Note b, F4-6)

$0.5F_y \leq F_L \leq 0.7F_y$. (AISC Table B4.1b Note b, F4-6)

Seismically Compact sections are compact sections that satisfy a more stringent width-thickness ratio limit, λ_{md} and λ_{hd}. These limits are presented in Table 4-1 in Chapter 4, which is dedicated to the seismic code.

In classifying web slenderness of I-Shapes, Box, Channel, Double Channel, and all other sections, it is assumed that there are no intermediate stiffeners. Double angles and channels are conservatively assumed to be separated.

Stress check of Too Slender sections is beyond the scope of this program.

3.4 Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are P_r, M_{r33}, M_{r22}, V_{r2}, V_{r3} and T_r corresponding to factored values of the axial load, the major and minor moments and shears, and torsion, respectively. These factored loads are calculated at each of the previously defined stations.

The factored forces can be amplified to consider second order effects, depending on the choice of analysis method chosen in the Preferences. If the analysis method is chosen to be General Second Order Elastic Analysis or any of the Direct Analysis methods with General Second Order Analysis, it is assumed that the analysis considers the influence of second-order effects (P-Δ and P-δ effects); hence the analysis results are used without amplification (AISC C1). Second-order effects due to overall sway of the structure can usually be accounted for, conservatively, by considering the second-order effects on the structure under one set of loads (usually the most severe gravity load case), and performing all other analyses as linear using the stiffness matrix developed for this one set of P-delta loads (see also White and Hajjar 1991). For a more accurate analysis, it is always possible to define each loading combination as a nonlinear load case that considers only geometric nonlinearities. For both approaches, when P-δ effects are expected to be important, use more than one element per line object (accomplished using the automatic frame subdivide option; refer to the program Help for more information about automatic frame subdivide).

If the analysis method is chosen to be Second Order Analysis by Amplified First Order Analysis or any of the Direct Analysis Methods with Amplified First Order Analysis (AISC C2.1(2), App.8.2), it is assumed that the analysis does not consider the influence of second order effects (P-Δ and P-δ). Hence the analysis results are amplified using B_1 and B_2 factors using the following approximate second-order analysis for calculating the required flexural and axial strengths in members of lateral load resisting systems. The required second-order flexural strength, M_r, and axial strength, P_r are determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(AISC A-8-1)}$$

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(AISC A-8-1)}$$

where,

$$B_1 = \frac{C_m}{1 - \alpha \dfrac{P_r}{P_{e1}}} \geq 1, \text{ and} \qquad \text{(AISC A-8-3)}$$

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{\text{story}}}{P_{e,\text{story}}}} \geq 1, \qquad \text{(AISC A-8-6)}$$

where,

$$\alpha = \begin{cases} 1.0 \text{ for LRFD,} \\ 1.6 \text{ for ASD,} \end{cases}$$

M_r = required second-order flexural strength using LFRD and ASD load combinations, kip-in (N-mm)

M_{nt} = first-order moment using LFRD and ASD load combinations, assuming there is no lateral translation of the frame, kip-in. (N-mm)

M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in (N-mm)

P_r = required second-order axial strength using LRFD or ASD load combinations, kip (N)

P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, kips (N)

P_{story} = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, kips (N)

P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, kips (N)

C_m = a coefficient assuming no lateral translation of the frame, whose value is taken as follows:

(i) For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4\left(M_a / M_b\right), \qquad \text{(AISC A-8-4)}$$

where, M_a and M_b, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_a / M_b is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(ii) For beam-columns subjected to transverse loading between supports, the value of C_m is conservatively taken as 1.0 for all cases.

When M_b is zero, C_m is taken as 1.0, the program defaults C_m to 1.0, if the unbraced length is more than actual member length. The user can overwrite the value of C_m for any member. C_m can be expressed as follows:

$$C_m = \begin{cases} 1.00, & \text{if length is more than actual length,} \\ 1.00, & \text{if tension member,} \\ 1.00, & \text{if both ends unrestrained,} \\ 0.6 - 0.4\dfrac{M_a}{M_b}, & \text{if no transverse loading, and} \\ 1.00, & \text{if transverse loading is present.} \end{cases}$$

(AISC A-8-4, App 8.2.1)

P_{el} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, kips (N)

$$P_{e1} = \frac{\pi^2 EI}{\left(K_1 L\right)^2} \quad \text{(AISC A-8-5)}$$

If any of the direct analysis methods are used, the reduced value of *EI* is used (AISC C3.3).

$P_{e,\text{story}}$ = elastic critical buckling resistance for the story determined by sidesway buckling analysis, kips (N)

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is the elastic story sidesway buckling resistance and calculated as

$$P_{e,\text{ story}} = R_M \frac{HL}{\Delta_H}, \quad \text{(AISC A-8-7)}$$

where,

E = modulus of elasticity of steel = 29,000 ksi (200,000 MPa)

If any of the direct analysis methods are used, the reduced value of *EI* is used (AISC App. 8.2.1).

I = moment of inertia in the plane of bending, in.^{4} (mm^4)

L = story height, in. (mm)

K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation. It is taken to be equal to 1.0, conservatively. The Overwrites can be used to change the value of K_1 for the major and minor directions.

K_2 = effective length factor in the plane of bending, calculated based on a sidesway buckling analysis. The Overwrites can be used to change the value of K_2 for the major and minor directions.

In the expression of B_1, the required axial force P_r is used based on its first order value. The magnification factor B_1 must be a positive number. Therefore, αP_r must be less than P_{e1}. If αP_r is found to be greater than or equal to P_{e1} a failure condition is declared.

If the program assumptions are not satisfactory for a particular structural model or member, the user has the choice to explicitly specify the values of B_1 for any member.

Currently, the program does not calculate the B_2 factor. The user is required to overwrite the values of B_2 for the members.

3.5 Calculation of Nominal Strengths

The nominal strengths in compression, tension, bending, and shear are computed for Compact, Noncompact, and Slender members in accordance with the following sections. The nominal flexural strengths for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I-Shape, Box, Channel, Double Channel, Circular, Pipe, T-Shape, 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 except shear are based on that.

For *all* sections, the nominal shear strengths are calculated for directions aligned with the geometric axes, which typically coincide with the principal axes. Again, the exception is the Single Angle section.

If the user specifies nonzero nominal capacities for one or more of the members on the Steel Frame Overwrites form, those values will override the calculated values for those members. The specified capacities should be based on the principal axes of bending for flexure, and the geometric axes for shear.

3.5.1 Nominal Tensile Strength

This section applies to the members subject to axial tension.

Although there is no maximum slenderness limit for members designed to resist tension forces, the slenderness ratio preferably should not exceed 300 (AISC D1). A warning message to that effect is printed for such slender elements under tension.

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n / Ω_t, of tension members is taken as the lower value obtained according to the limit states of yielding of gross section under tension and tensile rupture in the net section.

3.5.1.1 Tensile Yielding in the Gross Section

$P_n = F_y A_g$ (AISC D2-1)

$\phi_1 = 0.90$ (LRFD) (AISC D2)

$\Omega_t = 1.67$ (ASD) (AISC D2)

3.5.1.2 Tensile Rupture in the Net Section

$P_n = F_u A_e$ (AISC D2-2)

$\phi_1 = 0.75$ (LRFD) (AISC D2)

$\Omega_t = 2.00$ (ASD) (AISC D2)

The effective net area, A_e, is assumed to be equal to the gross cross-sectional area, A_g, by default. For members that are connected with welds or members with holes, the A_e / A_g ratio must be modified using the steel frame design Overwrites to account for the effective area.

3.5.2 Nominal Compressive Strength

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n / Ω_c, of members subject to axial compression are addressed in this section. The resistance and safety factors used in calculation of design and allowable compressive strengths are:

ϕ_c= 0.90 (LRFD) (AISC E1)

Ω_c= 1.67 (ASD) (AISC E1)

In the determination, the effective length factor K_2 is used as the K-factor. If the chosen analysis method in the Preferences is the General Second Order Elastic Analysis, the First Order Analysis using Amplified First Order Analysis, or the Limited First Order Analysis, the calculated K_2 factors are used. If the user overwrites the K_2 factors, the overwritten values are used. If the chosen analysis method is one of any Direct Analysis Methods, the effective length factor, K, for calculation of P_n is taken as one (AISC C3). The overwritten value of K_2 will have no effect for the latter case.

The nominal axial compressive strength, P_n, depends on the slenderness ratio, Kl/r, where

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

For all sections except Single Angles, the principal radii of gyration r_{22} and r_{33} are used. For Single Angles, the minimum (principal) radius of gyration, r_z, is used instead of r_{22} and r_{33}, conservatively, in computing Kl/r. K_{33} and K_{22} are two values of K_2 for the major and minor axes of bending.

Although there is no maximum slenderness limit for members designed to resist compression forces, the slenderness ratio preferably should not exceed 200 (AISC E2). A warning message to that effect is given for such slender elements under compression.

The members with any slender element and without any slender elements are handled separately.

The limit states of torsional and flexural-torsional buckling are ignored for closed sections (Box and Pipe sections), solid sections, general sections, and sections created using Section Designer.

3.5.2.1 Members without Slender Elements

The nominal compressive strength of members with compact and noncompact sections, P_n, is the minimum value obtained according to the limit states of flexural buckling, torsional and flexural-torsional buckling.

3.5.2.1.1 Flexural Buckling

For compression members with compact and noncompact sections, the nominal compressive strength, P_n, based on the limit state of flexural buckling, is given by

$$P_n = F_{cr} A_g. \qquad \text{(AISC E3-1)}$$

The flexural buckling stress, F_{cr}, is determined as follows:

$$F_{cr} = \begin{cases} \left(0.658^{\frac{F_y}{F_e}}\right) F_y, & \text{if } \dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}, \\ 0.877 F_e, & \text{if } \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}, \end{cases} \qquad \text{(AISC E3-2, E3-3)}$$

where F_e is the elastic critical buckling stress given by

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2}. \qquad \text{(AISC E3-4)}$$

3.5.2.1.2 Torsional and Flexural-Torsional Buckling

For compression members with compact and noncompact sections, the nominal compressive strength, P_n, based on the limit state of torsional and flexural-torsional buckling is given by

$$P_n = F_{cr} A_g \qquad \text{(AISC E4-1)}$$

where A_g is the gross area of the member. The flexural buckling stress, $F_{c\tau}$, is determined as follows.

3.5.2.1.2.1 Box, Pipe, Circular, Rectangular, General and Section Designer Sections

The limit states of torsional and flexural-torsional buckling are ignored for members with closed sections, such as Box and Pipe sections, solid sections (Circular and Rectangular), General sections and sections created using the Section Designer.

3.5.2.1.2.2 Double Angle and T-Shapes

$$F_{cr} = \left(\frac{F_{cr22} + F_{crz}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{cr22} F_{crz} H}{\left(F_{cr22} + F_{crz} \right)^2}} \right], \qquad \text{(AISC E4-2)}$$

where,

$$F_{cr22} = \begin{cases} \left(0.658^{\frac{F_y}{F_e}}\right) F_y, & \text{if } \dfrac{KL_{22}}{r_{22}} \le 4.71\sqrt{\dfrac{E}{F_y}}, \\ 0.877 F_e, & \text{if } \dfrac{KL_{22}}{r_{22}} > 4.71\sqrt{\dfrac{E}{F_y}}, \end{cases} \qquad \text{(AISC E3-2, E3-3)}$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL_{22}}{r_{22}} \right)^2}, \quad \text{and} \qquad \text{(AISC E3-4)}$$

$$F_{crz} = \frac{GJ}{A_g \bar{r}_0^2}. \qquad \text{(AISC E4-3)}$$

3.5.2.1.2.3 I-Shape, Double Channel, Channel, Single Angle Sections

For I-Shape, Double Channel, Channel, and Single Angle sections, $F_{c\tau}$ is calculated using the torsional or flexural-torsional elastic buckling stress, F_e, as follows:

$$F_{cr} = \begin{cases} \left(0.658^{\frac{F_y}{F_e}} \right) F_y, & \text{if } \dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}, \\ 0.877 F_e, & \text{if } \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}. \end{cases} \qquad \text{(AISC E3-2, E3-3, E4b)}$$

where F_e is calculated from the following equations:

3.5.2.1.2.3.1 <u>I-Shapes and Double Channel Sections</u>

$$F_e = \left[\frac{\pi^2 E C_w}{\left(K_z L_z \right)^2} + GJ \right] \left(\frac{1}{I_{22} + I_{33}} \right) \qquad \text{(AISC E4-4)}$$

3.5.2.1.2.3.2 Channel Sections

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{\left(F_{e33} + F_{ez}\right)^2}}\right] \qquad \text{(AISC E4-5)}$$

3.5.2.1.2.3.3 Single Angle Sections with Equal Legs

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{\left(F_{e33} + F_{ez}\right)^2}}\right] \qquad \text{(AISC E4-5)}$$

3.5.2.1.2.3.4 Single Angle Sections with Unequal Legs

F_e is the lowest root of the cubic equation.

$$\left(F_e - F_{e33}\right)\left(F_e - F_{e22}\right)\left(F_e - F_{ez}\right) - F_e^2\left(F_e - F_{e22}\right)\left(\frac{x_o}{\bar{r}_0}\right)^2 - F_e^2\left(F_e - F_{e33}\right)\left(\frac{y_o}{\bar{r}_0}\right) = 0$$

(AISC E4-6)

In the preceding equations,

C_w is the warping constant, in^6 (mm^6)

x_0, $\mathbf{y}_0$ are the coordinates of the shear center with respect to the centroid, x_0 = 0 for Double Angle and T-Shaped members (y-axis symmetry)

$\bar{r}_0 = \sqrt{x_o^2 + y_o^2 + \dfrac{I_{22} + I_{33}}{A_g}}$ = polar radius of gyration about the shear center (AISC E4-11)

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}^2}\right) \qquad \text{(AISC E4-10)}$$

$$F_{e33} = \frac{\pi^2 E}{\left(K_{33}L_{33}/r_{33}\right)^2} \qquad \text{(AISC E4-7)}$$

$$F_{e22} = \frac{\pi^2 E}{\left(K_{22}L_{22}/r_{22}\right)^2} \qquad \text{(AISC E4-8)}$$

$$F_{ez} = \left[\frac{\pi^2 EC_w}{(K_z L_z)^2} + GJ\right]\frac{1}{A\bar{r}_0^2} \qquad \text{(AISC E 4-9)}$$

K_{22}, K_{33} are effective length factors K_2 in minor and major directions

K_z is the effective length factor for torsional buckling, and it is taken equal to K_{LTB} in this program; it can be overwritten

L_{22}, L_{33} are effective lengths in the minor and major directions

r_{22}, r_{33} are the radii of gyration about the principal axes

L_z is the effective length for torsional buckling and it is taken equal to L_{22} by default, but it can be overwritten.

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. The principal maximum value r_{max} is used for calculating F_{e33}, and the principal minimum value r_{min} is used in calculating F_{e22}.

3.5.2.2 Members with Slender Elements

The nominal compressive strength of members with slender sections, P_n, is the minimum value obtained according to the limit states of flexural, torsional and flexural-torsional buckling.

3.5.2.2.1 Flexural Buckling

For compression members with slender sections, the nominal compressive strength, P_n, based on the limit state of flexural buckling, is given by

$$P_n = F_{cr} A_g. \qquad \text{(AISC E7-1)}$$

The flexural buckling stress, F_{cr}, is determined as follows:

$$F_{cr} = \begin{cases} Q\left(0.658^{\frac{QF_y}{F_e}}\right)F_y, \text{ if } & \dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}, \text{ and} \\ 0.877F_e, & \text{if } \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}, \end{cases} \quad \text{(AISC E7-2, E7-3)}$$

where F_e is the elastic critical buckling stress for flexural buckling limit state.

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2}. \quad \text{(AISC E3-4)}$$

3.5.2.2.2 Torsional and Flexural-Torsional Buckling

For compression members with slender sections, the nominal compressive strength, P_n, based on Torsional and Flexural-Torsional limit state is given by:

$P_n = F_{cr}A_g$, where (AISC E7-1)

F_{cr} is determined as follows:

$$F_{cr} = \begin{cases} Q\left(0.658^{\frac{QF_y}{F_e}}\right)F_y \text{ if } & \dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}, \text{ and} \\ 0.877F_e & \text{if } \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}, \end{cases} \quad \text{(AISC E7-2, E7-3)}$$

where, F_e is the elastic critical buckling stress for torsional and flexural-torsional limit states, which are given for different shapes as follows.

3.5.2.2.2.1 Box, Pipe, Circular, Rectangular, General and Section Designer Sections

The limit states of torsional and flexural-torsional buckling are ignored for members with closed (Box and Pipe), solid (Circular and Rectangular), General sections and sections created using the Section Designer.

3.5.2.2.2.2 *I-Shape and Double Channel Sections*

$$F_e = \left[\frac{\pi^2 E C_w}{\left(K_2 L_2\right)^2} + GJ\right]\left(\frac{1}{I_{22} + I_{33}}\right) \qquad \text{(AISC E7, E4-4)}$$

3.5.2.2.2.3 *Channel Sections*

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{\left(F_{e33} + F_{ez}\right)^2}}\right] \qquad \text{(AISC E7, E4-5)}$$

3.5.2.2.2.4 Double Angle Sections *and T-Shapes*

$$F_e = \left(\frac{F_{e22} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e22}F_{ez}H}{\left(F_{e33} + F_{ez}\right)^2}}\right] \qquad \text{(AISC E7, E4-5)}$$

3.5.2.2.2.5 *Single Angle Sections with Equal Legs*

$$F_e = \left(\frac{F_{e33} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{\left(F_{e33} + F_{ez}\right)^2}}\right] \qquad \text{(AISC E7, E4-5)}$$

3.5.2.2.2.6 *Single Angle Sections with Unequal Legs*

F_e is the lowest root of the cubic equation.

$$\left(F_e - F_{e33}\right)\left(F_e - F_{e22}\right)\left(F_e - F_{ez}\right) - F_e^2\left(F_e - F_{e22}\right)\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2\left(F_e - F_{e33}\right)\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0$$

(AISC E7, E4-6)

The variables used in the preceding expressions for F_e, such as C_w, x_0, y_0, $\bar{r}_o$, H, F_{e33}, F_{e22}, F_{ez}, K_{22}, K_{33}, K_z, L_{22}, L_{33}, L_z, KL/r, and so on, were explained in the previous section.

3.5.2.2.3 Reduction Factor for Slenderness

The reduction factor for slender compression elements, Q, is computed as follows:

$$Q = Q_s Q_a, \qquad \text{(AISC E7)}$$

where Q_s and Q_a are reduction factors for slender unstiffened compression elements (flanges of I-Shapes, T-Shapes, Double Angles, Channels, and Double

Channels; legs of angles; and stems of T-Shapes) and slender stiffened compression elements (webs of I-Shapes, Channels, and Boxes; and Pipe sections), respectively. For cross-sections composed of only unstiffened slender elements, $Q = Q_s (Q_a = 1)$ and for cross-sections composed of only stiffened slender elements, $Q = Q_a (Q_s = 1)$.

The reduction factor, Q_s, for slender unstiffened elements is defined as follows:

3.5.2.2.3.1 Flange of I-Shape, T-Shape, Channel and Double Channel Sections

Rolled:

$$Q_s = \begin{cases} 1.0, & \text{if } \dfrac{b}{t} \le 0.56\sqrt{\dfrac{E}{F_y}}, \\ 1.415 - 0.74\left(\dfrac{b}{t}\right)\sqrt{\dfrac{F_y}{E}}, & \text{if } 0.56\sqrt{\dfrac{E}{F_y}} < \dfrac{b}{t} \le 1.03\sqrt{\dfrac{E}{F_y}}, \text{ and} \\ \dfrac{0.69E}{F_y\left(\dfrac{b}{t}\right)^2}, & \text{if } \dfrac{b}{t} > 1.03\sqrt{\dfrac{E}{F_y}}. \end{cases}$$

(AISC E7-4, E7-5, E7-6)

Built-Up:

$$Q_s = \begin{cases} 1.0, & \text{if } \dfrac{b}{t} \le 0.64\sqrt{\dfrac{Ek_c}{F_y}}, \\ 1.415 - 0.65\left(\dfrac{b}{t}\right)\sqrt{\dfrac{F_y}{Ek_c}}, & \text{if } 0.64\sqrt{\dfrac{Ek_c}{F_y}} < \dfrac{b}{t} \le 1.17\sqrt{\dfrac{Ek_c}{F_y}}, \\ \dfrac{0.90Ek_c}{F_y\left(\dfrac{b}{t}\right)^2}, & \text{if } \dfrac{b}{t} > 1.17\sqrt{\dfrac{Ek_c}{F_y}}, \end{cases}$$

(AISC E7-7, E7-8, E7-9)

where $k_c = \dfrac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.76$, (AISC E7.1b) and b/t is defined as

$$\frac{b}{t} = \begin{cases} \left(b_t/2t_f\right) & \text{for I Shapes,} \\ \left(b_f/2t_f\right) & \text{for T Shapes,} \\ b_f/t_f & \text{for Channels,} \\ b_f/t_f & \text{for Double Channels.} \end{cases} \qquad \text{(AISC B4.1a, E7.1)}$$

3.5.2.2.3.1.1 Legs of Single and Double Angle Sections

$$Q_s = \begin{cases} 1.0, & \text{if } \frac{b}{t} \le 0.45\sqrt{\frac{E}{F_y}}, \\ 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}, & \text{if } 0.45\sqrt{\frac{E}{F_y}} < \frac{b}{t} \le 0.91\sqrt{\frac{E}{F_y}}, \text{ and} \\ \frac{0.53E}{F_y\left(\frac{b}{t}\right)^2}, & \text{if } \frac{b}{t} > 0.91\sqrt{\frac{E}{F_y}}, \end{cases}$$

(AISC E7-10, E7-11, E7-12)

where b is the full width of the longest leg, and t is the corresponding thickness (AISC B4.1a, E7.1c).

3.5.2.2.3.1.2 Stem of T-Sections

$$Q_s = \begin{cases} 1.0, & \text{if } \frac{d}{t} \le 0.75\sqrt{\frac{E}{F_y}}, \\ 1.908 - 1.22\sqrt{\frac{F_y}{E}}, & \text{if } 0.75\sqrt{\frac{E}{F_y}} < \frac{d}{t} \le 1.03\sqrt{\frac{E}{F_y}}, \text{ and} \\ \frac{0.69E}{F_y\left(\frac{d}{t}\right)^2}, & \text{if } \frac{d}{t} > 1.03\sqrt{\frac{E}{F_y}}, \end{cases}$$

(AISC E7-13, E7-14, E7-15)

where d is the full nominal depth of the tee and t is the thickness of the element (AISC B4.1b).

