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Integrated Solution for
Structural Analysis and Design

Steel Frame Design Manual

CSA-S16-14

Steel Frame
Design Manual

CSA-S16-14

For SAP2000®

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Chapter 1
Introduction

The design/check of steel frames is seamlessly integrated within the program. 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 no additional user input is required if the design defaults are acceptable.

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.

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 when it is checking a user-specified shape or when checking 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 con-

tinuity 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 "CSA S16-14." This option covers the "CSA S16-14 — Design of steel structures" (CSA 2014). The implementation covers loading and load combinations from "NBCC 2010 Minimum Design Loads for Buildings and Other Structures" (NBCC 2010).

The design codes supported under "CSA S16-14" are written in Newton-millimeter units. All the associated equations and requirements have been implemented in the program in Newton-millimeter 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.

1.1 Organization

This manual is designed to help you quickly become productive using the CSA S16-14 steel frame design option. Chapter 2 addresses prerequisites related to modeling and analysis for a successful design in accordance with CSA S16-14. Chapter 3 provides detailed descriptions of the specific requirements as implemented in CSA S16-14. Chapter 4 provides detailed descriptions of the specific requirements for seismic loading as required by the specification CSA S16-14.

1.2   Recommended Reading/Practice

It is strongly recommended that you read this manual and review any applicable "Watch & Learn" Series^{TM} tutorials, which are found on our web site, http://www.csiamerica.com, before attempting to design a steel frame. Additional information can be found in the on-line Help facility available from within the program.

Chapter 2
Modeling, Analysis and Design Prerequisites

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 CSA S16-14 code is identified with the prefix "**CSA**."

- Reference to the NBCC 2010 code is identified with the prefix "**NBCC**."

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 of 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 (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 for checking each section in this list for adequacy 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 section 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. Refer to the program Help for more information about Auto Select section lists.

2.2   Analysis Sections vs. Design Sections

Analysis sections are those section properties used to analyze the model when the analysis is run. 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 more 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.

2.3   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 0.5 meter apart (2 feet if the model has been created in US units). The user can overwrite the number of stations in an object before the analysis is made using the **Assign** menu. The user can refine the design along the length of a member by requesting more stations.

2.4   Demand/Capacity Ratios

Determination of the controlling 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.

2.5   Design Load Combinations

The design load combinations are the various combinations of the prescribed analysis 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 analysis cases, each with its own scale factor. The scale factors are applied to the forces and moments from the analysis 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), 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 static load response cases declared as dead or live to be additive. However, each static 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 cases are added to load combinations involving gravity loads only.

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 should 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 combina-

tions with negative factors for response spectrum analysis 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.

2.6   Second Order 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 2-1, 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 2-1, 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.

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:
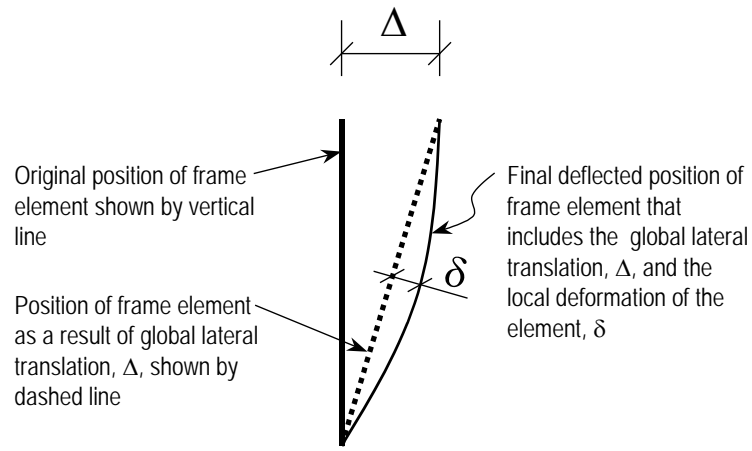
Figure 2-1 P-Δ and P-δ effects

$$M_{CAP} = U_1\left(M_{nt} + U_2 M_{lt}\right) \qquad\qquad \text{(CSA 8.4.2, 27.1.8.2)}$$

where,

M_{CAP} = Required flexural design capacities

M_{nt} = Required 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 2-1)

M_{lt} = Required 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 2-1)

U_1 = Unitless amplification factor multiplying M_{nt}

U_2 = Unitless amplification factor multiplying $\left(M_{nt} + U_2 M_{lt}\right)$

A rigorous second order analysis or the amplification of first order analysis results to estimate the effect of second order effects is required (CSA 8.4.2, 27.1.8.2). 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., U_1 and U_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 $P\text{-}\delta$ effect in a finite element analysis, each element, especially column elements, must be broken into multiple finite elements, which is not really desired for other reasons. Although a single element per member can capture the $P\text{-}\delta$ effect to some extent, the program considers that inadequate. The program thus uses the U_1 factor even if the analysis considers the $P\text{-}\Delta$ effects. This is a conservative approach.

Thus, in general, the steel frame design feature requires consideration of P-Delta effects in the analysis before the check/design is performed. Although one element per line object is generally adequate to capture the $P\text{-}\Delta$ effect, it is recommended to use more than one element per line object for the cases where both $P\text{-}\Delta$ and $P\text{-}\delta$ effects are to be considered. 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 $P\text{-}\Delta$ and $P\text{-}\delta$ effects are captured better.

2.7   Notional Load Cases

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 (CSA 8.4.1). These 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 case as a percentage of the previously defined gravity load case to be applied in one of the global lateral directions: X or Y. The user can define more than one notional load case associated with one gravity load by considering different factors and different directions.

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. Those ratios times the frame object lengths give the unbraced lengths for the member. Those ratios also can be overwritten by the user on a member-by-member basis, if desired, using the design overwrite option. The unsupported length for minor direction bending or for lateral-torsional buckling also can be defined more precisely by using precise bracing points in the Lateral Bracing option; refer to the program Help for more information about lateral bracing. If the unsupported length is defined using the precise bracing point definition and if it is also overwritten in the overwrites, the value used in the design overwrites prevails.

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} (also termed L_z), 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.
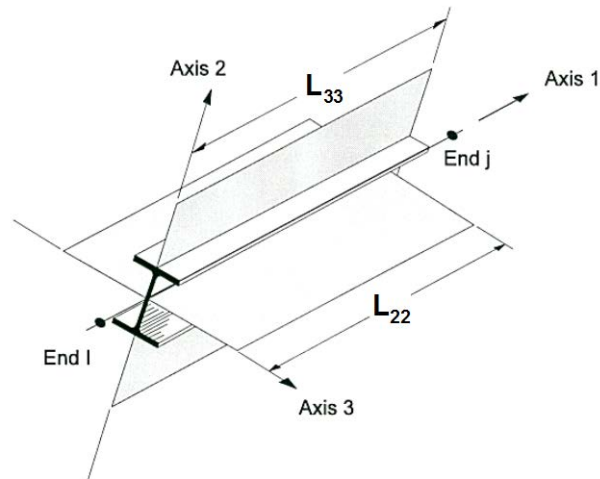
Figure 2-2 Unsupported lengths L₃₃ and L₂₂

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 that 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.

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 that 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 no intermediate brace is present 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 the 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 that factor 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. ω_l factor: The ω_l 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 (CSA 13.8.4). The program already calculates the ω_l 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 ω_l 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 ω_l factor manually for the critical combination and overwrite its value for that segment.

4. ω_2 factor: The logic is similar to that for the ω_l factor.

5. U_l factor: This factor amplifies the factored moments for the P-δ effect (CSA 13.8.4). In its expression, there are the ω_l 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 U_l factor would be correct. If

the axial force is small, the U_1 factor can be 1 and have no effect with respect to modeling the single segment or multi-segment element.

6. U_2 factor: The program does not calculate the U_2 factor. The program assumes that the user turns on the P-Δ feature. In such cases, U_2 can be taken as equal to 1 (CSA 8.4.2). That means that modeling with one or with 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 U_1 and U_2 will be picked up correctly. The factors ω_1 and ω_2 will be picked up correctly if there is no intermediate bracing point. The calculated ω_1 and ω_2 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-factor are correct, the effect of U_1 and U_2 will be picked up correctly. The factors ω_1 and ω_2 will be picked up correctly if the member is broken at the bracing points. The calculated ω_1 and ω_2 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, C_r, through an empirical column curve that accounts for geometric imperfections, distributed yielding, and residual stresses present in the cross-section.

The 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 C_r. The K-factor is always greater than 1 if the frame is a sway frame. The program calculates the K-factor automatically based on sway condition. The program also allows the user to overwrite K-factors on a member-by-member basis. The same K-factor is supposed to be used in calculation of the U_2 factor. However the program does not calcu-

late U_2 factors and relies on the overwritten values. If the frame is not really a sway frame, the user should overwrite the K-factors.

K has two values: one for major direction, K_{major}, and the other for minor direction, K_{minor}.

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

The rest of this section is dedicated to the determination of K-factors.

The K-factor algorithm has been developed for building-type structures, where the columns are vertical and the beams are horizontal, and the behavior is basically that of a moment-resisting 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:

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

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

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 $\left(EI/L \right)_x$ and $\left(EI/L \right)_y$ 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:

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

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

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-factor for the corresponding direction is calculated by solving the following relationship for α:

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

from which $K = \pi/\alpha$. 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 previously. 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 occasionally can 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*-factor 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.

2.11   Supported Framing Types

The code (CSA S16-14) recognizes the following types of framing systems.

Framing Type	References
Type D MRF (Ductile Moment-Resisting Frame)	CSA 27.2
Type MD MRF (Moderately Ductile Moment-Resisting Frame)	CSA 27.3
Type LD MRF (Limited-Ductility Moment-Resisting Frame)	CSA 27.4
Type MD CBF(TC) (Moderately Ductile Concentrically Braced Frame – Tension-Compression)	CSA 27.5.2.3
Type MD CBF(V) (Moderately Ductile Concentrically Braced Frame – Chevron)	CSA 27.5.2.4
Type MD CBF(TO) (Moderately Ductile Concentrically Braced Frame – Tension only)	CSA 27.5.2.5
Type MD CBF(OT) (Moderately Ductile Concentrically Braced Frame – Others)	CSA 27.5.3.2
Type LD CBF(TC) (Limited-Ductility Concentrically Braced Frame – Tension-Compression)	CSA 27.6.2.1
Type LD CBF(V) (Limited-Ductility Concentrically Braced Frame – Chevron)	CSA 27.6.2.2
Type LD CBF(TO) (Limited-Ductility Concentrically Braced Frame – Tension only)	CSA 27.6.2.3
Type LD CBF(OT) (Limited-Ductility Concentrically Braced Frame – Others)	CSA 27.6.3
EBF (Ductile Eccentrically Braced Frame)	CSA 27.7
Conventional MF	CSA 27.11
Conventional BF	CSA 27.11
Cantilever Column	CSA 27.11.2

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.