For T-Shapes, the Q_s is calculated for the flange and web separately, and the minimum of the two values is used as Q_s. For Angle and Double Angle sections, Q_s is calculated based on the leg that gives the largest b/t and so the smallest Q_s.

The reduction factor, Q_a, for slender stiffened elements is defined as follows:

$$Q_a = \frac{A_{eff}}{A}, \qquad \text{(AISC E7-16)}$$

where A is the total cross sectional area of the member, and A_{eff} is the summation of the effective areas of the cross-section,

$$A_{eff} = A - \sum (b - b_e)t,$$

based on the reduced effective width, b_e, which is determined as follows.

3.5.2.2.3.1.3 Webs of I -Shapes, Channels, and Double Channels

$$b_e = \begin{cases} 1.92t\sqrt{\dfrac{E}{f}}\left[1 - \dfrac{0.34}{(b/t)}\sqrt{\dfrac{E}{F}}\right] \le b, & \text{if } \dfrac{b}{t} \ge 1.49\sqrt{\dfrac{E}{f}}, \text{ and} \\ b, & \text{if } \dfrac{b}{t} < 1.49\sqrt{\dfrac{E}{f}}, \end{cases} \qquad \text{(AISC E7-17)}$$

where f is taken as F_{cr} with Q = 1.0 (AISC 7.2a), and b is taken for rolled shapes as the clear distance between flanges less the corner radius, and is taken for welded shapes as the clear distance between flanges.

3.5.2.2.3.2 Webs and Flanges of Box Sections

$$b_e = \begin{cases} 1.92t\sqrt{\dfrac{E}{f}}\left[1 - \dfrac{0.38}{(b/t)}\sqrt{\dfrac{E}{F}}\right] \le b, & \text{if } \dfrac{b}{t} \ge 1.40\sqrt{\dfrac{E}{f}}, \text{ and} \\ b, & \text{if } \dfrac{b}{t} < 1.40\sqrt{\dfrac{E}{F_y}}, \end{cases} \qquad \text{(AISC E7-18)}$$

where f is conservatively taken as F_y (AISC 7.2b). The flange, b, is taken as $b_f–3t_w$, and for webs, b is taken as $h–3t_f$ (AISC B4.1b). The design wall thickness is modified for the welding process (AISC B4.2)

3.5.2.2.3.3 Pipe Sections

The reduction factor for slender stiffened elements is given directly by:

$$Q_a = Q = \begin{cases} 1.0, & \text{if } D/t < 0.11\dfrac{E}{F_y}, \\ \dfrac{0.038\text{E}}{F_y\left(D/t\right)} + \dfrac{2}{3}, & \text{if } 0.11\dfrac{E}{F_y} < \dfrac{D}{t} < 0.45\dfrac{E}{F_y}, \text{ and} \\ 1.0, & \text{if } D/t > 0.45\dfrac{E}{F_y}, \end{cases} \qquad \text{(AISC E7-19)}$$

where D is the outside diameter and t is the wall thickness. The design wall thickness is modified for the welding process (AISC B4.2). If D/t exceeds $0.45\,E/F_y$, the section is considered to be too slender and it is not designed.

3.5.3 Nominal Flexure Strength

This section applies to members subject to simple bending about one principal axis. The members are assumed to be loaded in a plane parallel to a principal axis that passes through the shear center, or restrained against twisting.

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n/Ω_b, are determined using the following resistance and safety factors:

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \text{(AISC F1(1))}$$

$$\Omega_b = 1.67 \text{ (ASD)} \qquad \text{(AISC F1(1))}$$

When determining the nominal flexural strength about the major principal axis for any sections for the limit state of lateral-torsional buckling, it is common to use the term C_b, the lateral-torsional buckling modification factor for non-uniform moment diagram. C_b is calculated as follows:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_c} \le 3.0, \qquad \text{(AISC F1-1, H1.2)}$$

where,

M_{max} = absolute value of maximum moment in unbraced segment, kip-in. (N-mm)

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

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}, is redefined to be more than the length of the member by the user or the program, i.e., if the unbraced length is longer than the member length. The Overwrites can be used to change the value of C_b for any member.

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 bending strength is the minimum value obtained according to the limit states of yielding, lateral-torsional buckling, flange local buckling, web local buckling, tension flange yielding as appropriate to different structural shapes. The following sections describe how different members are designed against flexure in accordance with AISC Chapter F. AISC, in certain cases, gives options in the applicability of its code section, ranging from F2 to F12. In most cases, the program follows the path of the sections that gives more accurate results at the expense of more detailed calculation. In some cases, the program follows a simpler path. For an easy reference, Table 3-3 shows the AISC sections for the various scenarios.

Table 3.3 Selection Table for the Application of Chapter F Sections

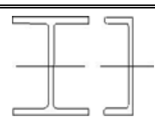
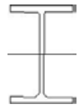
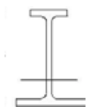
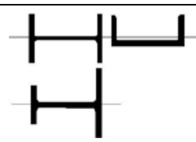
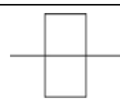
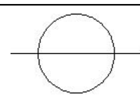
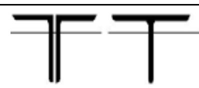
Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB

Table 3.3 Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F3		NC, S	C	LTB, FLB
F4		C, NC, S	NC	Y, LTB, FLB
F5		C, NC, S	S	Y, LTB, FLB
F4		C, NC, S	C, NC	Y, LTB, FLB, TFY
F5		C, NC, S	S	Y, LTB, FLB, TFY
F6		C, NC, S	Any	Y, FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	Any	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	Any	Y, LTB
F12	Unsymmetrical shapes	N/A	N/A	All limit states

Y = yielding
LTB = lateral-torsional buckling
FLB = flange local buckling
WLB = web local buckling
TFY = tension flange yielding
LLB = leg local buckling
LB = local buckling
C = compact or seismically compact
NC = noncompact
S = slender

3.5.3.1 Doubly Symmetric I-Sections

3.5.3.1.1 Major Axis Bending

The nominal flexural strength for major axis bending depends on compactness of the web and flanges.

3.5.3.1.1.1 Compact Webs with Compact Flanges

The nominal flexural strength is the lowest value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

3.5.3.1.1.1.1 Yielding

$$M_n = M_p = F_y Z_{33}\,, \qquad \text{(AISC F2-1)}$$

where, Z_{33} is the plastic section modulus about the major axis.

3.5.3.1.1.1.2 Lateral-Torsional Buckling

$$M_n = \begin{cases} M_p, & \text{if } L_b \le L_p, \\ C_b\left[M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p, & \text{if } L_p < L_b < L_r, \text{ and} \\ F_{cr} S_{33} \le M_p, & \text{if } L_p > L_r, \end{cases}$$

(AISC F2-1, F2-2, F2-3)

where, S_{33} is the elastic section modulus taken about the major axis, L_b is the unbraced length, L_p and L_r are limiting lengths, and F_{cr} is the critical buckling stress. F_{cr}, L_p, and L_r are given by:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_{33}h_0}\left(\frac{L_b}{r_{ts}}\right)^2}\,, \qquad \text{(AISC F2-4)}$$

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}\,, \qquad \text{(AISC F2-5)}$$

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_{33}h_0}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_{33}h_0}{Jc}\right)^2}}\,, \qquad \text{(AISC F2-6)}$$

where,

$$r_{ts}{}^{2} = \frac{\sqrt{I_y C_w}}{S_{33}}, \qquad \text{(AISC F2-7)}$$

$c = 1$, and (AISC F2-8a)

h_0 is the distance between flange centroids.

3.5.3.1.1.2 Compact Webs with Noncompact or Slender Flanges

The nominal flexural strength is the lowest value obtained from the limit states of lateral-torsional buckling and compression flange local buckling.

3.5.3.1.1.2.1 Lateral-Torsional Buckling

The provisions of lateral-torsional buckling for "Compact Web and Flanges" as described in the provision pages also apply to the nominal flexural strength of I-Shapes with compact webs and noncompact or slender flanges bent about their major axis.

$$M_n = \begin{cases} M_p, & \text{if } L_b \le L_p, \\ C_b\left[M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p, & \text{if } L_p < L_b < L_r, \text{ and} \\ F_{cr} S_{33} \le M_p, & \text{if } L_p > L_r. \end{cases}$$

(AISC F3.1, F2-1, F2-2, F2-3)

3.5.3.1.1.2.2 Compression Flange Local Buckling

$$M_n = \begin{cases} M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges,} \\ \dfrac{0.9 E k_c S_{33}}{\lambda^2}, & \text{for slender flanges,} \end{cases}$$

(AISC F3-1, F3-2)

where λ, λ_{pf}, and λ_{rf} are the slenderness and limiting slenderness for compact and noncompact flanges from Table 3.1, respectively,

$$\lambda = \frac{b_f}{2t_f},$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}, \qquad \text{(AISC Table B4.1b, F3.2)}$$

$$\lambda_{rf} = \begin{cases} 1.0\sqrt{\dfrac{E}{F_y}} & \text{(Rolled)}, \\ 0.95\sqrt{\dfrac{k_c E}{F_L}} & \text{(Welded)}, \end{cases} \qquad \text{(AISC Table B4.1b, F3.2)}$$

and k_c is given by

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \le k_c \le 0.76. \qquad \text{(AISC F.3.2)}$$

3.5.3.1.1.3 Noncompact Webs with Compact, Noncompact and Slender Flanges

The nominal flexural strength is the lowest values obtained from the limit states of compression flange yielding, lateral-torsional buckling, and compression flange local buckling.

3.5.3.1.1.3.1 Compression Flange Yielding

$M_n = R_{pc} M_y$, (AISC F4-1)

where, R_{pc} is the web plasticity factor, which is determined as follows:

$$R_{pc} = \begin{cases} 1 & \text{if } \quad I_{yc}/I_y \le 0.23, \\ \dfrac{M_p}{M_y}, & \text{if } \quad \lambda \le \lambda_{pw}, \text{and } I_{yc}/I_y > 0.23, \\ \left[\dfrac{M_p}{M_y} - \left(\dfrac{M_p}{M_y} - 1\right)\left(\dfrac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \dfrac{M_p}{M_y}, & \text{if } \quad \lambda_{pw} < \lambda_w \le \lambda_{rw}, \text{and } I_{yc}/I_y > 0.23, \end{cases}$$

(AISC F4-9a, F4-9b, F4-10)

where,

M_p = $Z_{33}F_y \leq 1.6S_{33}F_y$ (AISC F4-2)

S_{33} = elastic section modulus for major axis bending

$\lambda_w = \dfrac{h_c}{t_w}$ (AISC F4.2, Table B4.1)

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, as given in Table 3-1 (AISC Table B4.1, F4.2)

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, as given in Table 3-1 (AISC Table B4.1, F4.2)

and M_y is the yield moment, which is determined as follows:

$M_y = S_{33}F_y$ (AISC F4-1)

3.5.3.1.1.3.2 <u>Lateral-Torsional Buckling</u>

$$M_n = \begin{cases} R_{pc}M_y, & \text{if } L_b \leq L_p, \\ C_b\left[R_{pc}M_y - \left(R_{pc}M_y - F_L S_{33}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \leq R_{pc}M_y, & \text{if } L_p < L_b \leq L_r, \\ F_{cr}S_{33} \leq R_{pc}M_y, & \text{if } L_b > L_r, \end{cases}$$

(AISC F4-1, F4-2, F4-3)

where,

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{JC}{S_{33}h_o}\left(\frac{L_b}{r_t}\right)^2}$$ (AISC F4-5)

$$r_t = \frac{b_f}{\sqrt{12\left(\dfrac{h_0}{d} + \dfrac{1}{6}a_w\dfrac{h^2}{h_0 d}\right)}}$$ (AISC F4-11)

$$a_w = \frac{h_c t_w}{b_f t_f} \leq 10$$ (AISC F4-12)

$$C = \begin{cases} 1, & \text{if } I_{yc}/I_y > 0.23 \\ 0, & \text{if } I_{yc}/I_y \le 0.23 \end{cases} \quad \text{(AISC F4.2)}$$

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad \text{(AISC F4-7)}$$

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{33} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L}{E} \frac{S_{33} h_o}{J} \right)^2}} \quad \text{(AISC F4-8)}$$

$$F_L = 0.7 F_y \quad \text{(AISC F4-6a)}$$

R_{pc} = web plastification factor, which is determined using a formula describe previously (AISC F4-9)

I_{yc} = moment of inertia of the compression flange about the minor axis

I_y = moment of inertia of the entire section about the minor axis.

3.5.3.1.1.3.3 Compression Flange Local Buckling

$$M_n = \begin{cases} R_{pc} M_y, & \text{if flanges are compact,} \\ \left[R_{pc} M_y - \left(R_{pc} M_y - F_L S_{33} \right) \left(\frac{\lambda - \lambda_{pt}}{\lambda_{rf} - \lambda_{pt}} \right) \right], & \text{if flanges are noncompact, and} \\ \frac{0.9 E k_c S_{33}}{\lambda^2}, & \text{if flanges are slender,} \end{cases}$$

(AISC F4-1, F4-13, F4-14)

where,

F_L = $0.5 F_y$ (AISC F4-6a, F4.3)

R_{pc} = is the web plastification factor, which is determined using a formula described previously (AISC F4-9, F4.3)

k_c = $\frac{4}{\sqrt{h/t_w}}$, $35 \le k_c \le 0.76$ (AISC F4.3, Table B4.1)

$\lambda \quad = \quad \dfrac{b_f}{2t_f}$

λ_{pf} = λ_p, the limiting slenderness for compact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3)

λ_{rf} = λ_r, the limiting slenderness for noncompact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3).

3.5.3.1.1.4 Slender Webs with Compact, Noncompact, and Slender Flanges

The nominal flexural strength is the lowest value obtained from the limit states of compression flange yielding, lateral-torsional buckling, and compression flange local buckling.

3.5.3.1.1.4.1 Compression Flange Yielding

$M_n = R_{pg} F_y S_{33}$, (AISC F5-1)

where R_{pg} is the bending strength reduction factor given by

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0, \qquad \text{(AISC F5-6)}$$

$$a_w = \frac{h t_w}{b_f t_f} \leq 10, \qquad \text{(AISC F5.2, F4-12)}$$

where h_0 is the distance between flange centroids (AISC F4.2).

3.5.3.1.1.4.2 Lateral-Torsional Buckling

$M_n = R_{pg} F_{cr} S_{33}$, (AISC F5-2)

where F_{cr} is the critical lateral-torsional buckling stress given by

$$F_{cr} = \begin{cases} F_y, & \text{if} \quad L_b \le L_p, \\ C_b \left[F_y - \left(0.3F_y\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y, & \text{if} \quad L_p < L_b \le L_r, \text{ and} \\ \dfrac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \le F_y, & \text{if} \;\; L_p > L_r, \end{cases}$$

(AISC F5-1, F5-3, F5-4)

where,

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$$ (AISC F5.2, 4-7)

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}}$$ (AISC F5-5)

$$r_t = \frac{b_f}{\sqrt{12\left(\dfrac{h_0}{d} + \dfrac{1}{6} a_w \dfrac{h^2}{h_0 d}\right)}}$$ (AISC F5.2, F4-11)

$R_{pg}$is the bending strength reduction factor, which has been described in the previous section.

3.5.3.1.1.4.3 Compression Flange Local Buckling

$M_n = R_{pg} F_{cr} S_{33}$, (AISC F5-7)

where F_{cr} is the critical buckling stress given by

$$F_{cr} = \begin{cases} F_y, & \text{if flanges are compact,} \\ \left[F_y - \left(0.3F_y\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right], & \text{if flanges are noncompact, and} \\ \dfrac{0.9Ek_c}{\left(\dfrac{b_f}{2t_f}\right)^2} \leq F_y, & \text{if flanges are slender,} \end{cases}$$

(AISC F5-1, F5-8, F5-9)

and λ, λ_{pf}, and λ_{rf} are the slenderness and the limiting slenderness ratios for compact and noncompact flanges from Table 3-1, respectively, and k_c is given by

$$k_c = \frac{4}{\sqrt{h/t_w}} \text{ where } 0.35 \leq k_c \leq 0.76. \qquad \text{(AISC 5.3)}$$

3.5.3.1.2 Minor Axis Bending

The nominal flexural strength is the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

3.5.3.1.2.1 Yielding

$M_n = M_p = F_y Z_{22} \leq 1.6 F_y S_{22}$, (AISC F6-1)

where S_{22} and Z_{22} are the section and plastic moduli about the minor axis, respectively.

3.5.3.1.2.2 Flange Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact flange,} \\ M_p - \left(M_p - 0.7F_y S_{22}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ F_{cr} S_{22}, & \text{for slender flanges,} \end{cases}$$

(AISC F6-1, F6-2, F6-3)

where,

$$F_{cr} = \frac{0.69E}{\lambda^2} \qquad \text{(AISC F6-4)}$$

$$\lambda = \frac{b_f}{2t_f} \qquad \text{(AISC F6.2)}$$

and λ_{pf} and λ_{rf} are the limiting slendernesses for compact and noncompact flanges, respectively, as described in Table 3-1 (AISC B4.1b).

3.5.3.2 Singly Symmetric I-Sections

3.5.3.2.1 Major Axis Bending

The nominal of flexural strength for major axes bending depends on compactness of the web and flanges.

3.5.3.2.1.1 Compact and Noncompact Webs with Compact, Noncompact and Slender Flanges

The nominal flexural strength is the lowest values obtained from the limit sates of compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

3.5.3.2.1.2 Compression Flange Yielding

$M_n = R_{pc}M_{yc}$, (AISC F4-1)

where, R_{pc} is the web plasticity factor, which is determined as follows:

$$R_{pc} = \begin{cases} 1 & \text{if} \quad I_{yc}/I_y \le 0.23, \\ \dfrac{M_p}{M_y}, & \text{if} \quad \lambda \le \lambda_{pw}, \text{and } I_{yc}/I_y > 0.23, \\ \left[\dfrac{M_p}{M_y} - \left(\dfrac{M_p}{M_y} - 1\right)\left(\dfrac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \dfrac{M_p}{M_y}, & \text{if} \quad \lambda_{pw} < \lambda_w \le \lambda_{rw}, \text{and } I_{yc}/I_y > 0.23, \end{cases}$$

(AISC F4-9a, F4-9b)

where,

$M_p \;=\; Z_{33}F_y \le 1.6S_{33c}F_y$ (AISC F4-2)

S_{33c} = elastic section modulus for major axis bending referred to compression flange

S_{33t} = elastic section modulus for major axis bending referred to tension flange

$\lambda = \dfrac{h_c}{t_w}$ (AISC F4.2, Table B4.1b)

λ_{pw} = λ_p, the limiting slenderness for a compact web, as given in Table 3-1 (AISC Table B4.1b)

λ_{rw} = λ_r, the limiting slenderness for a noncompact web, as given in Table 3-1 (AISC Table B4.1b)

and M_{yc} is the yield moment for compression flange yielding, which is determined as follows:

$M_{yc} = S_{33c}F_y$. (AISC F4-1)

3.5.3.2.1.3 Lateral-Torsional Buckling

$$M_n = \begin{cases} R_{pc}M_{yc}, & \text{if } L_b \le L_p \\ C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{33c}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc}, & \text{if } L_p < L_b \le L_r, \\ F_{cr}S_{33c} \le R_{pc}M_{yc}, & \text{if } L_b > L_r, \end{cases}$$

(AISC F4-1, F4-2, F4-3)

where,

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{JC}{S_{33}h_o}\left(\frac{L_b}{r_t}\right)^2}$$ (AISC F4-5)

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\dfrac{h_0}{d} + \dfrac{1}{6}a_w\dfrac{h^2}{h_0 d}\right)}}$$ (AISC F4-11)

$$a_w = \frac{h_c t_w}{b_{fe} t_{fc}} \le 10 \quad \text{(AISC F4-12)}$$

$$C = \begin{cases} 1, & \text{if} \quad I_{yc}/I_y > 0.23 \\ 0, & \text{if} \quad I_{yc}/I_y \le 0.23 \end{cases} \quad \text{(AISC F4.2)}$$

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad \text{(AISC F4-7)}$$

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{33} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L}{E} \frac{S_{33c} h_o}{J} \right)^2}} \quad \text{(AISC F4-8)}$$

$$F_L = \begin{cases} 0.7 F_y, & \text{if } \dfrac{S_{33t}}{S_{33c}} \ge 0.7 \\ \dfrac{S_{33t}}{S_{33c}} F_y \ge 0.5 F_y, & \text{if } \dfrac{S_{33t}}{S_{33c}} \le 0.7 \end{cases} \quad \text{(AISC F4-6a, F4-6b)}$$

R_{pc} = web plastification factor, which is determined using a formula describe previously (AISC F4-9)

I_{yc} = moment of inertia of the compression flange about the minor axis

I_y = moment of inertia of the section about the minor axis.

3.5.3.2.1.4 Compression Flange Local Buckling

$$M_n = \begin{cases} R_{pc} M_{yc}, & \text{if flanges are compact,} \\ \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{33c} \right) \left(\dfrac{\lambda - \lambda_{pt}}{\lambda_{rf} - \lambda_{pt}} \right) \right], & \text{if flanges are noncompact, and} \\ \dfrac{0.9 E k_c S_{33c}}{\lambda^2}, & \text{if flanges are slender,} \end{cases}$$

(AISC F4-1, F4-12, F4-13)

where,

F_L = is a calculated stress, which has been defined previously (AISC F4-6a, F4-6b, F4.3)

R_{pc} = is the web plastification factor, which is determined using a formula described previously (AISC F4-9, F4.3)

k_c = $\dfrac{4}{\sqrt{h/t_w}}$, $35 \le k_c \le 0.76$ (AISC F4.3, Table B4.1)

λ = $\dfrac{b_{fc}}{2t_{fc}}$

λ_{pf} = λ_p, the limiting slenderness for compact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3)

λ_{rf} = λ_r, the limiting slenderness for noncompact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3).