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.
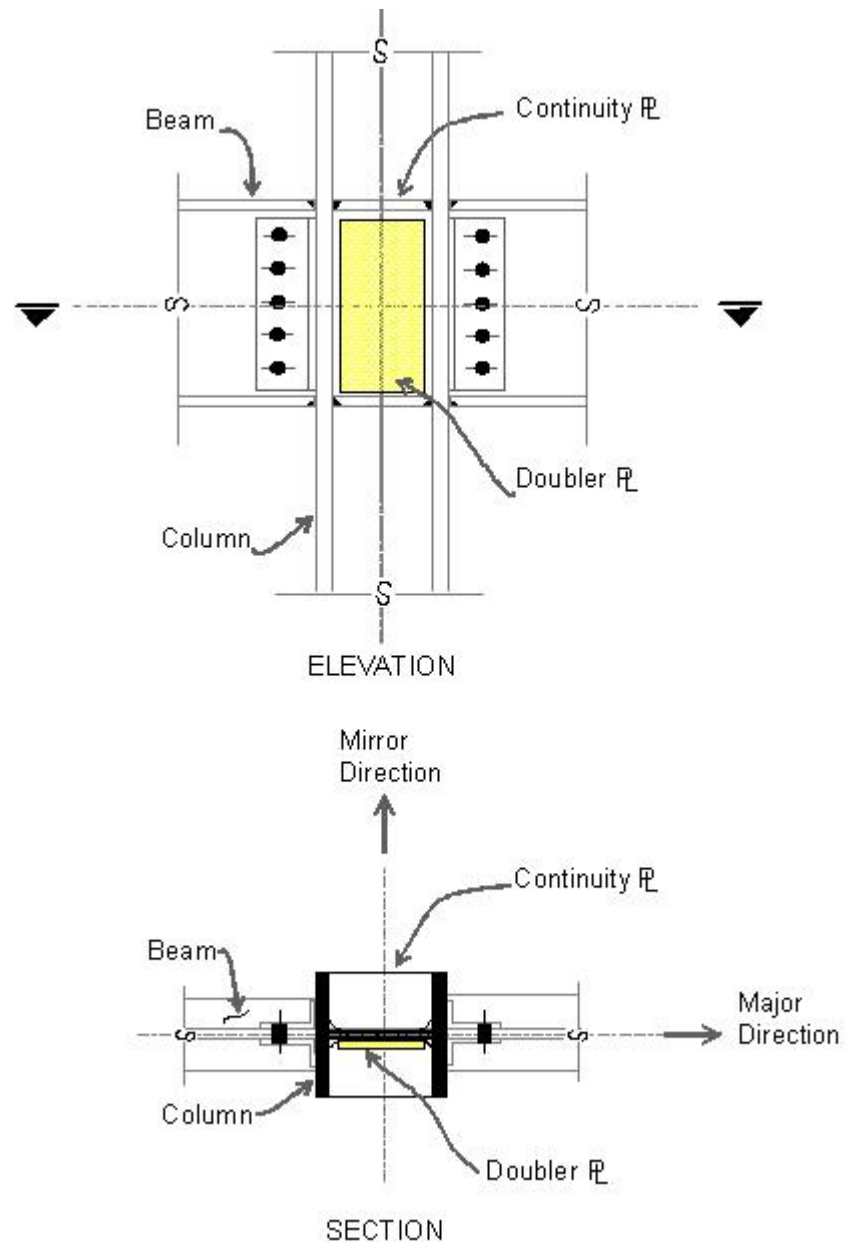
Figure 2-3 Doubler Plates and Continuity Plates

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   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 program allows the user to turn the design of specific members off and on by selecting *No Design* or *Default from material*. Refer to the program Help form more information about overwriting the design procedure.

ETABS supports both regular steel frame design and composite beam design. The determination of design procedure is different. If the material is concrete, the design procedure is concrete. If the material is steel, the default design pro-

cedure can be steel frame design or composite beam design. If the section is of steel material, and the member satisfies a host of other criteria, such as the member is horizontal (beam), it supports a filled deck or slab, it is an I-shaped member, it is hinged at both ends and so on, then the default design procedure is taken as composite beam design; otherwise, the default design procedure is taken as steel frame design. ETABS allows the user to overwrite a steel member frame design procedure to steel frame design, composite beam design, default, or no design. Refer to the program Help for more information about changing the design procedure. A change in design will be successful only if the design procedure is valid for that member, i.e., the program will not allow the user to change the design procedure for a steel frame object to concrete frame design.

2.15   Interactive Design

Interactive Design 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. Refer to the program Help for more information about interactive design.

2.16   Automated Iterative Design

If Auto Select sections have been assigned to frame objects, ETABS can automatically perform the iterative steel frame design process. To initiate the process in ETABS, first use the **Design menu > Steel Frame Design > View/Revise Preferences** command and set the maximum number of auto iterations to the maximum number of design iterations the program is to run automatically. Next, run the analysis. Then, begin the design, making sure that no objects are selected.

The program will then start the cycle of (1) performing the design, (2) comparing the last-used Analysis Sections with the Design Sections, (3) setting the Analysis Sections equal to the Design Sections, and (4) rerunning the analysis. This cycle will continue until one of the following conditions has been met:

▪ The Design Sections and the last-used Analysis Sections are the same.

▪ The number of iterations performed is equal to the number of iterations specified for the maximum number on the Preferences form.

Chapter 3
Design

This chapter provides a detailed description of the algorithms used by the programs in the design/check of structures in accordance with "CSA S16-14 — Design of Steel Structures" (CSA 2014). The implementation covers load combinations from "CSA S16-14," which is described in *Section 3.4 Design Loading Combinations* in this chapter. The loading based on "NBCC 2010" has been described in a separate document entitled "Automated Lateral Loads Manual" (CSI 20145.

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

- Reference to the CSA S16-14 code is identified with the prefix "**CSA**."

- Reference to the NBCC 2010 code is identified with the prefix "**NBCC**."

3.1   Notations

The various notations used in this manual are described herein.

A Cross-sectional area, mm^2

A_e Effective cross-sectional area for slender sections, mm^2

A_g	Gross cross-sectional area, mm^2
A_{v2}, A_{v3}	Major and minor shear areas, mm^2
A_w	Shear area, equal dt_w per web, mm^2
C_e	Euler buckling strength, N
C_f	Factored compressive axial load, N
C_r	Factored compressive axial strength, N
C_w	Warping constant, mm^6
C_y	Compressive axial load at yield stress, $A_g F_y$, N
D	Outside diameter of pipes, mm
E	Modulus of elasticity, N/mm^2
F_{cre}	Elastic critical plate-buckling stress in shear, N/mm^2
F_{cri}	Inelastic critical plate-buckling stress in shear, N/mm^2
F_s	Ultimate stress, N/mm^2
F_y	Specified minimum yield stress of material, N/mm^2
G	Shear modulus, N/mm^2
I_{22}	Minor moment of inertia, mm^4
I_{33}	Major moment of inertia, mm^4
J	St. Venant torsional constant for the section, mm^4
K	Effective length factor
K_{33}, K_{22}	Effective length K-factors in the major and minor directions
K_z	Effective length K-factor for lateral torsional buckling
L	Laterally unbraced length of member, mm
L_{22}, L_{33}	Laterally unbraced length of member for major and minor axes bending, mm

L_z	Unbraced length for lateral-torsional buckling, mm
M_{f33}, M_{f22}	Factored bending loads in major and minor directions, N-mm
M_{p33}, M_{p22}	Plastic moments in major and minor directions, N-mm
M_{r33}, M_{r22}	Factored moment resistance in major and minor directions, N-mm
M_u	Critical elastic moment, N-mm
M_{y33}, M_{y22}	Yield moments in major and minor directions, N-mm
R_d	Ductility-related force modification factor
R_o	Overstrength-related force modification factor
R_y	Factor applied to F_y to estimate the probable yield strength
$S_a(0.2)$	5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 0.2 sec
$S_a(1.0)$	5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 1.0 sec
S_{33}, S_{22}	Major and minor elastic section moduli, mm^3
T_f	Factored tensile axial load, N
T_r	Factored tensile resistance, N
U_1	Moment magnification factor to account for deformation of member between ends (P-δ effect)
U_2	Moment magnification factor (on sidesway moments) to account for P-Δ
V_{f2}, V_{f3}	Factored major and minor shear loads, N
V_{r2}, V_{r3}	Factored major and minor shear resistance, N
Z	Plastic modulus, mm^3
Z_{33}, Z_{22}	Major and minor plastic moduli, mm^3
b	Nominal dimension of plate in a section, mm 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, mm
b_{el}	Width of stiffened or unstiffened compression elements, mm
b_f	Flange width, mm
d	Overall depth of member, mm
h	Clear distance between flanges less fillets, taken as $(d - 2t_f)$, mm
k	Distance from outer face of flange to web toe of fillet, mm
r	Radius of gyration, mm
$r_{33},\ r_{22}$	Radii of gyration in the major and minor directions, mm
r_z	Least radian of gyration for analysis sections, mm
t	Thickness, mm
t_f	Flange thickness, mm
t_w	Thickness of web, mm
β	Coefficient used in an interaction equation
λ	Slenderness parameter
ϕ	Resistance factor (clause 3.1, 13.1)
ω_1	Moment coefficient
$\omega_{12},\ \omega_{13}$	Moment coefficient in major and minor directions
ω_2	Bending coefficient

3.2   Design Preferences

The steel frame design preferences are basic assignments that apply to all of the steel frame members. Table 3-1 lists steel frame design preferences for "CSA S16-14." 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 the next section. Overwritten values take precedence over the preferences. Refer to the program Help for information about changing Preferences.