3.5.3.2.1.5 Tension Flange Yielding

$$M_n = \begin{cases} M_p, & \text{if } S_{33t} \ge S_{33c} \\ R_{pt} M_{yt}, & \text{if } S_{33t} < S_{33c} \end{cases} \qquad \text{(AISC F4-15)}$$

where, R_{pt} is the web plastification factor corresponding to the tension flange yielding limit state. It is determined as follows:

$$R_{pt} \begin{cases} \dfrac{M_p}{M_{yt}}, & \text{if } \lambda \le \lambda_{pw} \\ \dfrac{M_p}{M_{yt}} - \left(\dfrac{M_p}{M_{yt}} - 1\right)\left(\dfrac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right), & \text{if } \lambda_{pw} < \lambda \le \lambda_{rw} \end{cases}$$

(AISC F4-16a, F4-16b)

where,

M_p = $Z_{33}F_y$ (AISC F2-1)

S_{33c} = elastic section modulus for major axis bending referred to compression flange

S_{33t} = elastic section modulus for major axis bending referred to tension flange

$\lambda_w = \dfrac{h_c}{t_w}$ (AISC F4.4, Table B4.1b)

λ_{wp} = λ_p, the limiting slenderness for a compact web, as given in Table 3-1 (AISC Table B4.1b, F4.4)

λ_{rw} = λ_r, the limiting slenderness for a noncompact web, as given in Table 3-1. (AISC Table B4.1b, F4.4)

3.5.3.2.1.6 *Slender Webs with Compact, Noncompact and Slender Flanges*

The nominal flexural strength is the lowest value obtained from the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

3.5.3.2.1.6.1 Compression Flange Yielding

$M_n = R_{pg} F_y S_{33c}$, (AISC F5-1)

where, R_{pg} is the bending strength reduction factor given by

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0 \qquad \text{(AISC F5-6)}$$

$$a_w = \frac{h t_w}{b_f t_f} \le 10 \qquad \text{(AISC F5.2, F4-12)}$$

where, h_0 is the distance between flange centroids (AISC F2.2).

3.5.3.2.1.6.2 Lateral-Torsional Buckling

$M_n = R_{pg} F_{cr} S_{33c}$, (AISC F5-2)

where, F_{cr} is the critical lateral-torsional buckling stress given by

$$F_{cr} = \begin{cases} F_y, & \text{if } L_b \le L_p, \\ C_b \left[F_y - \left(0.3F_y\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y, & \text{if } L_p < L_b \le L_r, \text{ and} \\ \dfrac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \le F_y, & \text{if } L_p > L_r, \end{cases}$$

(AISC F5-1, F5-3, F5-4)

where,

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$$ (AISC F5.2, 4-7)

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}}$$ (AISC F5-5)

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\dfrac{h_0}{d} + \dfrac{1}{6} a_w \dfrac{h^2}{h_0 d}\right)}}$$ (AISC F5.2, F4-11)

R_{pg} is the bending strength reduction factor, which has been described in a previous section.

3.5.3.2.1.6.3 Compression Flange Local Buckling

$M_n = R_{pg} F_{cr} S_{33c}$, (AISC F5-7)

where, F_{cr} is the critical buckling stress given by

$$F_{cr} = \begin{cases} F_y, & \text{if flanges are compact,} \\ \left[F_y - \left(0.3F_y\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right], & \text{if flanges are noncompact, and} \\ \dfrac{0.9Ek_c}{\left(\dfrac{b_{fc}}{2t_{fc}}\right)^2} \le F_y, & \text{if flanges are slender,} \end{cases}$$

(AISC F5-1, F5-8, F -9)

and λ, λ_{pf}, and λ_{rf} are the slenderness and the limiting slenderness ratios for compact and noncompact flanges from Table 3-1, respectively, and k_c is given by

$$k_c = \frac{4}{\sqrt{h/t_w}}, \text{ where } 0.35 \le k_c \le 0.76. \qquad \text{(AISC 5.3)}$$

3.5.3.2.1.6.4 Tension Flange Yielding

$$M_n = \begin{cases} M_p & \text{if } \; S_{33t} \ge S_{33c}, \\ F_y S_{33t} & \text{if } \; S_{33t} < S_{33c}. \end{cases} \qquad \text{(AISC F5-10)}$$

3.5.3.2.2 Minor Axis Bending

The nominal flexural strength is the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

3.5.3.2.2.1 Yielding

$M_n = M_p = F_y Z_{22} \le 1.6 F_y S_{22}$, (AISC F6-1)

where, S_{22} and Z_{22} are the section and plastic moduli about the minor axis, respectively.

3.5.3.2.2.2 Flange Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact flange,} \\ M_p - \left(M_p - 0.7F_y S_{22}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ F_{cr} S_{22}, & \text{for slender flanges,} \end{cases}$$

(AISC F6-1, F6-2, F6-3)

where,

$$F_{cr} = \frac{0.69E}{\lambda^2} \qquad \text{(AISC F6-4)}$$

$$\lambda = \max\left\{\frac{b_{fb}}{t_{tb}}, \frac{b_{ft}}{t_{ft}}\right\} \qquad \text{(AISC F6.2)}$$

and λ_{pf} and λ_{rf} are the limiting slendernesses for compact and noncompact flanges, respectively, as described in Table 3-1 (AISC B4.1b).

3.5.3.3 Channel and Double Channel Sections

3.5.3.3.1 Major Axis Bending

The nominal flexural strength is the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, and compression flange local buckling.

3.5.3.3.1.1 Yielding

$M_n = M_p = F_y Z_{33}$, (AISC F2-1)

where Z_{33} is the plastic section modulus about the major axis.

3.5.3.3.1.2 Lateral-Torsional Buckling

$$M_n = \begin{cases} M_p, & \text{if } L_b \le L_p, \\ C_b\left[M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p & \text{if } L_p < L_b < L_r, \text{ and} \\ F_{cr} S_{33} \le M_p, & \text{if } L_p > L_r, \end{cases}$$

(AISC F2-1, F2-2, F2-3)

where S_{33} is the elastic section modulus taken about the major axis, L_b is the unbraced length, L_p and L_r are limiting lengths, and F_{cr} is the critical buckling stress. F_{cr}, L_p and L_r are given by

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_{33}h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(AISC F2-4)}$$

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(AISC F2-5)}$$

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_{33}h_0}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_{33}h_0}{Jc}\right)^2}} \qquad \text{(AISC F2-6)}$$

where

$$r_{ts}{}^2 = \frac{\sqrt{I_y C_w}}{S_{33}} \qquad \text{(AISC F2-7)}$$

$$C = \begin{cases} 1 & \text{for Double Channel sections} \\ \dfrac{h_o}{2}\sqrt{\dfrac{I_y}{C_w}} & \text{for Channel sections} \end{cases} \qquad \text{(AISC F2-8a, F2-8b)}$$

and h_0 is the distance between flange centroids.

3.5.3.3.1.3 Compression Flange Local Buckling

The nominal strength for compression flange local buckling is determined based on whether the web is compact, noncompact, or slender.

If the web is compact,

$$M_n = \begin{cases} M_p, & \text{for compact flanges,} \\ M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ \dfrac{0.9Ek_c S_{33}}{\lambda^2}, & \text{for slender flanges,} \end{cases}$$

(AISC F2-1, F3-1, F3-2)

if the web is noncompact,

$$M_n = \begin{cases} R_{pc} M_y, & \text{for compact flange,} \\ R_{pc} M_y - \left(R_{pc} M_y - F_L S_{33}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ \dfrac{0.9Ek_c S_{33}}{\lambda^2}, & \text{for slender flanges,} \end{cases}$$

(AISC F4-1, F4-12, F4-13)

and if the web is slender,

$$M_n = R_{pg} F_{cr} S_{33} \qquad \text{(AISC F5-7)}$$

where, $F_{cr}$is the critical buckling stress give by

$$F_{cr} = \begin{cases} F_y, & \text{if flanges are compact,} \\ F_y - \left(0.3F_y\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{if the flanges are noncompact,} \\ \dfrac{0.9Ek_c}{\lambda^2} \le F_y, & \text{if the flanges are slender,} \end{cases}$$

(AISC F5-1, F5-8, F5-9)

where,

λ $= \dfrac{b_f}{2t_f}$

λ_{pf} = λ_p, the limiting slenderness for compact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3)

λ_{rf} = λ_r, the limiting slenderness for noncompact flange, as given in Table 3-1 (AISC Table B4.1b, B4.3)

k_c $= \dfrac{4}{\sqrt{h/t_w}}, \quad 35 \leq k_c \leq 0.76$ (AISC F4.3, Table B4.1b)

F_L $= 0.7F_y$ (AISC F4-6a, F4.3)

$$R_{pc} = \begin{cases} 1 & \text{if} \quad I_{yc}/I_y \leq 0.23, \\ \dfrac{M_p}{M_y}, & \text{if} \quad \lambda \leq \lambda_{pw}, \text{and } I_{yc}/I_y > 0.23, \\ \left[\dfrac{M_p}{M_y} - \left(\dfrac{M_p}{M_y} - 1\right)\left(\dfrac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \dfrac{M_p}{M_y}, & \text{if} \quad \lambda_{pw} < \lambda_w \leq \lambda_{rw}, \text{and } I_{yc}/I_y > 0.23, \end{cases}$$

(AISC F4-9a, F4-9b)

R_{pg} $= 1 - \dfrac{a_w}{1200 + 300a_w}\left(\dfrac{h_c}{t_w} - 5.7\sqrt{\dfrac{E}{F_y}}\right) \leq 1.0$ (AISC F5-6)

S_{33} = elastic section modulus for major axis bending,

λ_w $= \dfrac{h_c}{t_w}$

λ_{pw} = λ_p, the limiting slenderness for compact web, as given in Table 3-1 (AISC Table B4.1)

λ_{rw} = λ_r, the limiting slenderness for a noncompact web, as given in Table 3-1 (AISC Table B4.1)

R_{pg} is the bending strength reduction factor, which has been described in a previous section.

3.5.3.3.2 Minor Axis Bending

The nominal flexural strength is the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

3.5.3.3.2.1 Yielding

$M_n = M_p = F_y Z_{22} \leq 1.6 F_y S_{22}$ (AISC F6-1)

where, S_{22} and Z_{22} are the section and plastic moduli about the minor axis, respectively.

3.5.3.3.2.2 Flange Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact flange,} \\ M_p - \left(M_p - 0.7F_y S_{22}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ F_{cr} S_{22}, & \text{for slender flanges,} \end{cases}$$

(AISC F6-1, F6-2, F6-3)

where,

$$F_{cr} = \frac{0.69E}{\lambda^2} \quad \text{(AISC F6-4)}$$

$$\lambda = \frac{b_f}{2t_f} \quad \text{(AISC F6.2)}$$

and λ_{pf} and λ_{rf} are the limiting slendernesses for compact and noncompact flanges, respectively, as described in Table 3-1 (AISC B4.1b).

3.5.3.4 Box Sections

This section applies to Box sections with compact or noncompact webs and compact, noncompact or slender flanges, bent about either axis. The program uses the same set of formulas for both major and minor direction bending, but with appropriate parameters.

The nominal flexural strength is the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling and web local buckling.

3.5.3.4.1 Yielding

$$M_n = M_p = F_y Z, \qquad \text{(AISC F7-1)}$$

where, Z is the plastic section modulus about the axis of bending.

3.5.3.4.2 Flange Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact flanges,} \\ M_p - \left(M_p - F_y S\right)\left(3.57\dfrac{b}{t}\sqrt{\dfrac{F_y}{E}} - 4.0\right) \le M_p, & \text{for noncompact flanges, and} \\ F_y S_{eff}, & \text{for slender flanges,} \end{cases}$$

(AISC F7-1, F7-2, F7-3)

where, S_{eff} is the effective section modulus determined using the effective compressive flange width, b_e,

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t}\sqrt{\frac{E}{F_y}}\right] \le b. \qquad \text{(AISC F7-4)}$$

See the "Reduction Factor for Slenderness" section for details (AISC F7, E7.2).

3.5.3.4.3 Web Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact webs, and} \\ M_p - \left(M_p - F_y S_{33}\right)\left(0.305\dfrac{h}{t_w}\sqrt{\dfrac{F_y}{E}} - 0.738\right) \le M_p, & \text{for noncompact webs,} \\ M_p - \left(M_p - F_y S_{33}\right)\left(0.305\dfrac{h}{t_w}\sqrt{\dfrac{F_y}{E}} - 0.738\right) \le M_p, & \text{for slender webs.} \end{cases}$$

(AISC F7-1, F7-5)

Note that the code does not cover the Box section flexure strength if the web is slender. The program uses the same flexure strength formula for Box sections with noncompact and slender webs, even though the formula applies only to noncompact section.

3.5.3.5 Pipe Sections

This section applies to pipe sections with D/t ratio less than $\frac{0.45E}{F_y}$. If a Pipe section violates this limit, the program reports an error.

The nominal flexural strength is the lowest value obtained according to the limit states of yielding (plastic moment) and local buckling. The same set of formulas is used for both major and minor axes of bending.

3.5.3.5.1 Yielding

$M_n = M_p = F_y Z$ (AISC F8-1)

3.5.3.5.2 Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact sections,} \\ \left(\dfrac{0.021E}{\frac{D}{t}} + F_y \right) S, & \text{for noncompact sections, and} \\ F_{cr} S, & \text{for slender sections,} \end{cases} \quad \text{(AISC F8-1, F8-2, F8-3)}$$

where, S is the elastic section modulus and F_{cr} is the critical buckling stress, where,

$$F_{cr} = \frac{0.33E}{\frac{D}{t}}. \quad \text{(AISC F8-4)}$$

3.5.3.6 T-Shapes and Double Angle Sections

3.5.3.6.1 Major Axes Bending

The nominal flexural strength for T-Shapes and Double Angles bent about their major (3-3) axis, i.e., the axis perpendicular to the axis of symmetry, is taken as the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, and flange local buckling.

3.5.3.6.1.1 Yielding

$$M_n = \begin{cases} M_p = F_y Z_{33} \le M_y, & \text{for stems in compression, and} \\ M_p = F_y Z_{33} \le 1.6 M_y, & \text{for stems in tension.} \end{cases}$$

(AISC F9-1, F9-2, F9-3)

3.5.3.6.1.2 Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_y GJ}}{L_b}\left[B + \sqrt{1+B^2}\right], \qquad \text{(AISC F9-4)}$$

where, B is taken conservatively as:

$$B = \pm 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}. \qquad \text{(AISC5 F9-5)}$$

The plus sign for B applies when the stem is in tension (M > 0) and the minus sign applies when the stem is in compression (M < 0).

3.5.3.6.1.3 Flange Local Buckling of Tee

$$M_n = \begin{cases} M_p, & \text{for compact flange,} \\ M_p - \left(M_p - 0.7 F_y S_{33}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \le 1.6 M_y, & \text{for noncompact flanges, and} \\ F_{cr} S_{22}, & \text{for slender flanges,} \end{cases}$$

(AISC F9-1, F9-6, F9-7)

where,

$$F_{cr} = \frac{0.7E}{\lambda^2} \qquad \text{(AISC 9-7)}$$

$$\lambda = \frac{b_f}{2t_f} \quad \text{(for T-Shapes)}$$

$$\lambda = \frac{b_f}{t_f} \quad \text{(for Double Angles)}$$

and λ_{pf} and λ_{rf} are the limiting slendernesses for compact and noncompact flanges, respectively, as described in Table 3-1 (AISC B4.1b).

3.5.3.6.1.4 Local Buckling for Tee Stems in Flexural Compression

When the flange is under compression (i.e., when the factored moment M_r is positive), the nominal moment capacity is taken as follows:

$$M_n = F_{cr} S_x, \qquad \text{(AISC F9-8)}$$

where. S_x is the elastic section modulus about the compression flange, and F_{cr} is determined as follows:

$$F_{cr} = \begin{cases} F_y, & \text{for } \dfrac{d}{t_w} \le 0.84\sqrt{\dfrac{E}{F_y}}, \\ F_y\left(1.19 - 0.50\left(b/t\right)\sqrt{\dfrac{F_y}{E}}\right), & 0.84\sqrt{\dfrac{E}{F_y}} < \dfrac{d}{t_w} < 1.03\sqrt{\dfrac{E}{F_y}}, \text{ and} \\ \dfrac{0.69E}{\left(b/t\right)^2}, & \dfrac{d}{t_w} > 1.03\sqrt{\dfrac{E}{F_y}}, \end{cases}$$

(AISC F9-9, F9-10, F9-11)

where,

$$\frac{b}{t} = \frac{d}{t_w}, \quad \text{(for T-Shapes)}$$

$$\frac{b}{t} = \frac{d}{t_w}, \quad \text{(for Double Angles)}$$

and d and t_w are described in Figure 3-1.

When the stem is in tension, i.e., when the factored moment M_r is positive, this limit state is not considered in the program.

3.5.3.6.2 Minor Axes Bending

The nominal flexural strength for T-Shapes and Double Angles bent about their minor (2-2) axes, i.e., the axis of symmetry, is taken as the lowest value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

3.5.3.6.2.1 Yielding

$M_n = M_p = F_y Z_{22} \leq 1.6 F_y S_{22}$ (AISC F6-1)

where, S_{22} and Z_{22} are the section and plastic moduli about the minor axis, respectively.

3.5.3.6.2.2 Flange Local Buckling

$$M_n = \begin{cases} M_p, & \text{for compact flange,} \\ M_p - \left(M_p - 0.7 F_y S_{22}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right), & \text{for noncompact flanges, and} \\ F_{cr} S_{22}, & \text{for slender flanges,} \end{cases}$$

(AISC F6-6)

where,

$$F_{cr} = \frac{0.69E}{\lambda^2} \quad \text{(AISC F6-4)}$$

$$\lambda = \frac{b_f}{2t_f} \quad \text{(for T-Shapes)}$$

$$\lambda = \frac{b_f}{t_f} \quad \text{(for Double Angles)}$$

and λ_{pf} and λ_{rf} are the limiting slendernesses for compact and noncompact flanges, respectively, as described in Table 3-1 (AISC B4.1b).

3.5.3.7 Single Angle Sections

The nominal flexural strength of angle sections is conservatively calculated based on the principal axes of bending. The nominal flexural strength about the major principal axis is the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, and leg local buckling.

3.5.3.7.1 Yielding

$M_n = 1.5 M_y$, (AISC F10-1)

where, M_y is the yield moment about the axis of bending.

3.5.3.7.2 Lateral-Torsional Buckling

The nominal flexure strength for bending about the major principal axis for the limit state of lateral-torsional buckling is given as follows:

$$M_n = \begin{cases} \left(0.92 - \dfrac{0.17M_e}{M_y}\right)M_e, & \text{if } M_e \le M_y \text{, and} \\ \left(0.92 - 1.17\sqrt{\dfrac{M_y}{M_e}}\right)M_y \le 1.5M_y, & \text{if } M_e > M_y, \end{cases}$$

(AISC F10-2, F10-3)

where M_e is the elastic lateral-torsional buckling moment defined as follows:

$$M_e = \begin{cases} \dfrac{0.46Eb^2t^2C_b}{L} & \text{for equal-leg angles,} \\ \dfrac{4.9EI_zC_b}{L^2}\left(\sqrt{\beta_w^2 + 0.052\left(\dfrac{Lt}{r_z}\right)^2} + \beta_w\right) & \text{for unequal-leg angles.} \end{cases}$$

(AISC F10-4, F10-5)

where,

C_b = lateral-torsional buckling modification factor for nonuniform moment diagram. It is computed using equation AISC F1-1. A limit on C_b is imposed ($C_b \le 1.5$) in the program (AISC F10).

L = laterally unbraced length of the member. It is taken as the max(L_{22}, L_{33}) in the program because L_{22} and L_{33} are not defined in the principal direction, in. (mm).

I_2 = minor principal axis moment of inertia, in.^{4} (mm^4),

r_t = radius of gyration for the minor principal axis, in. (mm),

t = angle leg thickness, in. (mm). It is taken as min(t_b, t_f)

β_w = a section property for unequal-legged angles. It is given as follows:

$$\beta_w = \frac{1}{I_w}\int_A z\left(w^2 + z^2\right)dA - 2z_0 \qquad \text{(AISC Table C-F10.2)}$$

β_w is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w should be used (AISC F10.2). It is conservatively taken as negative for unequal-leg angles.

z = coordinate along the minor principal axis

w = coordinate along the major principal axis

z_0 = coordinate of the shear center along the z-axis with respect to the centroid

I_w = major principal axis moment of inertia

I_z = minor principal axis moment of inertia.

In the preceding equation, M_y is taken as the yield moment about the major principal axis of bending, considering the possibility of yielding at the heel and both of the leg tips.

The nominal flexural strength for bending about the minor principal axis for the limit state of lateral-torsional buckling is not needed because the limit state of lateral-torsional buckling does not apply for minor axis bending.

3.5.3.7.3 Leg Local Buckling

The nominal flexural strength for bending about the major and minor principal axes of single angle sections for the limit state of h_g local buckling are given as follows:

$$M_n \begin{cases} 1.5F_yS_c & \text{if compact,} \\ F_yS_c\left(2.43-1.72\left(\frac{b}{t}\right)\right)\sqrt{\frac{F_y}{E}} & \text{if noncompact, and} \\ \frac{0.71E}{\left(b/t\right)^2}S_c & \text{if slender,} \end{cases}$$

(AISC F10-7, F10-8, F10-9)

where,

S_c = elastic section modulus to the toe in compression relative to the axis of bending

t = thickness of the leg under consideration

b = outside width of the leg under consideration.

In calculating the bending strengths for single-angles for the limit state of leg local buckling, the capacities are calculated for both the principal axes considering the assumption that either of the two tips (toes) can be under compression. The minimum capacities are considered.

3.5.3.8 Rectangular Sections

This section applies to rectangular sections bent about either axis.

The nominal flexural strength is the lowest value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

3.5.3.8.1 Yielding

$$M_n = M_p = F_y Z \leq 1.6 M_y \qquad \text{(AISC F11-1)}$$

3.5.3.8.2 Lateral-Torsional Buckling

Limit state of lateral-torsional buckling does not apply to rectangular sections bent about their moving axis. For rectangular sections, M_n about major axis, is given by the following:

$$M_e = \begin{cases} M_p, & \text{if } \dfrac{L_b d}{t^2} \leq \dfrac{0.08E}{F_y}, \\ C_b\left[1.52 - 0.274\left(\dfrac{L_b d}{t^2}\right)\dfrac{F_y}{E}\right]M_y \leq M_p, & \text{if } \dfrac{0.08E}{F_y} < \dfrac{L_b d}{t^2} \leq \dfrac{1.9E}{F_y}, \\ F_{cr} S_{33} \leq M_p, & \text{if } \dfrac{L_b d}{t^2} > \dfrac{1.9E}{F_y}, \end{cases}$$

(AISC F11-1, F11-2, F11-3)

where, F_{cr} is the critical buckling stress, which is given by

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}}, \qquad \text{(AISC F11-4)}$$

where, t is the width of the rectangular bar parallel to axis of bending; d is the depth of the rectangular bar; and L_b is the length unbraced against lateral displacement of the compression region or twist of the cross-section.

3.5.3.9 Circular Sections

The nominal flexural strength is the value obtained according to the limit states of yielding (plastic moment).

$$M_n = M_p = F_y Z \leq 1.6M_y \qquad \text{(AISC F11-1)}$$

For this section, the limit state of lateral-torsional buckling need not be considered (AISC F11.2(c)).

3.5.3.10 General Sections and Section Designer Sections

For General sections and Section Designer sections, the nominal major and minor direction bending strengths are assumed as:

$$M_n = M_y = SF_y.$$

The program does not check any lateral-torsional buckling, flange local buckling, web local buckling, or tension flange yielding. The program assumptions may not be conservative. The user is expected to calculate the capacity and overwrite it.

3.5.4 Nominal Shear Strength

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

In calculating nominal strength for shear, V_n, it is assumed that there is no intermedial stiffeners used to enhance shear strength of a section (AISC G2-2). The code allows the use of one of following methods: (a) the limit state of shear yielding and shear buckling without using tension-field-action (AISC

G2); and (b) post buckling strength of the member or tension-field-action (AISC G3). The program uses the first method to calculate shear strengths.

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v, are determined using the following factors.

For all sections in both the major and minor directions, except for the web of rolled I-Shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$

$$\phi_v = 0.9 \text{ (LRFD)} \qquad \text{(AISC G1)}$$

$$\Omega_v = 1.67 \text{ (LRFD)} \qquad \text{(AISC G1)}$$

For the web of rolled I-Shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$ and for major direction shear only

$$\phi_v = 1.0 \text{ (LRFD)} \qquad \text{(AISC G2.1a)}$$

$$\Omega_v = 1.50 \text{ (ASD)} \qquad \text{(AISC G2.1a)}$$

3.5.4.1 Shear in the Major Direction

The nominal shear strength for major direction, V_{n2}, is evaluated according to the limit states of shear yielding and shear buckling

$$V_n = 0.6 F_y A_w C_v, \qquad \text{(AISC G2-1)}$$

where A_w is the area of the web (overall depth times the web thickness, dt_w), and C_v is the web shear coefficient. The expressions of A_w and $C_v$differ from section to section, as follows.

3.5.4.1.1 I-Shapes

For all I-shaped members, A_w is taken as the overall depth of the member times the web thickness

$$A_w = dt_w. \qquad \text{(AISC G2.1(b))}$$

For the webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$, $C_v$is taken as 1.

$$C_v = 1. \qquad \text{(AISC G2-2)}$$

For webs of all other I-shaped members, including both singly and doubly symmetric and both rolled and welded sections, C_v is taken as follows:

$$C_v = \begin{cases} 1.0, & \text{if} \quad \dfrac{h}{t_w} \le 1.10\sqrt{k_v E/F_y}, \\ \dfrac{1.10\sqrt{k_v E/F_y}}{h/t_w}, & \text{if} \quad 1.10\sqrt{k_v E/F_y} < \dfrac{h}{t_w} \le 1.37\sqrt{k_v E/F_y}, \text{ and} \\ \dfrac{1.51Ek_v}{\left(h/t_w\right)^2 F_y}, & \text{if} \quad \dfrac{h}{t_w} > 1.37\sqrt{k_v E/F_y}, \end{cases}$$

(AISC G2-3, G2-4, G2-5)

where k_v is the web plate bucking coefficient and it is taken as

$k_v = 5.$ (AISC G2.1(b)(i))

In the preceding expression, for rolled shapes, h is taken as the clear distance between flanges less the fillet or corner radii, and for built-up welded sections, h, is taken as the clear distance between flanges (AISC G2.1(b), B4.2).

It should be observed that the ϕ_v factor, also the Ω_v factor, differs for the web of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$ from the web of other I-shaped members.

3.5.4.1.2 Channel, Double Channel, and T-Shape Sections

A_w is taken as the overall depth of the member times the web thickness

$A_w = dt_w$ for Channel sections, (AISC G2.1(b))

$A_w = 2dt_w$ for Double Channel sections, (AISC G2.1(b))

$A_w = dt_w$ for T-Shape sections. (AISC G2.1(b))

C_v is taken as follows:

$$C_v = \begin{cases} 1.0, & \text{if } \dfrac{h}{t_w} \le 1.10\sqrt{k_v\, E/F_y}, \\ \dfrac{1.10\sqrt{k_v\, E/F_y}}{h/t_w} & \text{if } 1.10\sqrt{k_v\, E/F_y} < \dfrac{h}{t_w} \le 1.37\sqrt{k_v\, E/F_y}, \text{ and} \\ \dfrac{1.51Ek_v}{(h/t_w)^2 F_y} & \text{if } \dfrac{h}{t_w} > 1.37\sqrt{k_v\, E/F_y}, \end{cases}$$

(AISC G2-3, G2-4, G2-5)

where,

$k_v = 5$ for Channel and Double Channel sections (AISC G2.1(b)(i))

$k_v = 1.2$ for T-Shape sections (AISC G2.1(b)(i))

and for Channel and Double Channel sections, h is taken as the clear distance between flanges less the fillet or corner radii (AISC G2.1(b), B4.2); and for T-Shape sections, h is taken as the overall depth (AISC G2.1(b)).

3.5.4.1.3 Box Sections

A_w is taken as follows:

$A_w = 2ht_w$. (AISC G5)

C_v is taken as follows:

$$C_v = \begin{cases} 1.0, & \text{if } \dfrac{h}{t_w} \le 1.10\sqrt{k_v\, E/F_y}, \\ \dfrac{1.10\sqrt{k_v\, E/F_y}}{h/t_w}, & \text{if } 1.10\sqrt{k_v\, E/F_y} < \dfrac{h}{t_w} \le 1.37\sqrt{k_v\, E/F_y}, \text{ and} \\ \dfrac{1.51Ek_v}{(h/t_w)^2 F_y}, & \text{if } \dfrac{h}{t_w} > 1.37\sqrt{k_v\, E/F_y}, \end{cases}$$

(AISC G5, G2-3, G2-4, G2-5)

where,

$k_v = 5$ (AISC G5)

$h = d - 3t_f$. (AISC G5)

Note that the design thickness of the Box section can differ by a reduction factor from its nominal thickness if the welding process is ERW (AISC B3.12). The choice of reduction and the reduction factor C can be overwritten on a member-by-member basis.

3.5.4.1.4 Angle Sections

A_w andC_v are taken as follows:

$$A_w = bt \qquad \text{(AISC G4)}$$

$$K_v = 1.2 \qquad \text{(AISC G4)}$$

where b is the width of the leg resisting the shear force.

3.5.4.1.5 Double Angle Sections

A_w andC_v are taken as follows:

$$A_w = 2bt \qquad \text{(AISC G4)}$$

$$K_v = 1.2 \qquad \text{(AISC G4)}$$

where b is the width of the leg resisting the shear force.

3.5.4.1.6 Rectangular, Circular (Round), General and SD Sections

For these sections, A_w is calculated as the effective shear area from the basic principle of elastic shear distribution, and C_v is taken as 1 for simplicity.

3.5.4.1.7 Pipe Section

The nominal shear strength, V_n, of round HSS (i.e, Pipe sections) according to the limit state of shear yielding and shear buckling is taken as

$$V_n = F_{cr} A_g / 2 \qquad \text{(AISC G6-1)}$$

where,

$$F_{cr} = \frac{0.78E}{(D/t)^{3/2}} \leq 0.6F_y. \qquad \text{(AISC G6-2b)}$$

The code allows F_{cr} to be taken as the maximum of two alternate values. The program conservatively uses one of the two expressions only. In the preceding equations,

A_g = gross area of section based on design wall thickness

D = outside diameter

t = design wall thickness.

The design wall thickness is equal to 0.93 times the nominal wall thickness for ERW HSS, and equal to the nominal wall thickness for SAW HSS. However, the choice of thickness reduction and the reduction factor can be overwritten in the program on a member-by-member basis.

3.5.4.2 Shear in Minor Direction

The nominal shear strength for minor directions, V_{n3}, is evaluated according to the limit states of shear yielding and shear buckling as follows:

$$V_n = 0.6F_y A_w C_v, \qquad \text{(AISC G2-1)}$$

where,A_w is the shear area for minor direction shear, andC_v is the web shear coefficient. The expression of A_w andC_v differs from section to section as follows.

3.5.4.2.1 I-Shapes, Channel, Double Channel, and T- Shapes

A_w is taken as the sum of flange areas.

$$A_w = \begin{cases} 2b_f t_f & \text{Doubly Symmetric I-Shapes} \\ b_{fb} t_{fb} + b_{ft} t_{ft} & \text{Singly Symmetric I-Shapes} \\ 2b_f t_f & \text{Channels} \\ 4b_f t_f & \text{Double Channels} \\ b_f t_f & \text{T Shapes} \end{cases} \qquad \text{(AISC G7)}$$

C_v is taken as follows:

$$C_v = \begin{cases} 1.0, & \text{if } \dfrac{h}{t_w} \le 1.10\sqrt{k_v E/F_y}, \\ \dfrac{1.10\sqrt{k_v E/F_y}}{h/t_w}, & \text{if } 1.10\sqrt{k_v E/F_y} < \dfrac{h}{t_w} \le 1.37\sqrt{k_v E/F_y}, \text{ and} \\ \dfrac{1.51Ek_v}{(h/t_w)^2 F_y}, & \text{if } \dfrac{h}{t_w} > 1.37\sqrt{k_v E/F_y}, \end{cases}$$

(AISC G7, G2-3, G2-4, G2-5)

where h/t_w is really meant for flange and is taken as follows,

$$\frac{h}{t_w} = \begin{cases} b_f/2t_f & \text{I Shapes} \\ b_f/t_f & \text{Channels} \\ b_f/t_f & \text{Double Channels} \\ b_f/2t_f & \text{T Shapes} \end{cases} \qquad \text{(AISC G7)}$$

and k_v is taken as 1.2,

$$k_v = 1.2. \qquad \text{(AISC G7)}$$

All dimensions used in the preceding equation are explained in Figure 3-1. For Singly Symmetric I-Shapes where each flange has its own properties, the shear capacity contribution is calculated for each flange separately based on its own dimensions, and then the combinations are added together.

3.5.4.2.2 Box Sections, Angles, and Double Angles

The shear capacity in the minor direction, V_{n3}, is calculated in exactly the same way as for calculation of the major shear capacity, V_{n2}, except that the appropriate dimensions are used (AISC G4, G5, G1).

3.5.4.2.3 Pipe Sections

The shear capacity in the minor direction, V_{n3}, is exactly the same as that for major direction (AISC G6).

3.5.4.2.4 Rectangular, Circular (Round), General and SD Sections

For these sections, A_w is calculated as the effective shear area from the basic principle of elastic stress distribution. C_v is taken as 1 for simplicity. Then, equation G2-1 is used to calculate the shear capacity.

3.5.5 Nominal Torsional Strength

The nominal torsion strengths are calculated for closed sections such as Boxes and Pipes only. Torsion is ignored in design for all other section types.

The design torsional strength, $\phi_T T_n$, and the allowable torsional strength, T_n/Ω_T, are determined using the following resistance and safety factors:

$\phi_T = 0.90$ (LRFD) (AISC H3.1)

$\Omega_T = 1.67$ (ASD) (AISC H3.1)

The nominal torsional strength, T_n, according to the limit states of torsional yielding and torsional buckling, is as follows:

$$T_n = F_{cr}C, \qquad \text{(AISC H3-1)}$$

where C is the torsional shear constant, and F_{cr} is the critical buckling stress.

For round HSS (i.e., Pipe sections), C is taken conservatively as

$$C = \frac{\pi(D-t)^2 t}{2} \qquad \text{(AISC H3.1 Note)}$$

F_{cr} is taken as,

$$F_{cr} = \max(F_{cr1}, F_{cr2}) \le 0.6F_y, \qquad \text{(AISC H3.1)}$$

where,

$$F_{cr1} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{5/4}}, \text{ and} \qquad \text{(AISC H3-2a)}$$

$$F_{cr2} = \frac{0.60E}{\left(\frac{D}{t}\right)^{3/2}}. \qquad \text{(AISC H3-2b)}$$

In the preceding equations,

L = torsional unbraced length of the member, L_{LTB}. This length is taken as the minor direction unbraced length for flexural buckling, L_{22}, by default. However, this length can be overwritten in the program.

D = outside diameter of the Pipe section

t = design wall thickness

For regular HSS (i.e., Box sections),

$$C = 2\left(b_f - t_w\right)\left(d - t_f\right)\left\{\min\left(t_w, t_f\right)\right\} - 4.5\left(4 - \pi\right)\left\{\left(\min\left(t_f, t_w\right)\right)^3\right\}$$

(AISC H3.1)

$$F_{cr} = \begin{cases} 0.6F_y, & \text{if } h/t \le 2.45\sqrt{E/F_y}, \\ 0.6F_y \dfrac{2.45\sqrt{E/F_y}}{\left(h/t\right)}, & \text{if } 2.45\sqrt{E/F_y} < h/t \le 3.07\sqrt{E/F_y}, \text{ and} \\ 0.458\pi^2 \dfrac{E}{\left(h/t\right)^2}, & \text{if } 3.07\sqrt{E/F_y} < h/t \le 26.0. \end{cases}$$

(AISC H3-3, H3-4, H3-5)

The variables b_f, t_w, d, t_f, h and t used in the preceding expression have been explained in Figure 3-1. In calculating h/t, the maximum of the ratio of depth to thickness and width to thickness are considered.

Here t is the design thickness. The design wall thickness is equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS. However, the choice of thickness reduction and the reduction factor can be overwritten in the program on a member-by-member basis.

3.6 Design of Members for Combined Forces

Previous sections of this design manual address members subject to only one type of force, namely axial tension, axial compression, flexure or shear. This section addresses the design of members subject to a combination of two or more of the individual forces.

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

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

3.6.1 Doubly and Singly Symmetric Members Subjected to Flexure and Axial Compression

The interaction of flexure and axial compression in all members with Doubly Symmetric sections (I-Shapes, Double Channel, Box, Pipe, Solid Circular, Solid Rectangular) and Singly Symmetric sections (Channel, T-Shape, Double Angle), with some exceptional cases, is given as follows:

For $\dfrac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{r33}}{M_{c33}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0. \qquad \text{(AISC H1-1a)}$$

For $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{r33}}{M_{c33}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0, \qquad \text{(AISC H1-1b)}$$

where, P_r and P_c are the required and available axial strengths; M_r and M_c are the required and available flexural strengths; and 3 and 2 represent the strong and weak axes, respectively.

Depending on the selected design provisions (LRFD or ASD), the aforementioned terms are defined as follows:

For design according to LRFD provisions:

P_t = required axial compressive strength using LRFD load combinations

P_c = design axial compressive strength = $\phi_c P_n$

M_r = required flexural strength using LRFD load combinations

M_c = design flexural strength = $\phi_b M_n$

ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for flexure = 0.90

For design according to ASD provisions:

P$_t$ = required axial compressive strength using ASD load combinations

P_c = allowable axial compressive strength = P_n/Ω_c

M_r = required flexural strength using ASD load combinations

M_c = allowable flexural strength = M_n/Ω_b

Ω_c = safety factor for compression = 1.67

Ω_b = safety factor for flexure = 1.67

As an exception, for Circular and Pipe sections, an SRSS (Square Root of Sum of Squares) combination is made first of the two bending components before adding the axial load component, instead of the single algebraic addition as implied by the interaction equations given by AISC H1-1a and AISC H1-1b. The resulting interaction equation is given by the following:

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\sqrt{\left(\frac{M_{r33}}{M_{c33}}\right)^2 + \left(\frac{M_{r22}}{M_{c22}}\right)^2}\right) \leq 1.0.$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\sqrt{\left(\frac{M_{r33}}{M_{c33}}\right)^2 + \left(\frac{M_{r22}}{M_{c22}}\right)^2}\right) \leq 1.0.$$

The philosophy behind the preceding modification is that the engineer has the freedom to choose the principal axis. The engineer can easily choose the principal axis to match with the resultant moment so that the design is always based on the uniaxial bending with axial force. In that case, the moment will be the resultant (SRSS) moment from the two components. The resultant D/C ratio calculated using the preceding equations will match the calculated D/C ratio from the pure resultant moment for the Pipe section. The reason is that M_n for the Pipe section is independent of the *K* and *L* factors. However, for solic circular (round) sections, the resultant stress ratio calculated using the preceding equations will not match exactly with the calculated D/C ratio from the pure resultant moment because M_n for this section depends on the K and L factors, which are known for only two given principal directions.

As an exception, for members with double symmetric sections in flexure and compression with negligible minor axis bending moment, two independent limit states, namely in-plane instability and out-of-plane buckling, also are considered separately considering the combined approach provided in AISC equations H1-1a and H1-1b. The minimum ratio provided by the original approach (AISC H1-1a, H1-1b) and the alternative approach described herein are taken as the resulting D/C ratio.

a) For the limit state of in-plane instability, equations H1-1a and H1-1b are used with M_c having a different meaning.

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{r33}}{M_{c33,\text{NoLTB}}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0. \qquad \text{(AISC H1-1a, H1.3a)}$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{r33}}{M_{c33,\text{NoLTB}}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0, \qquad \text{(AISC H1-1b, H1.3a)}$$

where, all the terms in the preceding equations are the same as explained earlier in this section, except $M_{c33,\text{NoLTB}}$ is explained as follows:

For LRFD:

$M_{c33,\text{NoLTB}} = \phi_b M_{n33}$ = design flexural strength determined in accordance with Chapter F, except that in determining M_{n33}, in this case, the lateral torsional buckling is not considered.

For ASD:

$M_{n33,\text{NoLTB}}$ = M_{n33}/Ω_b = allowable flexural strength determined in accordance with Chapter F, except that in determining M_{n33}, in this case, the lateral torsional buckling is not considered.

In general, $M_{n33,\text{NoLTB}}$ is either larger than the regular M_{n33}or equal to (for Pipe sections) the regular M_{n33}. The negligibility of the minor axis moment is tested in the program by using a tolerance (0.001) multiplied by the minor direction capacity (M_{n22}).

b) For the limit state of out-of-plane buckling, the following interaction equation is used

$$\frac{P_r}{P_{cy}}\left(1.5-0.5\frac{P_r}{P_{cy}}\right)+\left(\frac{M_{r33}}{C_b M_{c33}}\right)^2 \leq 1.0 \qquad \text{(AISC H1-2)}$$

where

P_{cy} = available compressive strength considering out-of-plane bending; it is taken equal to $\phi_c P_n$ (or P_n/Ω_c),

M_{c33} = available flexural strength considering all limit states, including lateral-torsional buckling.

As an exception, any singly-symmetric I-Shape section with I_{yc}/I_y beyond the range of $0.1 \leq I_{yc}/I_y \leq 0.9$ is considered beyond the scope of the code (AISC H.1, F13.2). Those sections are not checked by the program.

As an exception, all General sections and section created using Section Designer are treated as Doubly or Singly Symmetric sections. The equations H1-1a and H1-1b are use for calculation of the D/C ratios.

The program considers the left-hand side to calculate the D/C ratio. The D/C ratio is really compared with the D/C ratio limit rather than 1. By default, the D/C ratio limit is 0.95. This limit can be changed in the Preferences.

3.6.2 Doubly and Singly Symmetric Members Subjected to Flexure and Axial Tension

The interaction of flexure and axial tension in all members with Doubly Symmetric sections (I, Double Channel, Box, Pipe, Solid Circular, Solid Rectangular) and Singly Symmetric sections (Channel, T-Shapes, Double Angle), with some exceptional cases, are given as follows:

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{r33}}{M_{c33}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0 \qquad \text{(AISC H1.2, H1-1a)}$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{r33}}{M_{c33}} + \frac{M_{r22}}{M_{c22}}\right) \leq 1.0, \qquad \text{(AISCH1.2, H1-1b)}$$

where, P_r and P_c are the required and available axial strengths; M_r and M_c are the required and available flexural strengths; and 3 and 2 represent the strong and weak axes, respectively.

Depending on the selected design provisions (LRFD or ASD), the aforementioned terms are defined as follows:

For design in accordance with LRFD provisions:

P_t = required axial tensile strength using LRFD load combinations

P_c = design axial tensile strength = $\phi_c P_n$

M_r = required flexural strength using LRFD load combinations

M_c = design flexural strength = $\phi_b M_n$

ϕ_t = resistance factor for tension (0.90 yielding, 0.75 rupture) (AISC D2)

ϕ_b = resistance factor for flexure = 0.90

For design in accordance with ASD provisions:

P_t = required axial compressive strength using ASD load combinations

P_c = allowable axial compressive strength =P_n/Ω_c

M_r = required flexural strength using ASD load combinations

M_c = allowable flexural strength =M_n/Ω_b

Ω_t = safety factor for tension (1.67 yielding, 2.00 rupture) (AISC D2)

Ω_b = safety factor for flexure = 1.67

As an exception, for Circular and Pipe sections, an SRSS (Square Root of Sum of Squares) combination is made first of the two bending components before adding the axial load component, instead of the single algebraic addition as implied by the interaction equations given by AISC H1-1a and AISC H1-1b. The resulting interaction equation is given by the following:

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\sqrt{\left(\frac{M_{r33}}{M_{c33}}\right)^2 + \left(\frac{M_{r22}}{M_{c22}}\right)^2}\right) \leq 1.0$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\sqrt{\left(\frac{M_{r33}}{M_{c33}}\right)^2 + \left(\frac{M_{r22}}{M_{c22}}\right)^2}\right) \leq 1.0.$$

As an exception, any single-symmetric I-Shape section with I_{yc}/I_y beyond the range of $0.1 \leq I_{yc}/I_y \leq 0.9$ is considered beyond the scope of the code (AISC H.1, F13.2). These sections are not checked by the program.