Table 3-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
Design Code	Design codes available in the current version	AISC360-10/ IBC 2006	The selected design code. Subsequent design is based on this selected code.
Multi-Response Case Design	Envelopes, Step-by-Step, Last Step, Envelopes, All, Step-by-Step - All	Envelopes	Select to indicate how results for multivalued cases (Time history, Nonlinear static or Multi-step static) are considered in the design. - Envelope - considers enveloping values for Time History and Multi-step static and last step values for Nonlinear static. Step-by-Step - considers step by step values for Time History and Multi-step static and last step values for Nonlinear static. Last Step - considers last values for Time History, Multi-step static and Nonlinear static. Envelope - All - considers enveloping values for Time History, Multi-step static and Nonlinear static. Step-by-Step - All - considers step by step values for Time History, Multi-step static and Nonlinear static. Step-by-Step and Step-by-Step - All default to the corresponding Envelope if more then one multi-valued case is present in the combo.
Framing Type	Type LD MRF, Type MD MRF, Type D MRF, Type LD CBF(V), Type LD CBF(TC), Type LD CBF(TO), Type LD CBF(OT), Type MD CBF(V), Type MD CBF(TC), Type MD CBF(TO), Type MD CBF(OT), EBF, Cantilever, Column, Conventional MF, Conventional BF	Type LD MRF	This item is used for ductility considerations in the design.
Spectral Acceleration Ratio, Ie*Fa*Sa(0.2)	> 0	0.35	This is Sa(T=0.2sec) multiplied by Ie and Fa. It can assume different values in two orthogonal directions. The value specified here is solely used for design. The program uses the same value for all directions. See CSA S16-14 section 27.1.2, NBCC section 4.1.8.2, Table 4.1.8.9, Table 4.1.8.5, and Table 4.1.8.4B for details.

Table 3-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
Ductility Related Modification Factor, Rd	> 0	5	Overstrength-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behavior. This is a function of Seismic Force Resisting System. It can assume different values in two orthogonal directions. The program uses the same value for all directions. The value specified here is solely used for design. See CSA S16-14 section 27.1.2, NBCC sections 4.1.8.2 and 4.1.8.9, and NBCC Table 4.1.8.9 for details
Overstrength Related Modification Factor	> 0	1.5	This accounts for the dependable portion of reserve strength in a structure. This is a function of Seismic Force Resisting System. It can assume different values in two orthogonal directions. The program uses the same value for all directions. The value specified here is solely used for design. See CSA S16-14 section 27.1.2, NBCC sections 4.1.8.2 and 4.1.8.9, and NBCC Table 4.1.8.9 for details.
Phi (Bending)	≤1.0	0.9	Strength reduction factor.
Phi (Compression)	≤1.0	0.9	Strength reduction factor.
Phi (Tension)	≤1.0	0.9	Strength reduction factor.
Phi (Shear)	≤1.0	0.9	Strength reduction factor.
Slender Section Modification	Modify Geometry, Modify Fy	Modify Geometry	Select if the Class 4 sections should be handled by modifying the geometry or by modifying f_y.
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).
Consider Deflection?	Yes, No	Yes	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.
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.

Table 3-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
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.
Demand/Capacity 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.

3.3   Overwrites

The steel frame design Overwrites are basic assignments that apply only to those elements to which they are assigned. Table 3-2 lists steel frame design overwrites for "CSA S16-14." 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. Refer to the program Help for information about changing Overwrites.

Table 3-2 Steel Frame Design Overwrites

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.

Table 3-2 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Fame Type	Type LD MRF, Type MD MRF, Type D MRF, Type LD CBF(V), Type LD CBF(TC), Type LD CBF(TO), Type LD CBF(OT), Type MD CBF(V), Type MDCBF(TC), Type MD CBF(TO), Type MD CBF(OT), EBF, Cantilever, Column, Conventional MF, Conventional BF	From Preferences	This item is used for ductility considerations in the design.
Consider Deflection?	Yes/No	No	Toggle to consider (Yes) or not consider (No) deflection.
Deflection Check Type	Ratio, Absolute, Both	Program Determined	Choose to consider deflection limitations as absolute, as a divisor of the beam length (relative), as both.
DL Limit, L/	≥ 0	Program Determined	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	Program Determined	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	Program Determined	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	Program Determined	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	Program Determined	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	Program Determined	Deflection limit for dead load. Inputting zero means no check will be made of this item.
Super DL+LL Limit, abs	≥ 0	Program Determined	Deflection limit for superimposed dead plus live load. Inputting zero means no check will be made of this item.
Live Load Limit, abs	≥ 0	Program Determined	Deflection limit for superimposed live load. Inputting zero means no check will be made of this item.
Total Limit, abs	≥ 0	Program Determined	Deflection limit for total load. Inputting zero means no check will be made of this item.

Table 3-2 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Total–Camber Limit, abs	≥ 0	Program Determined	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	Program Determined	The specified amount of camber to be reported in the design output and to be used in the net deflection check.
Net Area to Total Area Ratio	≥ 0	Program Determined	The ratio of the net area at the design section to gross cross-sectional area of the section. This ratio affects the design of axial tension members. Specifying 0 means the value is program default which is 1.
Live Load Reduction Factor	≥ 0	Program Determined	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.
Unbraced Length Ratio (Major)	≥ 0	Program Determined	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, LTB)	≥ 0	Program Determined	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	Program Determined	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 (K Major)	≥ 0	Program Determined	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.
Effective Length Factor (K Minor)	≥ 0	Program Determined	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 U_1 factor.