As an exception, all General sections and section created using Section Designer are treated as Doubly or Singly Symmetric sections. The equations H1-1a and H1-1b are use for calculation of the D/C ratios.

The program considers the left-hand side to calculate the D/C ratio. The D/C ratio is really compared with the D/C ratio limit rather than 1. By default, the D/C ratio limit is 0.95. This limit can be changed in the Preferences.

3.6.3 Unsymmetric Members Subjected to Flexure and Axial Force

Unlike I-Shapes, Box, Channel, Double Channel, T-Shapes, Double Angle, Pipe, Circular, and Rectangular sections, the principal axes of unsymmetric (unequal leg) Single Angle sections do not coincide with their geometric axes. For Single Angle sections, the principal properties of the section are determined. The forces are resolved in the principal directions (w and z). The iteration of flexure and axial stress is calculated as follows:

$$\left| \frac{f_{ra}}{F_{ca}} + \frac{f_{rbw}}{F_{cbw}} + \frac{f_{rbz}}{F_{cbz}} \leq 1.0 \right| \qquad \text{(AISC H2-1)}$$

where f_{ra} and F_{ca} are the required and available axial stresses at the point of consideration; f_{ra} and F_{ca} are required and available bending stresses; w is the subscript related to the major principal axis bending; and z is the subscript related to the minor principal axis bending.

For design in accordance with LRFD provisions:

f_{ra} = required axial stress using LRFD load combinations

F_{ca} = design axial stress = $\phi_c F_{cr}$ or $\phi_t F_y$

f_{rbw}, f_{rbz} = required flexural stress using LRFD load combinations

F_{cbw}, F_{cbz} = design flexural stress = $\dfrac{\phi_b M_n}{S}$

ϕ_c = resistance factor for compression = 0.90

ϕ_f = resistance factor for tension (0.9 for yielding, 0.75 for rupture) (Section AISC D2)

ϕ_b = resistance factor for flexure = 0.90

For design in accordance with ASD provisions:

f_{ra} = required axial stress using ASD load combinations

F_a = allowable axial stress = F_{cr} / Ω_c or F_y / Ω_t

f_{rbw}, f_{rbz} = required flexural stress using ASD load combinations

F_{cbw}, F_{cbz} = allowable flexural stress = $\dfrac{M_n}{\Omega_b S}$

Ω_c = safety factor for compression = 1.67

Ω_f = safety factor for tension (1.67 for yielding, 2.00 for rupture) (Section AISC D2)

Ω_b = safety factor for flexure = 1.67

For General sections and Section Designer sections, it is assumed that the section properties are given in terms of principal directions. These two types of sections are treated as symmetric sections for interaction purposes.

3.6.4 Members Subject to Torsion, Flexure, Shear and Axial Force

The interaction of torsion, flexure, shear, and axial force is considered only for Box and Pipe sections and only if the torsion is significant. Torsion is considered significant when the required torsional strength, T_r, is more than 20% of the available torsional strength, T_c (AISC H3.2). The interaction of torsion with other forces is ignored when

$$T_r \leq 0.2 T_c \text{,} \qquad \text{(AICS H3.2)}$$

and for members with Box sections, the interaction of torsion, shear, flexure and axial force is given by

$$\frac{P_r}{P_c} + \left(\frac{M_{r33}}{M_{c33}} + \frac{M_{r22}}{M_{c22}} \right) + \left(\frac{V_{r3}}{V_{c3}} + \frac{V_{r2}}{V_{c2}} + \frac{T_r}{T_c} \right)^2 \leq 1.0, \qquad \text{(AISC H3-6)}$$

and for members with Pipe sections, the interaction of torsion, shear, flexure, and axial forces is given by

$$\frac{P_r}{P_c} + \left(\sqrt{\left(\frac{M_{r33}}{M_{c33}} \right)^2 + \left(\frac{M_{r22}}{M_{c22}} \right)^2} \right) + \left(\frac{V_{r3}}{V_{c3}} + \frac{V_{r2}}{V_{c2}} + \frac{T_r}{T_c} \right)^2 \leq 1.0, \qquad \text{(AISC H3-6)}$$

where, P_r and P_c are the required and available axial strength; M_r and M_c are required and available flexural strength; V_r and V_c are required and available shear strength; and T_r and T_c are required and available torsional strength, respectively.

For design in accordance with LRFD provisions:

P_r = required axial strength using LRFD load combinations

P_c = design compressive/tensile strength = $\phi_c P_n$ or $\phi_t P_n$

M_r = required flexural strength using LRFD load combinations

M_c = design flexural strength = $\phi_b M_n$

V_r = required shear strength using LRFD load combinations

V_c = desire shear strength = $\phi_v V_n$

T_r = required torsional strength using LRFD load combinations

T_c = design torsional strength = $\phi_T T_n$

ϕ_c = resistance factor for compression = 0.90

ϕ_t = resistance factor for tension (0.9 for yielding, 0.75 for rupture) (from AISC D2)

ϕ_b = resistance factor for flexure = 0.90

ϕ_v = resistance factor for shear = 0.90

ϕ_T = resistance factor for torsion = 0.9

For design in accordance with ASD provisions:

P_r = required axial strength using ASD load combinations

P_c = allowable compressive/tensile strength = P_n / Ω_c or P_n / Ω_t

M_r = required flexural strength using ASD load combinations

M_c = allowable flexural strength = M_n / Ω_b

V_r = required shear strength using ASD load combinations

V_c = allowable shear strength = V_n /Ω_v

T_r = required torsional strength using ASD load combinations

T_c = allowable torsional strength = T_n / Ω_r

Ω_c = safety factor for compression = 0.90

Ω_t = safety factor for tension (1.67 for yielding, 2.0 for rupture) (from AISC D2)

Ω_b = safety factor for flexure = 0.90

Ω_v = safety factor for shear = 0.90

Ω_r = safety factor for torsion = 0.90

Chapter 4
Special Seismic Provisions
(ANSI/AISC 341-10)

This chapter provides a detailed description of the algorithms related to special seismic provisions in the design/check of structures in accordance with the "ANSI/AISC 341-10—Seismic Provisions for Structural Steel Buildings" (AISC 2010c). The code option "AISC 360-10" covers these provisions. The same code option covers "ANSI/AISC 360-10—Specifications for Structural Steel Building" (AISC 2010a, b) as the basic code. The implementation covers load combinations from "ASCE/SEI 7-10," which is described in the section "Design Loading Combination" of Chapter 3. The loading based on "ASCE/SEI 7-10" has been described in a separate document entitled "CSI Lateral Load Manual" (CSI 2012). References are also made to IBC 2012 in this chapter.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- Reference to the ANSI/AISC 360-10 code carries the **AISC** prefix.
- Reference to the ANSI/AISC 341-10, Part I code carries the **AISC SEISMIC** prefix or sometimes **SEISMIC** only.
- Reference to the ASCE/SEI 7-10 code carries the **ASCE** prefix.

- Reference to the IBC 2012 code carries the **IBC** prefix.

4.1 Notations

Most of the notations used in this chapter have been described previously in Chapter 3. Any additional notations used in this chapter are described herein.

S_{DS} Design earthquake spectral response acceleration parameter at short period (ASCE 11.4.4)

ρ Redundancy factor (ASCE 12.3.4)

Ω_O Overstrength factor (ASCE Table 12.2-1)

E_h Effect of horizontal component of design seismic force, including the effect of ρ (ASCE 12.4.2.1)

E_v Effect of vertical component of design seismic force (ASCE 12.4.2.2)

E_{mh} Horizontal seismic load effects with overstrength factor (ASCE 12.4.3.1)

Q_E Effects of horizontal seismic forces (ASCE 12.4.2.1, 12.4.3.1)

4.2 Design Preferences

The steel frame design Preference are basic assignments that apply to all of the steel frame members. The Preferences are described in Appendix B. Tables B-1, B-2, and B-3 list the steel frame design Preferences. The following steel frame design Preferences are relevant to the special seismic provisions.

- Framing Type
- Notional Load Coefficient
- Seismic Design Category
- Ignore Seismic Code?
- Ignore Special Seismic Load?
- Is Doubler Plate Plug Welded?

4.3 Overwrites

The steel frame design Overwrites are basic assignments that apply only to those elements to which they are assigned. The overwrites are described in Appendix C. Table C-1 lists the steel frame design Overwrites. The following steel frame design overwrites are relevant to the special seismic provisions.

- Frame Type
- Omega0
- Overstrength factor, R_y

4.4 Supported Framing Types

The code (ANSI/AISC 341-10) now recognizes the following types of framing systems (AISC SEISMIC E, F).

Framing Type	**References**
OMF (Ordinary Moment Frame)	AISC SEISMIC E1
IMF (Intermediate Moment Frame)	AISC SEISMIC E2
SMF (Special Moment Frame)	AISC SEISMIC E3
STMF (Special Truss Moment Frame)	AISC SEISMIC E4
OCBF (Ordinary Concentrically Braced Frame)	AISC SEISMIC F1
SCBF (Special Concentrically Braced Frame)	AISC SEISMIC F2
EBF (Eccentrically Braced Frame)	AISC SEISMIC F3
BRBF (Buckling Restrained Braced Frame)	AISC SEISMIC F4
SPSW (Special Plate Shear Wall)	AISC SEISMIC F5

With regard to these framing types, the program has implemented specifications for all types of framing systems, except STMF, BRBF, and SPSW. Implementing those three types of framing requires further information about modeling.

The program recognizes the OCBF framing in its two separate incarnations: OCBF for regular Ordinary Concentrically Braced Frames (AISC SEISMIC

F1) and OCBFI for (base) Isolated Ordinary Concentrically Braced Frames (AISC SEISMIC F1.7).

By default, the frame type is taken as Special Moment Frame (SMF) in the program. However, the default frame type can be changed in the Preference form for all frames or in the Overwrites form on a member-by-member basis (Chapter 3). If a frame type Preference is revised in an existing model, the revised frame type does not apply to frames that have already been assigned a frame type through the Overwrites; the revised Preference applies only to new frame members added to the model after the Preference change and to the old frame members that were not assigned a frame type though the Overwrites.

4.5 Applicability of the Seismic Requirements

Ideally, the special seismic provisions shall apply when the seismic response modification factor, R, is taken as greater than 3, regardless of the seismic design category (AISC SEISMIC A1). R is specified by the applicable building code (local code or ASCE/SEI 7). IBC actually refers to ASCE 7-10 for the value of R (IBC 2205.2.1, ASCE Table 12.2-1). Again, when R is taken as 3 or less, the structure is not required to satisfy these provisions, unless specifically required by the applicable building code.

The applicable building code generally restricts buildings designed with an R factor of 3 or less to Seismic Design Category (SDC) A, B, or C; however, some systems that have R factors less than 3 are permitted in SDC D, E, or F (IBC 2205.2, ASCE 12.2.1, ASCE Table 12.2-1).

The program assumes that the special seismic provisions are applicable to any structural steel structure that is assigned to SDC D, E, or F, irrespective of the value of R, and to any structural steel structure designed with an R greater than 3 and that is assigned to SDC A, B, or C. In addition, the program allows the user to change the default applicability of special seismic provisions using the “Ignore Seismic Code?” Preference item.

4.6 Design Load Combinations

The program creates the default design load combinations based on the specification of ASCE 7-10 code. The default load combinations are generated differently when the LRFD (ASCE 2.3.2) and ASD (ASCE 2.4.1) provisions are chosen. The default combinations have been described in "Design Loading Combinations" in Chapter 3. Additional information may be found in "Design Load Combinations" in Chapter 2. The user may change the default load combinations if the governing code is different (AISC SEISMIC B2).

The program assumes that the defined earthquake load is really the strength level earthquake, which is equivalent to Q_E as defined in Section 12.4.2.1 in ASCE 7-10 code. For a regular earthquake, load is considered to have two components: horizontal, E_h and vertical E_v, which are taken as

$$E_h = \rho Q_E, \qquad \text{(ASCE 12.4.2.1)}$$

$$E_v = 0.2 S_{DS} D, \qquad \text{(ASCE 12.4.2.2)}$$

where, ρ is the redundancy factor as defined in Section 12.3.4 of ASCE 7-10, and the S_{DS} is the design earthquake spectral response acceleration parameters at short periods, as defined in Section 11.4.4 of ASCE 7-10 code.

Effectively, the seismic load combinations for the LRFD provision become

$$\left(1.2 + 0.2 S_{DS}\right) \text{ DL } \pm \rho Q_E \qquad \text{(ASCE 2.3.2-5, 12.4.2.3)}$$

$$\left(1.2 + 0.2 S_{DS}\right) \text{ DL } \pm \rho Q_E + 1.0 \text{ LL} \qquad \text{(ASCE 2.3.2-5, 12.4.2.3)}$$

$$\left(0.9 - 0.2 S_{DS}\right) \text{ DL } \pm \rho Q_E. \qquad \text{(ASCE 2.3.2-7, 12.4.2.3)}$$

The seismic load combinations for the ASD provision become

$$\left(1.0 + 0.14 S_{DS}\right) \text{ DL } \pm 0.7 \rho Q_E \qquad \text{(ASCE 2.4.1-5, 12.4.2.3)}$$

$$\left(1.0 + 0.105 S_{DS}\right) \text{ DL } \pm 0.75\left(0.7\rho\right) Q_E + 0.75 \text{ LL} \qquad \text{(ASCE 2.4.1-6b, 12.4.2.3)}$$

$$\left(0.6 - 0.14 S_{DS}\right) \text{ DL } \pm 0.7 \rho Q_E. \qquad \text{(ASCE 2.4.1-8, 12.4.2.3)}$$

The program automatically considers seismic load effects, including overstrength factors (ASCE 12.4.3), as special load combinations that are created automatically from each load combination involving seismic loads. In that case, the horizontal component of the force is represented by E_{mh}, and vertical component of the force is represented by E_v, where,

$$E_{mh} = \Omega_o Q_E \text{ and} \qquad \text{(ASCE 12.4.3.1)}$$

$$E_v = 0.2 S_{DS} D. \qquad \text{(ASCE 12.4.2.2)}$$

Effectively, the special seismic combinations for the LRFD provision are

$$(1.2 + 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E \qquad \text{(ASCE 2.3.2-5, 12.4.3.2)}$$

$$(1.2 + 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E + 1.0\text{LL} \qquad \text{(ASCE 2.3.2-5, 12.4.3.2)}$$

$$(0.9 - 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E \qquad \text{(ASCE 2.3.2-7, 12.4.3.2)}$$

and for the ASD provision, the combinations are

$$(1.0 + 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E \qquad \text{(ASCE 2.4.1-5, 12.4.3.2)}$$

$$(1.0 + 0.105S_{DS})\text{DL} \pm 0.75(0.7\Omega_0)Q_E + 0.75\text{LL} \qquad \text{(ASCE 2.4.1-6b,12.4.3.2)}$$

$$(0.6 - 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E \qquad \text{(ASCE 2.4.1-8,12.4.3.2)}$$

The factor S_{DS} is described earlier in this section. Ω_0 is the seismic force amplification factor that is required to account for structural overstrength. The factor Ω_0 is the overstrength factor, and it should be picked up from ASCE 7-10, Table 12.2-1 by the user and input in the Preferences and auto-seismic load definition. The default value of Ω_0 is taken as 3.0 in the program. If the user defines one or more auto-seismic loads, the values of Ω_0 are taken as the maximum of all the Ω_0 defined for each auto-seismic load case. Also if the user defines special seismic data that includes an Ω_0 value and requests that the program include the special seismic design data, then this Ω_0 takes precedence over the default values and those defined for the auto-seismic load cases. Moreover, Ω_0 can be overwritten for each individual member. The overwritten Ω_0 gets the highest precedence. The guideline for selecting a reasonable value for Ω_0 can be found in ASCE 7-10, Table 12.2-1.

Those combinations involving Ω_0 are internal to the program. The user does NOT need to create additional load combinations for those load combinations. The special circumstances for which those load combinations are additionally checked are described later in this chapter, as appropriate. The special load combination factors are applied directly to the 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 load cases.

The program assumes that the seismic loads defined as the strength level load is the program load case. Otherwise, the factors ρ, Ω_0, and S_{DS} will be unable to scale the load to the desired level.

4.7 Classification of Sections for Local Buckling

The sections are classified in the program as Seismically Compact, Compact, Noncompact, Slender, and Too Slender. The sections are classified as Compact, Noncompact, or Slender sections in accordance with Section B4 of the regular code (AISC B4, AISC Table B4.1). The limiting width/thickness ratios, λ_p, λ_r, and λ_s, that demarcate the slenderness limits for Compact, Noncompact, Slender and Too Slender sections were given in Table 3-1 in "Classification of Sections for Local Buckling" in Chapter 3.

Seismically compact sections are capable of developing the full plastic strength before local buckling occurs when the section goes through low cycle fatigue and withstands reversal of load under seismic conditions. The width/thickness ratio (λ) should be less than the limit, λ_{md} or λ_{hd}, as appropriate, for the section to be Seismically Compact. The limiting width/thickness ratios, λ_{md} and λ_{hd}, for compression elements are given in Table 4-1 and are based on the Seismic code (AISC SEISMIC D1.1b, Table D1.1). The Seismically Compact sections are reported as "Seismically Compact," or sometimes as "Seismic" only for brevity in the design output.

For members designated as moderately ductile members, the width-to-thickness ratios of compression elements shall not exceed the limiting width-to-thickness ratio, λ_{md}. For members designated as highly ductile members, the width-to-thickness ratio of the compression elements shall not exceed the limiting width-to-thickness ratio, λ_{hd}.

The table uses the dimension-related variables, such as b_f, t_f, t_w, b, t, h, D, and so forth. Some of those variable have been explained in the table itself.

Table 4-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections

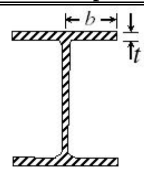
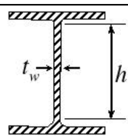
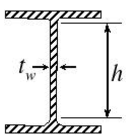
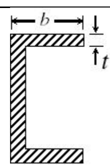
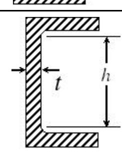
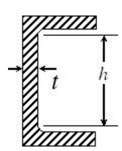
Section Type	Description of Element	Graphical Example	Given Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Elements: Highly Ductile Members (λ_{hd})	Limiting Width-Thickness Ratios for Compression Elements: Moderately Ductile Members (λ_{md})
Singly or Doubly Symmetric I Shapes	Flexural or uniform compression in flanges of rolled and welded I-Shaped sections		1	$b_f/2t_f$	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
	Flexural compression in webs of diagonal braces		5	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$
	Webs in flexural compression or combined flexural and axial compression		6	h/t_w	If $C_a \le 0.125$ $2.45\sqrt{\frac{E}{F_y}}(1-0.93C_a)$ If $C_a > 0.125$ $0.77\sqrt{\frac{E}{F_y}}(2.93-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$	If $C_a \le 0.125$ $3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$ If $C_a > 0.125$ $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$
Channel	Flexural or Uniform compression in flanges		1	b_f/t_f	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
	Flexural compression in webs of diagonal braces		5	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$
	Webs in flexural compression or combined flexural and axial compression		6	h/t_w	If $C_a \le 0.125$ $2.45\sqrt{\frac{E}{F_y}}(1-0.93C_a)$ If $C_a > 0.125$ $0.77\sqrt{\frac{E}{F_y}}(2.93-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$	If $C_a \le 0.125$ $3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$ If $C_a > 0.125$ $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$

Table 4-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections

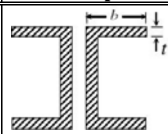
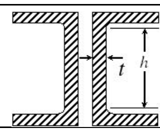
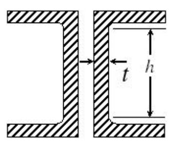
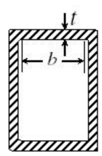
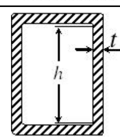
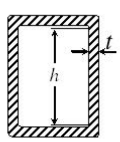
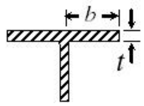
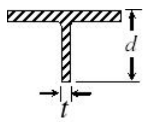
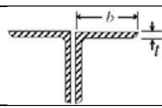
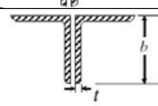
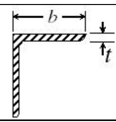
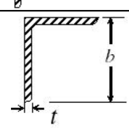
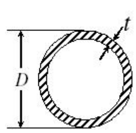
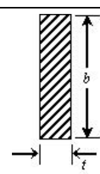
Section Type	Description of Element	Graphical Example	Given Case No.	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Elements: Highly Ductile Members (λ_{hd})	Limiting Width-Thickness Ratios for Compression Elements: Moderately Ductile Members (λ_{md})
Double Channel	Flexural or Uniform compression in flanges		1	b_f/t_f	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
Double Channel	Flexure compression in beam web		5	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$
Double Channel	Webs in flexural compression or combined flexural and axial compression		6	h/t_w	If $C_a \le 0.125$ $2.45\sqrt{\frac{E}{F_y}}(1-0.93C_a)$ If $C_a > 0.125$ $0.77\sqrt{\frac{E}{F_y}}(2.93-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$	If $C_a \le 0.125$ $3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$ If $C_a > 0.125$ $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$
Box	Flexural or axial compression of flanges under major axis bending		4	b/t	$0.55\sqrt{E/F_y}$ $0.60\sqrt{E}/F_y$ (columns in SMF)	$0.64\sqrt{E/F_y}$ (braces) $1.12\sqrt{E}/F_y$ (beams) $1.12\sqrt{E}/F_y$ (columns)
Box	Flexural compression of webs under minor axis bending		4	h/t	$0.55\sqrt{E/F_y}$ $0.60\sqrt{E}/F_y$ (columns in SMF)	$0.64\sqrt{E/F_y}$ (braces) $1.12\sqrt{E}/F_y$ (beams) $1.12\sqrt{E}/F_y$ (columns)
Box	Webs in flexural compression or combined flexural or axial compression		6	h/t	If $C_a \le 0.125$ $2.45\sqrt{\frac{E}{F_y}}(1-0.93C_a)$ If $C_a > 0.125$ $0.77\sqrt{\frac{E}{F_y}}(2.93-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$	If $C_a \le 0.125$ $3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$ If $C_a > 0.125$ $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \ge$ $1.49\sqrt{\frac{E}{F_y}}$

Table 4-1 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections

Section Type	Description of Element	Graphical Example	Given Case No.	Width-Thick-ness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Elements: Highly Ductile Members (λ_{hd})	Limiting Width-Thickness Ratios for Compression Elements: Moderately Ductile Members (λ_{md})
T Shape	Flexural or axial compression in flanges		1	$b_f/2t_f$	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
	Compression in stems with $M_{33} < 0$		3	d/t_w	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
Double Angle	Any type of compression in leg		1	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
	Any type of compression in leg		1	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
Angle	Any type of compression in any leg		1	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
	Any type of compression in any leg		1	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
Pipe	Flexural or axial compression		8	D/t	$0.038\frac{E}{F_y}$	$0.044\frac{E}{F_y}$ (braces) $0.070\frac{E}{F_y}$ (beams) $0.070\frac{E}{F_y}$ (columns)
Round Bar	———	———	——	———	Assumed non-compact	
Rectangular	Any compression			b/t	2.5	2.5
General	———	———	——	———	Assumed non-compact	
SD Section	———	———	——	———	Assumed non-compact	

Some of those variable are explained in Figure 3-1 and also explained in the code (AISC B4.1, B4.2, Table B4.1). The variable C_a can be expressed as follows:

$$C_a = \begin{cases} P_u / (\phi_c P_y) & \text{for LRFD} \\ \Omega_c P_a / P_y & \text{for ASD} \end{cases} \qquad \text{(AISC SEISMIC Table D1.1 Note [d])}$$

$$C_a \geq 0 \qquad \text{(AISC SEISMIC Table D1.1 Note [d])}$$

where,

P_a = Required compressive strength, used zero for tensile force (ASD)

P_u = Required compressive strength, used zero for tensile force (LRFD)

P_y = Axial yield strength

ϕ_b = 0.90

Ω_b = 1.67

When satisfying the Special Seismic criteria, it is in general not necessary for the design sections to be Seismically Compact (AISC SEISMIC D1.1b). However, for certain special cases, the design sections must be Seismically Compact (AISC SEISMIC D1.1a, D1.1b) or Compact (AISC SEISMIC D1, AISC Table B4.1) as described in the "Seismic Requirements" section of this chapter. For a situation when the code requires the design section to be Seismically Compact, but the section fails to satisfy the criteria, the user must modify the section. In that case, the program issues an error message in the output.

4.8 Special Check for Column Strength

The axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the amplified seismic load combinations (AISC SEISMIC B2, D1.4a(2), ASCE 12.4.3.2).

For LRFD provisions,

$$(1.2 + 0.2S_{DS})DL \pm \Omega_0 Q_E \qquad \text{(ASCE 2.3.2-5, 12.4.3.2)}$$

$(1.2 + 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E + 1.0\text{LL}$ (ASCE 2.3.2-5, 12.4.3.2)

$(0.9 - 0.2S_{DS})\text{DL} \pm \Omega_0 Q_E$ (ASCE 2.3.2-7, 12.4.3.2)

For ASD provisions,

$(1.0 + 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E$ (ASCE 2.4.1-5, 12.4.3.2)

$(1.0 + 0.105S_{DS})\text{DL} \pm 0.75(0.7\Omega_0)Q_E + 0.75\text{LL}$ (ASCE 2.4.1-6b,12.4.3.2)

$(0.6 - 0.14S_{DS})\text{DL} \pm 0.7\Omega_0 Q_E$ (ASCE 2.4.1-8,12.4.3.2)

where Ω_0 and S_{DS} are as described in the "Design Load Combinations" section in Chapter 3.

The preceding load combinations are used to calculate axial force only because other forces and moments are ignored. This axial capacity check is in addition to regular strength checks for the regularly specified load combinations (AISC SEISMIC D1.4a).

Those combinations involving Ω_0 are internal to the program. The user does NOT need to create additional load combinations for such load combinations. The special circumstances for which these load combinations are additionally checked are described later in this chapter, as appropriate. The special load combination factors are applied directly to the 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 load cases.

If the overwrite item "Ignore Special Seismic Load?" is set to yes, the preceding check will not be performed.

4.9 Member Design

This section describes the special requirements for designing a member. The section has been divided into subsections for each framing type.

4.9.1 Ordinary Moment Frames (OMF)

For this framing system, the following additional requirement is checked and reported (AISC SEISMIC E1).

- In columns, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the *special seismic load combinations* as described previously in the “Special Check for Column” section of this manual (AISC SEISMIC B2, D1.4a(2)).

4.9.2 Intermediate Moment Frames (IMF)

For this framing system, the following additional requirements are checked and reported (AISC SEISMIC E2).

- In columns, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the *special seismic load combinations* as described previously in the “Special Check for Column” section of this manual (AISC SEISMIC B2, D1.4a(2)).

- All beams and columns are checked to be Seismically Compact for moderately ductile members (AISC SEISMIC E2.5a, D1.1 Table D1.1, AISC Table B4.1). The limits of the width-thickness ratio, λ_{md}, have been presented in this manual in Table 4-1. If this criterion is not satisfied, the program issues an error message.

- The program checks the laterally unbraced length of beams between lateral braces not to exceed the limiting value $0.17\left(E/F_y\right)r_y$ consistent with moderately ductile steel beams (AISC SEISMIC E2.4a, D1.2a(a)(3)). If this criterion is not satisfied, the program issues an error message.

4.9.3 Special Moment Frames (SMF)

For this framing system, the following additional requirements are checked or reported (AISC SEISMIC E3).

- In columns, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the *special seismic load combinations* as described previously in the “Special Check for Column” section of this manual (AISC SEISMIC B2, D1.4a(2)).

- All beams and columns are required to be Seismically Compact for highly ductile members (AISC SEISMIC E3.5a, D1.1, Table D1.1). The limits of the width-thickness ratio, λ_{hd} , has been presented in this manual in Table 4-1. If these criteria are satisfied, the section is reported as SEISMIC as described earlier under the “Classification of Sections for Local Buckling” section. If these criteria are not satisfied, the program issues an error message.

- The program checks the slenderness ratio, L/r, for columns to be less than 60 (AISC SEISMC E3.4c(2)(2)). If this criterion is not satisfied, the program issues an error message.

- The program checks the laterally unsupported length of beams not to exceed $0.086\left(E/F_y\right)r_y$ consistent with highly ductile steel beams (AISC SEISMIC E3.4b, D1.2b). If this criterion is not satisfied, the program issues an error message.

4.9.4 Special Truss Moment Frames (STMF)

No special consideration for this type of framing system is given by the program. The user is required to check the seismic design requirements for STMF independently.

4.9.5 Ordinary Concentrically Braced Frames (OCBF)

For this framing system, the following additional requirements are checked or reported (AISC SEISMIC F1.1).

- In columns, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load combinations* as described previously in the “Special Check for Column” section of this manual (AISC SEISMIC F1.2, B2, D1.4a(2)).

- All brace members are checked to be Seismically Compact consistent with moderately ductile members in accordance with Table 4-1 (AISC SEISMIC F1.5a, D1.1, Table D1.1). If the criteria are satisfied, the section is reported as SEISMIC as described earlier in “Classification of Sections for Local Buckling” in this chapter. If this criterion is not satisfied, the program issues an error message.

- The maximum Kl/r ratio of the braces for V or inverted-V configurations is checked not to exceed $4.0\sqrt{E/F_y}$ (AISC SEISMIC F1.5b). If this criterion is not met, an error message is reported in the output.

Note: Beams intersected by chevron (V or inverted-V) braces are NOT currently checked to have a strength to support loads for the following two conditions (AISC SEISMIC 1):

(a) A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads from load combinations stipulated by the code, assuming the bracings are not present (AISC SEISMIC F1.4a(1)).

(b) A beam that is intersected by braces and supporting earthquake load shall be designed to resist the effects of the load combinations stipulated by the code, except the brace forces have to be replaced by their capacities. The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g$ (AISC SEISMIC F1.4a(1)(i)). The forces in all braces in compression shall be assumed to be equal to $0.3 P_n$ (AISC SEISMIC F1.4a(1)(ii)).

4.9.6 Ordinary Concentrically Braced Frames from Isolated Structures (OCBFI)

For this framing system, the following additional requirements are checked or reported (AISC SEIAMIC F1, F1.7).

- In columns, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the *special seismic load combinations* as described previously in the “Special Check for Column” section of this manual (AISC SEISMIC F1.2, B2, D1.4a(2)).

- All braces are required to be Seismically Compact consistent with moderately ductile members (AISC SEISMIC F1.1, F1.5a, D1.1, Table D1.1). If the criteria are satisfied, the section is reported as SEISMIC as described earlier in “Classification of Sections for Local Buckling” in this chapter. If this criterion is not satisfied, the program issues an error message.

- The maximum Kl/r ratio for the braces is checked not to exceed $4\sqrt{E/F_y}$ (AISC SEISMIC F1.1, F1.7b). If this criterion is not met, an error message is reported in the output.

4.9.7 Special Concentrically Braced Frames (SCBF)

For this framing system, the following additional requirements are checked or reported (AISC SEISMIC E3).

- In columns, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load combinations* as described previously in the "Special Check for Column" section of this manual (AISC SEISMIC B2, D1.4a(2)).
- All column and brace members are checked to be Seismically Compact for highly ductile members in accordance with Table 4-1 (AISC SEISMIC F2.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under "Classification of Sections for Local Buckling" in this chapter. If these criteria are not satisfied, the program issues an error message.
- All beam members are checked to be Seismically Compact for moderately ductile members in accordance with Table 4-1 (AISC SEISMIC F2.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under "Classification of Sections for Local Buckling" in this chapter. If these criteria are not satisfied, the program issues an error message.
- The maximum Kl/r ratio of the braces is checked not to exceed 200 (AISC SEISMC F2.5b(1)). If this criterion is not satisfied, the program issues an error message.

The program checks the following requirements for V-type and inverted V-type SCBF frames.

- The program checks the laterally unsupported length of beams not to exceed the limiting value $0.17\left(E/F_y\right)r_y$ (AISC SEISMIC F2.4b(2), D1.2a). If this criterion is not satisfied, the program issues an error message.
- The columns and beams are designed for load combinations, including the automatically generated special seismic combinations involving amplified seismic load. These combinations are the same as described in Section 4.8 "Special Check for Column Strength." However, for this case, all compo-

nents of forces, including moments and shears in addition to axial forces, are included.

4.9.8 Eccentrically Braced Frames (EBF)

For this framing system, the program looks for and recognizes the eccentrically braced frame configurations shown in Figure 4-1. The following additional requirements are checked or reported for the beams, columns and braces associated with these configurations (AISC SEISMIC F3).

- In columns, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the *special seismic load combinations* as described previously in the "Special Check for Column" section of this manual (AISC SEISMIC B2, D1.4a(2)).

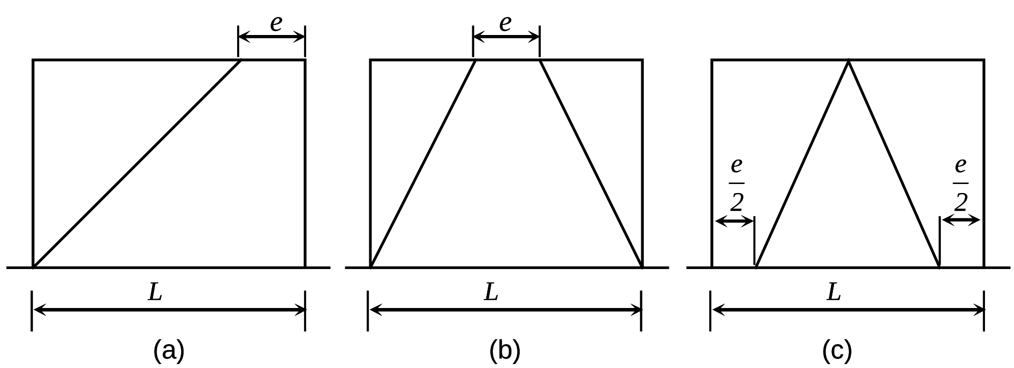


Figure 4-1 Eccentrically Braced Frame Configurations

- The beams are checked to be Seismically Compact for highly ductile members in accordance with Table 4-1 (AISC SEISMIC F3.5b(1), D1.1, Table D1.1). If this criterion is satisfied, the section is reported as SEISMIC as described earlier under "Classification of Sections for Local Buckling Section" in this chapter. If this criterion is not satisfied, the program issues an error message.

- The shear strength for link beams is taken as follows (AISC SEISMIC 15.2b):

$V_u \le \phi_v V_n$ (LRFD) or (AISC SEISMIC F3.5b(2))

$V_a \le V_n / \Omega_v$ (ASD), (AISC SEISMIC F3.5b(2))

where,

$V_n = \min(V_P,\ 2M_P / e)$ (AISC SEISMIC F3.5b(2))

$$V_p = \begin{cases} 0.6F_y A_{lw} & \text{for } P_r/P_c \le 0.15 \\ 0.6F_y A_{lw}\sqrt{1-\left(\dfrac{P_r}{P_c}\right)^2} & \text{for } P_r/P_c > 0.15 \end{cases}$$ (AISC SESIMIC F3-2, F3-3)

$$M_p = \begin{cases} F_{yZ} & \text{for } P_r/P_c \le 0.15 \\ F_{yZ}\left(\dfrac{1-P_r/P_c}{0.85}\right) & \text{for } P_r/P_c > 0.15 \end{cases}$$ (AISC SEISMIC F3-8, F3-9)

$$A_{lw} = \begin{cases} \left(d-2t_f\right)t_w & \text{for I-Shapes} \\ 2\left(d-2t_f\right)t_w & \text{for Boxes} \end{cases}$$ (AISC SEISMIC F3-4, F3-5)

$\phi = \phi_v$ (default is 0.9) (AISC SEISMIC F3-5b(2))

$\Omega = \Omega_v$ (default is 1.67) (AISC SEISMIC F3-5b(2))

$P_r = P_u$ (LRFD) (AISC SEISMIC F3-5b(2))

$P_r = P_a$ (ASD) (AISC SEISMIC F3-5b(2))

$P_c = P_y$ (LRFD) (AISC SEISMIC F3-5b(2))

$P_c = P_y / 1.5$ (ASD) (AISC SEISMIC F3-5b(2))

$P_y = A_g F_y$. (AISC SEISMIC F3-6)

- If $P_r/P_c > 0.15$, the link beam length, e, is checked not to exceed the following:

– if $\rho' \leq 0.5$

$$e \leq \left[1.6\frac{M_p}{V_p}\right] \quad \text{(AISC SEISMIC F3-10)}$$

– if $\rho' > 0.5$

$$e \leq \left[1.15 - 0.3\rho'\right]\left[1.6\frac{M_p}{V_p}\right] \quad \text{(AISC SEISMIC F3-11)}$$

where,

$$\rho' = \frac{P_r / P_c}{V_r / V_c} \quad \text{(AISC SEISMIC F3-5b(3))}$$

$$V_r = V_u \text{ (LRFD)} \quad \text{(AISC SEISMIC F3-5b(3))}$$

$$V_r = V_a \text{ (ASD)} \quad \text{(AISC SEISMIC F3-5b(3))}$$

$$V_c = V_y \text{ (LRFD)} \quad \text{(AISC SEISMIC F3-5b(3))}$$

$$V_c = V_y / 1.5 \text{ (ASD)} \quad \text{(AISC SEISMIC F3-5b(3))}$$

$$V_y = 0.6 F_y A_{dw} \quad \text{(AISC SEISMIC F3-5b(3))}$$

If the check is not satisfied, the program reports an error message.

- The link beam rotation, θ, of the individual bay relative to the rest of the beam is calculated as the story drift Δ times bay length (L) divided by the total lengths of link beams (e) in the bay.

  The link rotation, θ, is checked as follows (AISC SEISMIC F3.4a):

  $$\theta = \frac{\Delta L}{e}$$

  – $\theta \leq 0.08$ radian, where link beam clear length, $e \leq 1.6 M_p / V_p$

– $\theta \le 0.02$ radian, where link beam clear length, $e \ge 2.6 M_p / V_p$

– $\theta \le$ value interpolated between 0.08 and 0.02 as the link beam clear length varies from $1.6 M_p / V_p$ to $2.6 M_p / V_p$.

The story drift is calculated as

$$\Delta = \frac{\Delta_s C_d}{I}, \qquad \text{(ASCE 12.8-15)}$$

where C_d is a System Deflection Amplification Factor and I is the system Importance Factor.

- The beam strength outside the link is checked to be at least the beam force corresponding to the amplified controlling link beam shear strength, 1.25 $R_y V_n$ for I-Shapes, and 1.4 $R_y V_n$ for Box shapes (AISC SEISMIC F3.3). The controlling link beam nominal shear strength is taken as follows:

  $$V_n = \min(V_p, 2M_p / e). \qquad \text{(AISC SEISMIC F3.5b(2))}$$

  The values of V_p and M_p are calculated following the procedure described previously (AISC SEISMIC F3.5b(2)). 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.

  For load combinations including seismic effects, a load Q_1 is substituted for the term E, where Q_1 is defined as the axial forces and moments generated by at least 1.25 $R_y V_n$ for I-Shapes and 1.4 $R_y V_n$ for Box shapes, where V_n is the nominal shear strength of the link beam (ASIC SEISMIC F3.3).

- All braces are checked to be Seismically Compact for moderately ductile frames in accordance with Table 4-1 (AISC SEISMIC F3.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under the “Classification of Sections for Local Buckling” in this chapter. If these criteria are not satisfied, the program issues an error message.

- The brace strength is checked the brace forces corresponding to the amplified controlling link beam nominal shear strength 1.25R_yV_n for I-Shapes and 1.4R_yV_n for Box shapes (AISC SEISMIC F3.3). The controlling link beam nominal shear strengths are obtained by the process described earlier (AISC SEISMIC F3.5b(1)).

  For load combinations including seismic effects, a load Q_1 is substituted for the term E, where Q_1 is defined as axial forces and moments generated by at least 1.25 R_yV_n for I-Shapes and 1.4 R_yV_n for Box shapes, where V_n is the nominal shear strength of the link beam (ASIC SEISMIC 15.6a).

- All column members are checked to be Seismically Compact for highly ductile frames in accordance with Table 4-1 (AISC SEISMICF3.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under the "Classification of Sections for Local Buckling" in this chapter. If these criteria are not satisfied, the program issues an error message.

- The column strength is checked for the column forces corresponding to the controlling link beam nominal shear strength, 1.25 R_yV_n for I-Shapes and 1.4 R_yV_n for Box shapes, where V_n is the nominal shear strength of the link beam (AISC SEISMIC F3.3). The controlling link beam nominal shear strength and the corresponding forces are obtained by the process described previously.

Note: 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.

4.9.9 Buckling Restrained Braced Frames (BRBF)

For this framing system, the following additional requirements are checked or reported (AISC SEISMIC F4).

- In columns, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load com-*

binations (F4.3) as described previously in the "Special Check for Column" section of this manual (AISC SEISMIC B2, D1.4a(2)).

- All column members are checked to be Seismically Compact for highly ductile members in accordance with Table 4-1 (AISC SEISMIC F2.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under "Classification of Sections for Local Buckling" in this chapter. If these criteria are not satisfied, the program issues an error message.

- All beam members are checked to be Seismically Compact for moderately ductile members in accordance with Table 4-1 (AISC SEISMIC F2.5a, D1.1, Table D1.1). If these criteria are satisfied, the section is reported as SEISMIC as described earlier under "Classification of Sections for Local Buckling" in this chapter. If these criteria are not satisfied, the program issues an error message.

- The program checks the laterally unsupported length of beams not to exceed the limiting value $0.17\left(E/F_y\right)r_y$ (AISC SEISMIC F4.4a(2), D1.2a). If this criterion is not satisfied, the program issues an error message.

The columns and beams are designed for load combinations, including the automatically generated special seismic combinations involving amplified seismic load. These combinations are the same as described in Section 4.8 "Special Check for Column Strength."

- The beam to column connection is designed to resist the moment equal to the lesser of the following:

  (i) A moment corresponding to the expected beam flexural strengths multiplied by 1.1 (LRFD) or by 1.1/1.5 (ASD), as appropriate.

$$M_b = 1.1 R_y M_p \,(LRFD) \text{ or } M_b = \frac{1.1 R_y M_p}{1.5} (ASD)$$

  (ii) A moment corresponding to the sum of expected flexural strengths multiplied by 1.1 (LRFD) or by 1.1/1.5 (ASD), as appropriate.

$$M_c = 1.1 R_y F_y Z \,(LRFD) \text{ or } M_c = \frac{1.1 F_y Z}{1.5} (ASD)$$

4.9.10 Special Plate Shear Walls (SPSW)

No special consideration for this type of framing system is given by the program. The user is required to check the seismic design requirements for SPSW independently.

4.10 Joint Design

When using the SEISMIC design code, the structural joints are checked and designed for the following.

- Check the requirement of continuity plate and determination of its area
- Check the requirement of doubler plate and determination of its thickness
- Check the ratio of beam flexural strength to column flexural strength
- Report the beam connection shear
- Report the brace connection force

4.10.1 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.

- The steel beam frames in a direction parallel to the column major direction, i.e., the beam frames into the column flange.
- The steel beam frames in a direction parallel to the column minor direction, i.e., the beam frames into the column web.
- 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 proper beam-column moment connection strength, continuity plates such as shown in Figure 2-3 of Chapter 2 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 connection conditions described by the first bullet, 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 (OMF, IMF, SMF, BRBF) only. No check is made for braced frames (OCBE, SCBF, EBF).

The program first evaluates the need for continuity plates. When the required strength P_{bf} exceeds the available strength ϕR_n (LRFD) or R_n/Ω (ASD), as appropriate, a continuity plate will be required. The program checks the following limit states.