Table 3-2 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Effective Length Factor (LTB)	≥ 0	Program Determined	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 (Omega1 Major)	≥ 0	Program Determined	Unitless factor; Omega1 for major axis bending is used in determining the interaction ratio. It captures the effect of non-uniform moment distribution along the length. Inputting zero means the value is program determined.
Moment Coefficient (Omega1 Minor)	≥ 0	Program Determined	Unitless factor; Omega1 for minor axis bending is used in determining the interaction ratio. It captures the effect of non-uniform moment distribution along the length. Inputting zero means the value is program determined.
Bending Coefficient (Omega2)	≥ 0	Program Determined	Unitless factor; Omega2 is used in determining the interaction ratio. It captures the effect of non-uniform moment distribution along the length. Inputting zero means the value is program determined.
NonSway Moment Factor (U1 Major)	≥ 0	Program Determined	Unitless moment magnification factor for non-sway major axis bending moment. Specifying zero means the value is program determined.
NonSway Moment Factor (U1 Minor)	≥ 0	Program Determined	Unitless moment magnification factor for non-sway minor axis bending moment. Specifying zero means the value is program determined.
Sway Moment Factor (U2 Major)	≥ 0	Program Determined	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 (U2 Minor)	≥ 0	Program Determined	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.

Table 3-2 Steel Frame Design Overwrites

Item	Possible Values	Default Value	Description
Parameter for Compressive Resistance, n	≥ 0	Program Determined	Parameter for Compression Resistance. Unitless factor used in the calculation of compression resistance. Its value should be picked up for the compression curve. Typical values are 2.24 (SSRC Curve 1), 1.34 (SSRC Curve 2), and 1.00 (SSRC Curve 3). Lower values are conservative. Its effect is minimal if lambda is close to zero or very large (lambda > 2.0). See CSA S16-14 Section 13.3.1. Specifying 0 means the value is program determined.
Yield Stress, Fy	≥ 0	Program Determined	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.
Expected to specified Fy Ratio, Ry	≥ 0	Program Determined	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 Resistance, Cr	≥ 0	Program Determined	Allowable axial compressive capacity. Specifying zero means the value is program determined.
Tensile Resistance, Tr	≥ 0	Program Determined	Allowable axial tensile capacity. Specifying zero means the value is program determined.
Major Bending Resistance, Mr3	≥ 0	Program Determined	Allowable bending moment capacity in major axis bending. Specifying zero means the value is program determined.
Minor Bending Resistance, Mr2	≥ 0	Program Determined	Allowable bending moment capacity in minor axis bending. Specifying zero means the value is program determined.
Major Shear Resistance, Vr2	≥ 0	Program Determined	Allowable shear capacity force for major direction shear. Specifying zero means the value is program determined.
Minor Shear Resistance, Vr3	≥ 0	Program Determined	Allowable shear capacity force for minor direction shear. Specifying zero means the value is program determined.
Demand/Capacity Ratio Limit	≥ 0	Program Determined	The stress ratio limit to be used for acceptability. Stress ratios that are less than or equal to this value are considered acceptable. Program determined value means it is taken from the steel preferences. Specifying 0 means the value is program determined.

3.4   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 analysis cases, such as dead load (DL), live load (LL), wind load (WL), and horizontal earthquake load (EL).

For the CSA S16-14 code, the following default design combinations are generated by the program (CSA 7.2.2, NBCC Table 4.1.3.2.A):

1.4 DL	(Case 1, NBCC Table 4.1.3.2.A)
1.25 DL + 1.5 LL	(Case 2, NBCC Table 4.1.3.2.A)
1.25 DL + 1.5 LL $\pm$ 0.4 WL	(Case 2, NBCC Table 4.1.3.2.A)
1.25 DL $\pm$ 1.4 WL	(Case 4, NBCC Table 4.1.3.2.A)
0.9 DL $\pm$ 1.4 WL	(Case 4, NBCC Table 4.1.3.2.A)
1.25 DL $\pm$ 1.4 WL + 0.5 LL	(Case 4, NBCC Table 4.1.3.2.A)
1.0 DL $\pm$ 1.0 EL	(Case 5, NBCC Table 4.1.3.2.A)
1.0 DL $\pm$ 1.0 EL + 0.5 LL	(Case 5, NBCC Table 4.1.3.2.A)

The code is 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 cases should be considered in the combinations with appropriate factors, appropriate directions, and appropriate senses. Notional loads are automatically included in the default design load combinations that do not include lateral loads. Notional load patterns are only added to a load combination when their corresponding gravity load patterns are included in that combination, using the corresponding scale factor. For further information, refer to the "Notional Load Cases" section in Chapter 2.

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.
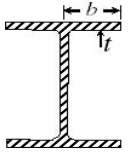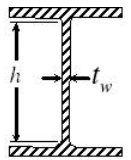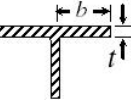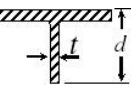
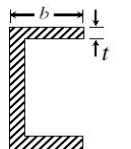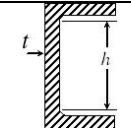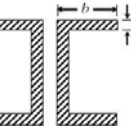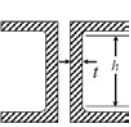# 3.5 Classification of Sections for Local Buckling

For the determination of the nominal strengths for axial compression and flexure, the sections are classified as Class 1 (Plastic), Class 2 (Compact), Class 3 (Noncompact), or Class 4 (Slender). The program classifies the individual sections in accordance with Table 3-3 (CSA 11.1, 11.2, 13.5.c, Table 1, Table 2). As specified in that table, a section is classified as Class 1, Class 2, Class 3 or Class 4 as applicable.

For elements supported along only one edge parallel to the direction of compression force, the width shall be taken as follows:
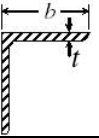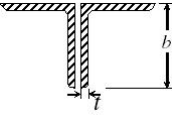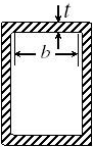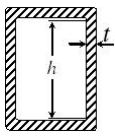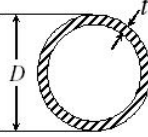
(a) For flanges of I-shaped members and tees, the width b_{el} is one-half the full nominal dimension, b_f.

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

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

(d) For webs of hot rolled sections, h is the clear distance between flanges.

Refer to Table 3-3 (CSA Table 1 and Table 2) for the graphic representation of element dimensions.

Table 3-3 Maximum Width-to-Thickness Ratios

Section Type	Description of Element	Example	Width-Thickness Ratio	Elements in Axial Compression	Limiting Width-to-Thickness Ratios for Elements in Flexural Compression			
					Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Non-compact)	Class 4 (Slender)
Doubly Symmetric I Shape	Flanges of rolled I Shapes		$b_{el}/2t$	$\leq \dfrac{200}{\sqrt{F_y}}$	$\leq \dfrac{145}{\sqrt{F_y}}$	$\leq \dfrac{170}{\sqrt{F_y}}$	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
	Web of rolled I Shapes		h/t_w	$\leq \dfrac{1100}{\sqrt{F_y}}(1-$	$\leq \dfrac{1700}{\sqrt{F_y}}(1-0.61$	$\leq \dfrac{1900}{\sqrt{F_y}}(1-0.6$	No limit	Flexure in web
T Shape	Flanges of T-sections		$b_{el}/2t$	$\leq \dfrac{200}{\sqrt{F_y}}$	$\leq \dfrac{145}{\sqrt{F_y}}$	$\leq \dfrac{170}{\sqrt{F_y}}$	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
	Stems of T-sections		d/t	$\leq \dfrac{340}{\sqrt{F_y}}$	---	---	$\leq \dfrac{340}{\sqrt{F_y}}$	Any
Channel	Flanges of Channels		b_{el}/t	$\leq \dfrac{200}{\sqrt{F_y}}$	$\leq \dfrac{145}{\sqrt{F_y}}$	$\leq \dfrac{170}{\sqrt{F_y}}$	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
	Webs of Channels		h/t_w	$\leq \dfrac{1100}{\sqrt{F_y}}(1-$	$\leq \dfrac{1700}{\sqrt{F_y}}(1-0.61$	$\leq \dfrac{1900}{\sqrt{F_y}}(1-0.6$	No limit	Flexure in web
Double Channel	Flanges of Double Channels		b_{el}/t	$\leq \dfrac{200}{\sqrt{F_y}}$	$\leq \dfrac{145}{\sqrt{F_y}}$	$\leq \dfrac{170}{\sqrt{F_y}}$	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
	Web of Double Channels		h/t_w	$\leq \dfrac{1100}{\sqrt{F_y}}(1-$	$\leq \dfrac{1700}{\sqrt{F_y}}(1-0.61$	$\leq \dfrac{1900}{\sqrt{F_y}}(1-0.6$	No limit	Flexure in web

Table 3-3 Maximum Width-to-Thickness Ratios

Section Type	Description of Element	Example	Width-Thickness Ratio	Elements in Axial Compression	Limiting Width-to-Thickness Ratios for Elements in Flexural Compression			
					Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Non-compact)	Class 4 (Slender)
Angle	Legs of Angles		b_{el}/t	$\leq \dfrac{200}{\sqrt{F_y}}$	---	---	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
Double Angle	Legs of double angles		b_{el}/t	$\leq \dfrac{200}{\sqrt{F_y}}$	---	---	$\leq \dfrac{200}{\sqrt{F_y}}$	≤ 60
Box	Flexural or axial compression of flanges under major axis bending		b_{el}/t	$\leq \dfrac{670}{\sqrt{F_y}}$	$\leq \dfrac{420}{\sqrt{F_y}}$ (rolled) $\leq \dfrac{525}{\sqrt{F_y}}$ (welded)	$\leq \dfrac{525}{\sqrt{F_y}}$	$\leq \dfrac{670}{\sqrt{F_y}}$	No limit
	Web under major axis bending		h/t_w	$\leq \dfrac{1100}{\sqrt{F_y}}(1$	$\leq \dfrac{1700}{\sqrt{F_y}}(1-0.61$	$\leq \dfrac{1900}{\sqrt{F_y}}(1-0.6$	No limit	Flexure in web
	Axial only compression		D/t	$\leq \dfrac{23,000}{F_y}$	$\leq \dfrac{13,000}{F_y}$	$\leq \dfrac{18,000}{F_y}$	$\leq \dfrac{66,000}{F_y}$	No limit
Round Bar	------	------	----	----	Assumed Class 2			
Rectangular	------	------	----	----	Assumed Class 2			
General	------	------	----	----	Assumed Class 3			
SD Section	------	------	----	----	Assumed Class 3			

For 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 dis-

tance from the centroid to the inside face of the compression flange less the fillet or corner radius.

(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.

(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.