- The column flange design strength, ϕR_n, and the allowable strength, R_n/Ω, for the limit state of flange local bending is given as follows:

  $R_n = 6.25 t_{fc}^2 F_{yc}$ if not at top story (AISC J10-1)

  $R_n = (0.5) 6.25 t_{fc}^2 F_{yc}$ if at top story (AISC J10-1, J10.1)

  where

  $\phi = 0.9$ (LRFD)

  $\Omega = 1.67$ (ASD)

- The available strength of the column web against local yielding at the toe of the fillet is given as follows:

  $R_n = (5.0 k_c + t_{fb}) F_{yc} t_{wc}$ if not at top story (AISC J10-2)

  $R_n = (2.5 k_c + t_{fb}) F_{yc} t_{wc}$ if at top story (AISC J10-3)

  where

  $\phi = 1.0$ (LRFD)

$\Omega = 1.5$ (ASD)

- The available strength of the column web against crippling is given as follows:

$$R_n = 0.80\, t_{wc}^2 \left[1 + 3\left(\frac{t_{fb}}{d_c}\right)\left(\frac{t_{tw}}{t_{fc}}\right)^{1.5} \right] \sqrt{\frac{EF_{yc} t_{fc}}{t_{wc}}}, \text{ if not at top story}$$

(AISC J10-4)

$$R_n = 0.40\, t_{wc}^2 \left[1 + 3\left(\frac{t_{fb}}{d_c}\right)\left(\frac{t_{tw}}{t_{fc}}\right)^{1.5} \right] \sqrt{\frac{EF_{yc} t_{fc}}{t_{wc}}}, \quad \text{if at top story and } t_{fb}/d_c \le 0.2$$

(AISC J10-5a)

$$R_n = 0.40\, t_{wc}^2 \left[1 + \left(4\frac{t_{fb}}{d} - 0.2\right)\left(\frac{t_{tw}}{t_{fc}}\right)^{1.5} \right] \sqrt{\frac{EF_{yw} t_{fc}}{t_{wc}}}, \quad \text{if at the top story and } t_{fb}/d_c > 0.2,$$

(AISC J10-5b)

where

$\phi = 0.75$ (LRFD)

$\Omega = 2.0$ (ASD)

- The available compressive strength of the column web against local buckling is given as follows:

$$R_n = \frac{24\, t_{wc}^3 \sqrt{EF_{yc}}}{d_c}, \quad \text{if not at the top story} \qquad \text{(AISC J10-8)}$$

$$R_n = \frac{12\, t_{wc}^3 \sqrt{EF_{yc}}}{d_c}, \quad \text{if at the top story.} \qquad \text{(AISC J10-8, J10.5)}$$

If any of the preceding conditions are not met, the program calculates the required continuity plate area as follows.

For LRFD:

$$A_{cp} = \frac{\left(P_{bf} - \phi R_n\right)}{\phi_c F_{cr}}, \qquad \text{(AISC J10, J10.8, E3)}$$

For ASD:

$$A_{cp} = \frac{\left(P_{bf} - R_n/\Omega\right)}{F_{cr}/\Omega_c}, \qquad \text{(AISC J10, J10.8, E3)}$$

In the preceding expressions, ϕ_c, Ω_c, and F_{cr} are taken as follows:

ϕ_c = 0.90 (LFRD) (AISC E1)

Ω_c, = 1.67 (ASD) (AISC E1)

F_{cr} = Flexural buckling stress of equivalent column related to the beam-column joint

The flexural buckling stress, F_{cr}, is determined as follows:

$$F_{cr} = \begin{cases} \left(0.658^{\frac{F_y}{F_e}}\right) F_y, & \text{if } \dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}, \\ 0.877 F_e, & \text{if } \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}, \end{cases} \qquad \text{(AISC J10.8, E3)}$$

where F_e is the elastic critical buckling stress given by

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2}. \qquad \text{(AISC E3-4)}$$

The member properties of the equivalent column are taken as follows:

- The cross-section is comprised of two stiffeners and a strip of the web having a width of $25t_{wc}$ at the interior stiffener and $12t_{wc}$ at the ends of the columns (AISC J10.8).

- The effective length is taken as $0.75h$, i.e., $K = 0.75$ and $L = h = d_c - 2t_{fc}$ (AISC J10.8).
- $\frac{KL}{r}$ is calculated based on the equivalent cross-section and equivalent length stated here.

In addition to satisfying the preceding limit states, it is made sure that the equivalent section, consisting of the stiffeners and part of the web plate, is able to resist the compressive concentrated force (AISC J8). This is similar to a column capacity check. For this condition, the program calculates the required continuity plate area as follows:

For LRFD:

$$A_{cp} = \frac{P_{bf}}{\phi_c F_{cr}} - 25t_{wc}^2, \quad \text{if not at top story} \qquad \text{(AISC J10.8, E8)}$$

$$A_{cp} = \frac{P_{bf}}{\phi_c F_{cr}} - 12t_{wc}^2, \quad \text{if at top story} \qquad \text{(AISC J10.8, E3)}$$

For ASD:

$$A_{cp} = \frac{P_{bf}}{F_{cr}/\Omega_c} - 25t_{wc}^2, \quad \text{if not at top story} \qquad \text{(AISC J10.8, E3)}$$

$$A_{cp} = \frac{P_{bf}}{F_{cr}/\Omega_c} - 12t_{wc}^2, \quad \text{if at top story} \qquad \text{(AISC J10.8, E3)}$$

An iterative process is involved as A_{cp}, r, and F_{cr} are interdependent. If A_{cp} is needed, iteration starts with the minimum thickness and minimum width of the continuity plate. A maximum of three iterations is performed.

If $A_{cp} \leq 0$, no continuity plates are required. If continuity plates are required, they must satisfy a minimum area specification defined as follows:

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

$$t_{cp}^{\min} = 0.5t_{fb} \geq b_{fb}/16 \qquad \text{(AISC J10.8)}$$

  If the maximum thickness is more than the upper limit, the program reports an error. Here it is assumed that the continuity plate can extend for the full width of the column flange.

- 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

$$b_{cp}^{\min} = 2\left(\frac{b_{fb}}{3} - \frac{t_{wc}}{2}\right) \qquad \text{(AISC J10.8)}$$

- So that the minimum area is given by

$$A_{cp}^{\min} = t_{cp}^{\min} b_{cp}^{\min}. \qquad \text{(AISC J10.8)}$$

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

In the preceding equations,

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 $M_u/(d_b - t_{tb})$

R_n = Nominal strength

t_{fb} = Beam flange thickness

t_{fc} = Column flange thickness

t_{wc} = Column web thickness

ϕ = Resistance factor (LRFD)

Ω = Safety factor (ASD)

The special seismic requirements additionally checked by the program are dependent on the type of framing used. Continuity plate requirements for seismic design are evaluated for moment frames (OMF, IMF, SMF, BRBF) only. No checks are performed for braced frames (OCBF, SCBF, and EBF).

- For OMF, the continuity plates are checked and designed for a beam flange force, P_{bf} ,

$$P_{bf} = 1.1 R_y F_y b_{fb} t_{fb} \text{ (LRFD)} \qquad \text{(AISC SEISMIC E1.6b)}$$

$$P_{bf} = (1.1/1.5) R_y F_y b_{fb} t_{fb} \text{ (ASD)} \qquad \text{(AISC SEISMIC E1.6b)}$$

- For SMF, IMF and BRBF, the continuity plates are checked and designed for a beam flange force, given below,

$$P_{bf} = 1.1 R_y F_y b_{fb} t_{fb} \text{ (LRFD)} \qquad \text{(AISC SEISMIC E2.6f, E3.6f)}$$

$$P_{bf} = (1.1/1.5) R_y F_y b_{fb} t_{fb} \text{ (ASD)} \qquad \text{(AISC SEISMIC E2.6f, E3.6f)}$$

Note that the code insists on designing the continuity plate to match with tested connection (AISC SEISMIC E2.6f, E3.6f, ANSI/AISC 358).

4.10.2 Design of 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, due to 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 2-3 of Chapter 2. 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 (OMF, IMF, SMF, BRBF) only. No check is made for braced frames (OCBF, SCBF, EBF).

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

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

The available strength of the web panel zone for the limit state of shear yielding is determined as ϕR_n (LFRD) or R_n/Ω (ASD), as appropriate. Assuming that the effect of panel zone deformation on frame stability has not been considered in analysis, the nominal strength, R_n, is determined as follows:

$$R_n = 0.6 F_y d_c t_p, \qquad \text{for } P_r \leq 0.4 P_c \text{, and} \qquad \text{(LRFD J10-9)}$$

$$R_n = 0.6F_y d_c t_p \left[1.4 - \frac{P_r}{P_c}\right], \text{ for } P_r > 0.4P_c. \qquad \text{(LRFD J10-10)}$$

By using $V_p = \phi R_n$ (LRFD) or $V_p = R_n / \Omega$ (ASD), as appropriate, the required column panel zone thickness t_p is found as follows.

For LRFD:

$$t_p = \frac{V_p / \phi}{0.6F_y d_c}, \qquad \text{for } P_r \le 0.4P_c$$

$$t_p = \frac{V_p / \phi}{0.6F_y d_c \left(1.4 - \frac{P_r}{P_c}\right)}, \quad \text{for } P_r > 0.4P_c.$$

For ASD:

$$t_p = \frac{\Omega V_p}{0.6F_y d_c}, \qquad \text{for } P_r \le 0.4P_c$$

$$t_p = \frac{\Omega V_p}{0.6F_y d_c \left(1.4 - \frac{P_r}{P_c}\right)}, \quad \text{for } P_r > 0.4P_c$$

The extra thickness, or the required thickness of the doubler plate is given as follows:

$$t_{dp} = t_p - t_w,$$

where

F_y = Column and doubler plate yield stress

t_p = Required column panel zone thickness

t_{fn} = Flange thickness of n-th beam

t_{dp} = Required doubler plate thickness

t_{fc} = Column flange thickness

t_w = Column web thickness

V_p = Required panel zone shear capacity k_c

V_c = Column shear in column above

n_b = Number of beams connecting to column

d_n = Overall depth of n-th beam connecting to column

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

d_c = Overall depth of column

M_{bn} = Factored beam moment from corresponding loading combination

R_n = Nominal shear strength of panel

P_r = Required axial strength, P_u (LRFD) or P_a (ASD)

P_y = Column axial yield strength, $F_y A$

P_c = P_y (LRFD) or $0.6F_y$ (ASD)

ϕ = 0.9 (LRFD)

Ω = 1.67 (ASD)

The largest calculated value of t_{dp}, calculated for any of the load combinations based on the factored beam moments and factored column axial loads, is reported.

Doubler plate requirements for seismic design are evaluated for SMF and BRBF only. No further check/design is performed for other types of frames (AISC SEISMIC E2.6e).

- For SMF and BRBF, the panel zone doubler plate requirements that are reported will develop at least the beam moments equal to the expected plastic moment capacity of the beam $\left(R_y M_p\right)$, or beam moments due to specified load combinations involving seismic load (AISC SEISMIC 9.3a). For seismic design, V_p is calculated using the equation given previously, except that M_{pb} is taken as $R_y F_y Z_{33}$.

  The capacity of the panel zone in resisting this shear is taken as:

$$V_n = 0.60 F_y d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p}\right), \text{ for } P_u \le 0.75 P_y \qquad \text{(AISC J10-11)}$$

$$V_n = 0.60 F_y d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p}\right)\left(1.9 - 1.2 \frac{P_r}{P_c}\right), \text{ for } P_r > 0.75 P_c \quad \text{(AISC J10-12)}$$

  giving the required panel zone thickness as

$$t_p = \frac{V_p / \phi_v}{0.6 F_y d_c} - \frac{3 b_{cf} t_{cf}^2}{d_b d_c}, \quad \text{if } P_r \le 0.75 P_c$$

(AISC SEISMIC E3.6e(1), AISC J10-11)

$$t_p = \frac{V_p / \phi_v}{0.6 F_y d_c \left(1.9 - 1.2 \frac{P_r}{P_c}\right)} - \frac{3 b_{cf} t_{cf}^2}{d_b d_c}, \quad \text{if } P_r > 0.75 P_c.$$

(AISC SEISMIC E3.6e(1), AISC J10-12)

  In the preceding expression, the term V_p / ϕ_v is replaced by $\Omega_v V_p$ if the ASD provision is followed. The required doubler plate thickness is calculated as

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

  where,

  ϕ = 1.0 (LRFD) (AISC J10.6, SEISMIC E3.6e(1))

Ω = 1.50 (ASD) (AISC J10.6, SEISMIC E3.6e(1))

b_{cf} = width of column flange

t_{cf} = thickness of column flange

t_p = required column web thickness

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

All other terms were explained previously.

- For SMF and BRBF, the program checks the following panel zone column web thickness requirement:

$$t \geq \frac{\left(d_c - 2t_{fc}\right) + \left(d_b - 2t_{fb}\right)}{90}. \qquad \text{(AISC SEISMIC E3-7)}$$

Here, t is taken as $t_{wc} + t_{dp}$ when the doubler plate is plug welded to prevent local buckling. In such cases, t_{dp} is increased if necessary to meet this criterion. If the doubler plate is not plug welded to the web, t is taken as t_{wc} and also as t_{dp} for checking both the plates. If t_{wc} cannot satisfy the criteria, then a failure condition is declared. If t_{dp} does not satisfy this criterion, then its value is increased to meet the criteria. If the check is not satisfied, it is noted in the output.

4.10.3 Weak Beam Strong Column Measure

Only for Special Moment Frames (SMF) the code requires that the sum of column flexure strengths at a joint should be more than the sum of beam flexure strengths (AISC SEISMIC E2.4a). The column flexure strength should reflect the presence of axial force present in the column. The beam flexural strength should reflect potential increase in capacity for strain hardening. To facilitate the review of the strong column weak beam criterion, the program will report 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

$$R_{\text{maj}} = \frac{\sum_{n=1}^{n_b} M^*_{pbn} \cos\theta_n}{M^*_{pcax} + M^*_{pcbx}}. \qquad \text{(AISC SEISMIC E2.4a, E3-1)}$$

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

$$R_{\text{min}} = \frac{\sum_{n=1}^{n_b} M^*_{pbn} \sin\theta_n}{M^*_{pcay} + M^*_{pcby}}, \qquad \text{(AISC SEISMIC E2.4a, E3-1)}$$

where,

R_{maj} = Plastic moment capacity ratios, in the major directions of the column

R_{min} = Plastic moment capacity ratios, in the minor directions of the column

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

$$M^*_{pc} = Z_c\left(F_{yc} - \left|\frac{P_{uc}}{A_g}\right|\right) \text{ (LRFD)} \qquad \text{(AISC SEISMIC E3-2a)}$$

$$M^*_{pc} = Z_c\left[F_{yc} - 1.5\left|\frac{P_{ac}}{A_g}\right|\right] \text{ (ASD)} \qquad \text{(AISC SEISMIC E3-2b)}$$

The plastic moment capacities of the beams are amplified for potential increase in capacity for strain hardening as

$$M^*_{pb} = 1.1 R_y F_{yb} Z_b f_{mv} \text{ (LFRD)} \qquad \text{(AISC SEISMIC E3-3a)}$$

$$M^*_{pb} = 1.1 R_y F_{yb} Z_b f_{mv}, \text{ (ASD)} \qquad \text{(AISC SEISMIC E3-3b)}$$

where,

Z_b = Plastic modulus of beam

Z_c = Plastic modulus of column

F_{yb} = Yield stress of beam material

F_{yc} = Yield stress of column material

P_{uc} = Axial compression force in column for given load combination

A_g = Gross area of column

f_{mv} = The moment amplification factor. It is taken as the ratio of beam moment at the centerline of column to the moment of the column face. This factor takes care of the M_{uv} (LRFD) or $1.5M_{av}$ (ASD) of the code (AISC SEISMIC E2.4a). f_{mv} is taken as follows:

$$f_{mv} = 1 + \frac{d_c}{L_b} \text{ where,}$$

d_c = Depth of column section, and

L_b = clear span length of the beam.

For the preceding 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.

The preceding ratios are not checked for single story buildings or the top story of a multistory building (AISC SEISMIC E2.4a(a)(ii)).

4.10.4 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 special seismic requirements additionally checked by the program are dependent on the type of framing used and the Seismic Design Category.

- For SMF, the beam connection shear is taken as the maximum of those from regular load combinations and those required for the development of full plastic moment capacity of the beam. The connection shear for the development of the full plastic moment capacity of beam is as follows:

  $$V_u = \frac{CM_{pb}}{L_h} + V_{DL} + V_{LL} \text{ (LRFD),} \qquad \text{(AISC SEISMIC E3.6d, ASCE 2.3.2-5)}$$

  $$V_a = (0.7)\frac{CM_{pb}}{L_n} + V_{DL} + V_{LL} \text{ (ASD),} \quad \text{(AISC SEISMIC E3.6d, ASCE 2.4.1-5)}$$

  where,

  V = Shear force corresponding to END I or END J or 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 beam = 1.1 $R_y M_p$

L_h = Clear length of the beam

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

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

- For IMF, the beam connection shear is taken as the maximum of the two values: (a) maximum shear from the load combinations and (b) maximum shear based on beam moment capacity of the beam (AISC SEISMIC E2.6d).

  The maximum shear from the load combinations is determined from all of the regular load combinations and also from the *amplified seismic load combinations* (AISC SEISMIC E2.6d). For LRFD or ASD provisions, appropriate load combinations are considered. The load combination for amplified seismic load combinations are also described previously in the "Design Loading Combinations" section.

  The maximum shear based on beam moment capacity is calculated for any load combination involving seismic load by replacing the seismic effect on shear force with the capacity shear as follows:

$$V_u = \frac{CM_{pb}}{L_h} + V_{DL} + V_{LL} \text{ (LFRD),} \qquad \text{(AISC SEISMIC E2.6d, ASCE 2.3.2-5)}$$

$$V_a = (0.7)\frac{CM_{pb}}{L_h} + V_{DL} + V_{LL} \text{ (ASD).} \quad \text{(AISC SEISMIC E2.6d, ASCE 2.4.1-5)}$$

  All parameters in the preceding equation have been described previously in this section.

- For OMF, the beam connection shear is taken as the maximum of the two values: (a) maximum shear from the load combinations and (b) maximum shear based on beam moment capacity of the beam (AISC SEISMIC E1.6b(a)).

The maximum shear from the load combinations is determined from all of the regular load combinations and also from the *amplified seismic load combinations* (AISC SEISMIC E1.6b(a), ASCE 12.4.2.3). For LRFD or ASD provisions, appropriate load combinations are considered. The load combination for amplified seismic load combinations also are described previously in the "Design Loading Combinations" section.

The maximum shear based on beam moment capacity is calculated for any load combination involving seismic load by replacing the seismic effect on shear force with the capacity shear as follows:

$$V_u = \frac{CM_{pb}}{L_h} + V_{DL} + V_{LL} \text{ (LFRD),} \qquad \text{(AISC SEISMIC E1.6b(a), ASCE 2.3.2-J)}$$

$$V_a = (0.7)\frac{CM_{pb}}{L_h} + V_{DL} + V_{LL} \text{ (ASD).} \qquad \text{(AISC SEISMIC E1.6b(a), ASCE 2.4.1-5)}$$

All parameters in the preceding equation have been described previously in this section. The moment connection is assumed to be FR.

- For SCBF the beam connection shear is taken as the maximum of those from regular load combination and those from *amplified seismic load combinations*.

- For OCBF and OBFI, the beam connection shear is taken as the maximum of those from regular load combinations and those from *amplified seismic load combinations*.

- For EBF, the beam connection shear is taken as the maximum of the two values: (a) maximum shear from the load combinations and (b) maximum shear based on link beam shear capacity (AISC SEISMIC 15.7, 11.2a(4)).

The maximum shear from the load combinations is determined from all of the regular load combinations and also from the *amplified seismic load combinations* (AISC SEISMIC 15.7, 11.2a(4), ASCE 14.2.3). For LRFD or ASD, appropriate load combinations are considered. The load combinations for amplified seismic load combinations also were described previously in the "Design Loading Combination" section of this chapter.

The maximum beam connection shear based on link beam shear capacity is taken as the beam connection shear that can be developed when the link beam yields in shear. The load factor for the seismic component of the load in the combination is calculated to achieve forces related to yielding of the link beam. For connection shear determination, the forces are further amplified by 1.1 R_y (AISC SEISMIC 15.7, 11.2a(4)).

If the beam-to-column connection is modeled with a pin in the program by releasing the beam end, it automatically affects the beam connection shear.

4.10.5 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 special seismic requirements additionally checked by the program are dependent on the type of framing used.

Bracer axial forces for seismic designs are evaluated for braced frames (SCBF, OCBF, EBF, BRBF) only. No special checks are performed for moment frames (OMF, IMF, SMF).

- For SCBF, the bracing connection force is taken as the minimum of the two values (AISC SEISMIC F2.6c):

  (a) The expected yield strength in tension of the bracing member, determined as $R_y F_y A_g$ (LFRD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate (AISC SEISMIC F2.6c(1)(a)).

  (b) The maximum load effect of the *amplified seismic load combination* (AISC SEISMIC F2.6c(1)(b)).

  Note that the required bracing connection force for the required compressive strength of the brace based on limit state of buckling that is equal to $1.1 R_y P_n$

(LFRD) or $(1.1/1.5)R_y P_n$ (ASD), as appropriate (AISC SEISMIC F2.6c(2)), is always less than the corresponding value considered in case (a). So this limit state is not considered.

- For OCBF or OCBFI, the bracing connection force is taken as the minimum of the two values (AISC SEISMIC F1.6a):

  (a) The expected yield strength in tension of the bracing member, determined as $R_y F_y A_g$ (LFRD) or $R_y F_y A_g/1.5$ (ASD), as appropriate (AISC SEISMIC F3.6b(a)).

  (b) The maximum load effect of the amplified seismic load combination (AISC SEISMIC F3.6b(b)(i)).

- For EBF, the required strength of the diagonal brace connection at both ends of the brace is taken as the maximum of the following two values: (a) the maximum connection force from the design load combinations, and (b) the maximum brace connection force based on the link beam shear capacity (AISC SEISMIC F3.6c).

  The maximum connection force from the load combinations is determined for all the regular load combinations. The amplified seismic load combinations are not considered.

  The maximum brace connection force based on link beam shear capacity is taken as the brace connection force that can be developed when the link beam yields in shear. The load factor for seismic component of the load in the combination is calculate to achieve forces related to yielding of the link beam. For connection force determination, the forces are further amplified by $1.25R_y$ for I-Shapes and $1.4R_y$ for Box links (AISC SEISMIC F3.3).

- For BRBF, the diagonal brace connections in tension and compression are computed as 1.1 times the adjusted brace strength in compression in accordance with Section F4.2a. The adjusted brace strength in compression is $P_{nc} = \beta \omega R_y P_{ysc}$ (AISC SEISMIC F4.2a) and the adjusted brace strength in tension is $P_{nt} = \omega R_y P_{ysc}$ (AISC SEISMIC F4.2a).

Appendix A
P-Delta Effects

Modern design provisions are based on the principle that the member forces are calculated by a second-order elastic analysis, where the equilibrium is satisfied on the deformed geometry of the structure. The effects of the loads acting on the deformed geometry of the structure are known as the second-order or the P-Delta effects.

The P-Delta effects come from two sources: global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure A-1, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by Δ. The local deformation of the member is shown as δ. The total second order P-Delta effects on this frame object are those caused by both Δ and δ.

The program has an option to consider P-Delta effects in the analysis. When you consider P-Delta effects in the analysis, the program does a good job of capturing the effect due to the Δ deformation (P-Δ effect) shown in Figure B-1, but it does not typically capture the effect of the δ deformation (P-δ effect), unless, in the model, the frame object is broken into multiple elements over its length.

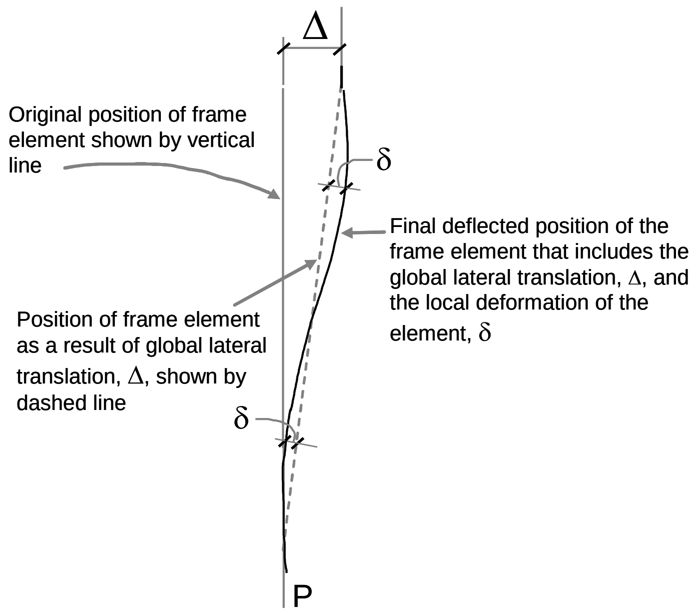


Figure A-1 *P-Δ* ***and*** *P-δ* ***effects***

In design codes, required strengths are usually required to be determined using a second-order analysis that considers both P-Δ and P-δ effects. Approximate second-order analysis procedures based on amplification of responses from first-order analysis for calculating the required flexural and axial strengths are common in current design codes and have the following general form:

$$M_{CAP} = B_1 M_{nt} + B_2 M_{lt} \quad \text{(AISC A-8-1)}$$

$$P_{CAP} = P_{nt} + B_2 P_{lt} \quad \text{(AISC A-8-2)}$$

where,

M_{CAP}, P_{CAP} = Required axial and flexural design capacities

M_{nt}, P_{nt} = Required axial and flexural capacities from first-order analysis of the member assuming there is no translation of the frame (i.e., associated with the δ deformation in Figure A-1)

M_{lt}, P_{lt} = Required axial and flexural capacities from first-order analysis of the member as a result of lateral translation of the frame only (i.e., associated with the Δ deformation in Figure A-1)

B_1 = Unitless amplification factor multiplying M_{nt}

B_2 = Unitless amplification factor multiplying M_{lt}, P_{lt}

Depending on the choice of analysis methods, which is provided by the ANSI/AISC 360-10 code, a rigorous second order analysis or the amplification of first order analysis results to estimate the effect of second order effects is required. The program has the capability of performing both. In the first case, the required strengths are determined directly from the analysis results without any amplification factors (i.e., B_1 and B_2 are equal to 1). However, these amplification factors can always be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

To properly capture the P-δ effect in a finite element analysis, each element, especially column elements, must be broken into multiple finite elements. Although a single element per member can capture the P-δ effect to some extent, the program considers that inadequate. For practical reasons, the software internally divides the column elements into two members. The user must provide additional subdivisions where a column is expected to have multiple inflection points.

In general, steel frame design requires consideration of P-Delta effects in the analysis before the check/design is performed. Although two elements per line object are generally adequate to capture the P-Δ effect, it is recommended to use more than two elements per line object for the cases where both P-Δ and P-δ effects are to be considered for a member having multiple points of inflection. However, explicit manual breaking of the member into elements has other consequences related to member end moments and unbraced segment end moment. It is recommended that the members be broken internally by the program. In this way, the member is recognized as one unit, end of the members are identified properly, and P-Δ and P-δ effects are captured better.

Appendix B
Steel Frame Design Preferences

The Steel Frame Design Preferences are basic assignments that apply to all of the steel frame members. Tables B-1, B-2, and B-3 list Steel Frame Design Preferences for "AISC 360-10." Default values are provided for all preference items. Thus, it is not necessary to specify or change any of the preferences. However, at least review the default values to ensure they are acceptable. Some of the preference items also are available as member specific overwrite items. The Overwrites are described in Appendix C. Overwritten values take precedence over the preferences.

Table B-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
Design Code	Design codes available in the current version		The selected design code. Subsequent design is based on this selected code.
Time History Design	Envelopes, Step-by-Step, Last Step, etc.	Envelopes	Toggle for design 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 combination has more than one time history case in it, that design combination is designed for the envelopes of the time histories, regardless of what is specified here.

Table B-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
Framing Type	SMF, IMF, OMF, SCBF, OCBF, OCBFI, EBF	SMF	This item is used for ductility considerations in the design, when seismic provisions are considered.
Seismic Design Category	A, B, C, D, E, F	D	This item varies with the Seismic Hazard Exposure Group and the Effective Peak Velocity Related Acceleration.
Design Provision	ASD, LRFD	LRFD	Application provisions for calculation of allowable/design and required strength of individual elements. Selection of ASD will enable additional fields for safety factor overwrites, whereas selection of LRFD will enable additional fields for resistance factors.
Analysis Method	7 Values	Direct Analysis Method with General 2nd Order Analysis and τ_b Fixed	Indicates the method used to check/design the steel members. The design module does not verify the acceptability of the selected design analysis method; the user is expected to verify acceptability. Also, the user is expected to set the appropriate stiffness modification factors and to supply the combinations with appropriate notional loads.
Notional Load Coefficient	> 0	0.002	The coefficient used to define the notional load as a fraction of a given gravity load pattern.
Ignore Seismic Code?	Yes, No	No	Toggle to consider (No) or not consider (Yes) the seismic part of the code in design.
Ignore Special Seismic Load?	Yes, No	No	Toggle to consider (No) or not consider (Yes) special seismic load combinations in design.
Is Doubler Plate Plug Welded?	Yes, No	Yes	Toggle to indicate if the doubler-plate is plug welded (Yes) or it is not plug welded (No).
HSS Welding Type	ERW, SAW	ERW	Indicates the procedure used for welding the steel members.
Reduce HSS Thickness?	Yes, No	Yes	Toggle to consider if the HS (Box or Pipe) thickness is reduced (Yes) or should not be reduced (No).
Consider Deflection?	Yes, No	No	Toggle to consider the deflection limit (Yes) or to not consider the deflection limit (No).
DL Limit, L/	≥ 0	120	Deflection limit for dead load. Inputting 120 means that the limit is L/120. Inputting zero means no check will be made of this item.

Table B-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
Super DL+LL Limit, L/	≥ 0	120	Deflection limit for superimposed dead plus live load. Inputting 120 means that the limit is L/120. Inputting zero means no check will be made of this item.
Live Load Limit, L/	≥ 0	360	Deflection limit for superimposed live load. Inputting 360 means that the limit is L/360. Inputting zero means no check will be made of this item.
Total Limit, L/	≥ 0	240	Deflection limit for total load. Inputting 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
Total-Camber Limit, L/	≥ 0	240	Limit for net deflection. Camber is subtracted from the total load deflection to get net deflection. Inputting 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
Pattern Live Load Factor	≤1.0	0.75	The live load factor for automatic generation of load combinations involving pattern live loads and dead loads.
D/C Ratio Limit	≤1.0	0.95	The demand/capacity ratio limit to be used for acceptability. D/C ratios that are less than or equal to this value are considered acceptable. The program will select members from the auto select list with D/C ratios less than or equal to this value.
Maximum Number of Auto Iteration	≥ 1	1	Sets the number of iterations of the analysis-design cycle that the program will complete automatically assuming that the frame members have been assigned auto select sections. This is currently only available in ETABS.

Table B-2 Additional Steel Frame Design Preferences LRFD Provision

Item	Possible Values	Default Value	Description
Phi (Bending)	≤1.0	0.9	Resistance factor for flexure.
Phi (Compression)	≤1.0	0.9	Resistance factor for compression.
Phi (Tension-Yielding)	≤1.0	0.9	Resistance factor for yielding in tension.
Phi (Tension-Fracture)	≤1.0	0.75	Resistance factor for tension rupture.
Phi (Shear)	≤1.0	0.9	Resistance factor for shear.
Phi (Shear, Short Webbed Rolled I-Shapes)	≤1.0	1.0	Resistance factor for shear for specific short-webbed rolled I-Shapes.
Phi (Torsion)	≤1.0	0.9	Resistance factor for torsion.

Table B-3 Additional Steel Frame Design Preferences for ASD Provision

Item	Possible Values	Default Value	Description
Omega (Bending)	≥ 1.0	1.67	Safety factor for flexure.
Omega (Compression)	≥ 1.0	1.67	Safety factor for compression.
Omega (Tension-Yielding)	≥ 1.0	1.67	Safety factor for yielding in tension.
Omega (Tension-Fracture)	≥ 1.0	2.00	Safety factor for tension rupture.
Omega (Shear)	≥ 1.0	1.67	Safety factor for shear.
Omega (Shear, Short Webbed Rolled I-Shape)	≥ 1.0	1.50	Safety factor for shear for specific short-webbed rolled I-Shapes.
Omega (Torsion)	≥ 1.0	1.67	Safety factor torsion.

Appendix C
Steel Frame Design Procedure Overwrites

The structural model may contain frame elements made of several structural materials: steel, concrete, aluminum, cold-formed steel and other materials. The program supports separate design procedures for each material type. By default the program determines the design procedure from the material of the frame member.

The software allows the user to turn off or turn on design of specific members by selecting *No Design* or *Default from material*. Refer to the program Help for information about overwriting the design procedure.

Overwrites

The steel frame design Overwrites are basic assignments that apply only to those elements to which they are assigned. Table C-1 lists Steel Frame Design Overwrites for “AISC 360-10.” Default values are provided for all overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values to ensure 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. overwritten values take precedence over the preferences (Appendix B).

Table C-1 Steel Frame Design Overwrites for "AISC 360-10"

Item	Possible Values	Default Value	Description
Current Design Section	Any defined steel section	Analysis section	The design section for the selected frame object. When this Overwrite is applied, any previous auto select section assigned to the frame object is removed.
Fame Type	SMF, IMF, OMF, SCBF, OCBF, OCBFI, EBF	From Preferences	This item is used for ductility considerations in the design.
Omega 0	≥ 1.0	Calculated	This factor is related to seismic force and ductility.
Consider Deflection?	Yes, No	From Preferences	Toggle to consider the deflection limit (Yes) or to not consider the deflection limit (No) in design.
Deflection Check Type	Ratio, Absolute, Both	Both	Choose to consider deflection limit as an absolute, as a divisor of the beam length, as both, or with no deflection limit.
DL Limit, L/	≥ 0	From Preferences	Deflection limit for dead load. Inputting 120 means that the limit is L/120. Inputting zero means no check will be made of this item.
Super DL+LL Limit, L/	≥ 0	From Preferences	Deflection limit for superimposed dead plus live load. Inputting 120 means that the limit is L/120. Inputting zero means no check will be made of this item.
Live Load Limit, L/	≥ 0	From Preferences	Deflection limit for superimposed live load. Inputting 360 means that the limit is L/360. Inputting zero means no check will be made of this item.
Total Limit, L/	≥ 0	From Preferences	Deflection limit for total load. Inputting 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
Total-Camber Limit, L/	≥ 0	From Preferences	Limit for net deflection. Camber is subtracted from the total load deflection to get net deflection. Inputting 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
DL Limit, abs	≥ 0	1.	Deflection limit for dead load. Inputting zero means no check will be made of this item.
Super DL+LL Limit, abs	≥ 0	1.	Deflection limit for superimposed dead plus live load. Inputting zero means no check will be made of this item.
Live Load Limit, abs	≥ 0	1.	Deflection limit for superimposed live load. Inputting zero means no check will be made of this item.
Total Limit, abs	≥ 0	1.	Deflection limit for total load. Inputting zero means no check will be made of this item.

Table C-1 Steel Frame Design Overwrites for "AISC 360-10"

Item	Possible Values	Default Value	Description
Total–Camber Limit, abs	≥ 0	1.	Deflection limit for net deflection. Camber is subtracted from the total load deflection to get net deflection. Inputting a value of 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
Specified Camber	≥ 0	0	The specified amount of camber to be reported in the design output and to be used in the net deflection check.
Live Load Reduction Factor	≥ 0	Calculated	The reducible live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying zero means the value is program determined.
Net Area to Total Area Ratio	≥ 0	1.0	The ratio of the net area at the end joint to gross cross-sectional area of the section. This ratio affects the design of axial tension members. Specifying zero means the value is the program default, which is 1.
Unbraced Length Ratio (Major)	≥ 0	Calculated	Unbraced length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Unbraced Length Ratio (Minor)	≥ 0	Calculated	Unbraced length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Unbraced Length Ratio (LTB)	≥ 0	L22	Unbraced length factor for lateral-torsional buckling for the frame object; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Effective Length Factor (K1 Major)	≥ 0	1.0	Effective length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the B_1 factor.

Table C-1 Steel Frame Design Overwrites for "AISC 360-10"

Item	Possible Values	Default Value	Description
Effective Length Factor (K1 Minor)	≥ 0	1.0	Effective length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the B_1 factor.
Effective Length Factor (K2 Major)	≥ 0	Calculated	Effective length factor for buckling about the frame object major axis assuming that the frame is braced at the joints against sidewsway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always 1, regardless of any other values specified in the Overwrites. The factor is used for axial compression capacity.
Effective Length Factor (K2 Minor)	≥ 0	Calculated	Effective length factor for buckling about the frame object minor axis assuming that the frame is braced at the joints against sidewsway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always 1, regardless of any other values specified in the Overwrites. The factor is used for axial compression capacity.
Effective Length Factor (K LTB)	≥ 0	K2 minor	Effective length factor for lateral-torsional buckling; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is taken as 1 by default. The values should be set by the user.
Moment Coefficient (Cm Major)	≥ 0	Calculated	Unitless factor; Cm for major axis bending is used in determining the interaction ratio. Inputting zero means the value is program determined.
Moment Coefficient (Cm Minor)	≥ 0	Calculated	Unitless factor; Cm for minor axis bending is used in determining the interaction ratio. Inputting zero means the value is program determined.
Bending Coefficient (Cb)	≥ 0	Calculated	Unitless factor; Cb is used in determining the interaction ratio. Inputting zero means the value is program determined.
NonSway Moment Factor (B1 Major)	≥ 0	Calculated	Unitless moment magnification factor for non-sway major axis bending moment. Specifying zero means the value is program determined.
NonSway Moment Factor (B1 Minor)	≥ 0	Calculated	Unitless moment magnification factor for non-sway minor axis bending moment. Specifying zero means the value is program determined.

Table C-1 Steel Frame Design Overwrites for "AISC 360-10"

Item	Possible Values	Default Value	Description
Sway Moment Factor (B2 Major)	≥ 0	1.0	Unitless moment magnification factor for sway major-axis bending moment. Specifying zero means the value is program determined. The program determined value is taken as 1 because it is assumed that P-Delta effects were specified to be included in the analysis, and thus no further magnification is required.
Sway Moment Factor (B2 Minor)	≥ 0	1.0	Unitless moment magnification factor for sway major-axis bending moment. Specifying zero means the value is program determined. The program determined value is taken as 1 because it is assumed that P-Delta effects were specified to be included in the analysis, and thus no further magnification is required.
HSS Welding Type	ERW, SAW	From Preferences	Indicates the procedure used to check/design the steel members.
Reduce HSS Thickness?	Yes, No	From Preferences	Toggle to consider if the HS (Box or Pipe) thickness is reduced (Yes) or should not be reduced (No).
Yield Stress, Fy	≥ 0	From Material	Material yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Overstrength Factor, Ry	≥ 0	From Material	The ratio of the expected yield strength to the minimum specified yield strength. This ratio is used in capacity-based design for special seismic cases. Specifying zero means the value is program determined.
Compressive Capacity, Pnc	≥ 0	Calculated	Nominal axial compressive capacity. Specifying zero means the value is program determined.
Tensile Capacity, Pnt	≥ 0	Calculated	Nominal axial tensile capacity. Specifying zero means the value is program determined.
Major Bending Capacity, Mn3	≥ 0	Calculated	Nominal bending moment capacity in major axis bending. Specifying zero means the value is program determined.
Minor Bending Capacity, Mn2	≥ 0	Calculated	Nominal bending moment capacity in minor axis bending. Specifying zero means the value is program determined.
Major Shear Capacity, Vn2	≥ 0	Calculated	Nominal shear capacity force for major direction shear. Specifying zero means the value is program determined.
Minor Shear Capacity, Vn3	≥ 0	Calculated	Nominal shear capacity force for minor direction shear. Specifying zero means the value is program determined.

Table C-1 Steel Frame Design Overwrites for "AISC 360-10"

Item	Possible Values	Default Value	Description
D/C Ratio Limit	≥ 0	Calculated	The demand/capacity ratio limit to be used for acceptability. D/C ratios that are less than or equal to this value are considered acceptable. Specifying zero means the value is program determined.

Appendix D
Interactive Steel Frame Design

The Interactive Steel Frame Design command is a powerful mode that allows the user to review the design results for any steel frame design and interactively revise the design assumptions and immediately review the revised results.

Note that a design must have been run for the interactive design mode to be available. With the design results displayed, right click on a frame object to display the Steel Stress Check Information form for the member. Click on the **Overwrites** button to display the Design Overwrites form, where the member section or other design parameters may be changed. Clicking **OK** on this form results in an immediate updating of the results displayed on the Steel Stress Check Information form.

Appendix E
Analysis Sections vs. Design Sections

Analysis sections are those section properties used to analyze the model when an analysis is run (refer to the program Help for information about running an analysis). The design section is whatever section is used in the steel frame design. It is possible for the last used analysis section and the current design section to be different. For example, an analysis may be run using a W18X35 beam, and then in the design, it may be found that a W16X31 beam worked. In that case, the last used analysis section is the W18X35 and the current design section is the W16X31. Before the design process is complete, verify that the last used analysis section and the current design section are the same. Refer to the program Help for information about completing this task.

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

- Assigning a frame section property assigns the section as both the analysis section and the design section.
- Running an analysis always sets the analysis section to be the same as the current design section.
- Assigning an Auto Select section list to a frame object initially sets the analysis and design section to be the section in the list with the median weight.

- Unlocking a model deletes the design results, but it does not delete or change the design section.
- Altering the Design Combinations in any way deletes the design results, but does not delete or change the design section.
- Altering any of the steel frame design preferences deletes the design results, but does not delete or change the design section.

Appendix F
Error and Warning Messages

This appendix provides all the possible error message that can be offered by the Steel Frame Design model of the program. Some of these messages are relevant to the "AISC 360-10" design code. Others are relevant to the other design codes supported by the program. However, the entire list is presented here for completeness.

Error Number	**Description**
1	kl/r > 200
2	l/r > 300
3	P > Pe
4	P > 0.85Py
5	P . 075Py
6	1/r > 720/sqrt(Fy)
7	1 > 96ry

Error Number	Description
8	h/tw > 380/sqrt(Fy)
9	Section is slender
10	Section is Class 4
11	Section is not plastic
12	Pu > Pe(B1 is undefined)
13	Cf > Ce (U1 is undefined)
14	bf/2tf > 52/sqrt(Fy)
15	1u > 76bf/sqrt(Fy)
16	fe > Fe
17	l/r > 180
18	l/r > 250
19	1/r > 120
20	l/r > 140
21	Pu > Phi * Pe (Delta b is undefined)
22	N/C (Not Calculated)
23	Internal Program Error
24	Section is too slender
25	1/r > 1000/sqrt (Fy)
26	Story drift too high
27	bf/2tf > 52/sqrt (Fy)

Error Number	Description
28	b/t > 100/Sqrt (Fy)
29	d/t > 1300/Fy
30	b/t > 52/sqrt (Fy)
31	Section is not compact
32	tz < (dz+wz)/90
33	b/t > 100/Sqrt (Fy)
34	1/r > 2500/Fy
35	Fy > 50 ksi
36	Link is too long
37	Section is not seismically compact
38	Phi factor less than or equal to zero
39	1o/f > 150*Sqrt (235/fy)
40	1o/f > 300*Sqrt (235/fy)
41	1o/f > 200*Sqrt (235/fy)
42	1o/f > 150*Sqrt (235/fy)
43	1o/f > 120*Sqrt (235/fy)
44	1o/f > 100*Sqrt (235/fy)
45	1o/f > 90*Sqrt (235/fy)
46	1o/f > 80*Sqrt (235/fy)
47	1o/f > 60*Sqrt (235/fy)

Error Number	Description
48	tz < (dz+wz)/70
49	1/r >60
50	1/r > 0.086*ry*E/Fy
51	kl/r > 4.23*Sqr(e/Ey
52	Link Rotaion is too high
53	Phib factor less than or equal to zero
54	Lambda_y > 120*sqrt(235/fy): Phib is no longer correct
55	0.8 N/Ney > .0 : Column is unstable
56	Beam/Column capacity ratio exceeds limit
57	Capacity ratio exceeds limit
58	Section is seismically slender &
59	tz < (dz+wz)/90
60	Beam/column capacity ratio exceed limit
61	Section is slender
62	Section is unknown
63	Section is not supported for design
64	Section is too slender – Effective area negative
65	Section is too slender – Effective Moment of inertia negative
66	Section is too slender – D/t high
67	kl/r > 150

Error Number	Description
68	kl/r > 250
69	l/r > 500
70	Unequal legs
71	Nue * N > Ncr
72	1/r > 5.87*ry*E/Fy
73	kl/r > 180 (IS 3.7, Table 3.1)
74	kl/r > 250 (IS 3.7, Table 3.1)
75	l/r > 400 (IS 3.7, Table 3.1)
76	Signa_ac, cal > 0.6*fcc (Excessive amplification) (IS 7.1.1)

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