(d) For flanges of rectangular hollow structural sections (HSS), the width b is the nominal outside dimension less four times the wall thickness. For webs of rectangular HSS, h is the nominal outside dimension less four times the wall thickness. The thickness, t, shall be taken as the design wall thickness.
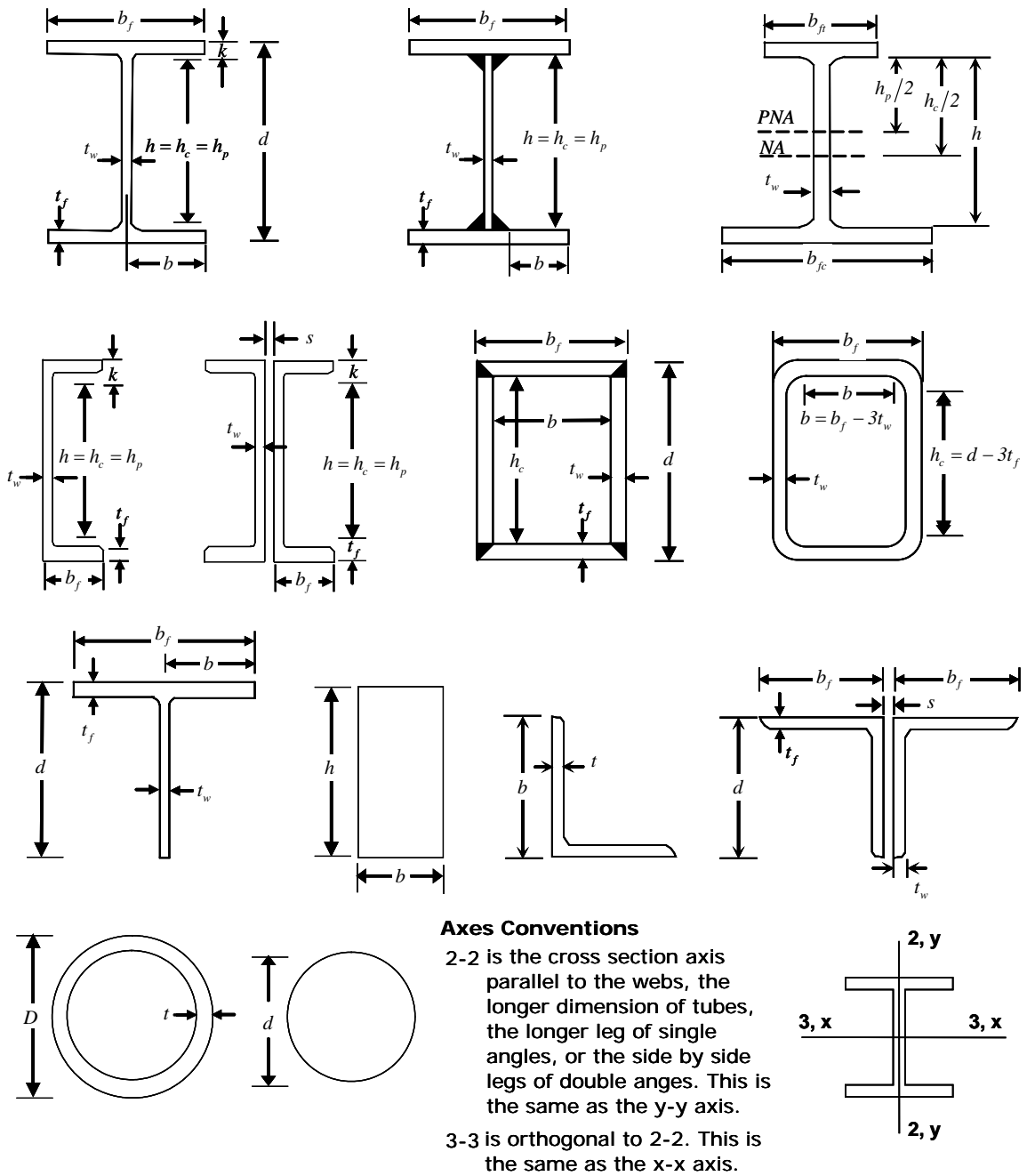
Figure 3-1 CSA S16-14 Definition of Geometric Properties

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. When slenderness parameters are larger than the limits for Class 4 sections, the section is classified as Too Slender. **Stress check of Too Slender sections is beyond the scope of this program.**

3.6 Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are C_f, M_{f33}, M_{f22}, V_{f2}, V_{f3} and T_f corresponding to factored values of the axial compression, the major and minor moments and shears, and axial tension, respectively. These factored loads are calculated at each of the previously defined stations.

3.7 Calculation of Factored Strengths

The nominal resistance in compression, tension, bending, and shear are computed for Plastic, Compact, Noncompact, and Slender members in accordance with the text in the following sections. The nominal flexural resistance for all shapes of sections are calculated based on their principal axes 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 Bending for flexure and on the geometric axes for shear.

3.7.1 Factored Tensile Strength

This section applies to the members subject to axial tension.

The factored axial tensile strength value, T_r, is taken as $\phi A_g F_y$ (CSA 13.2.(a)(i)). For members in tension, if L/r is greater than 300, a message is printed accordingly (CSA 10.4.2.2).

$$T_r = \phi A_g F_y. \qquad\qquad\qquad \text{(CSA 13.2(i))}$$

$$T_r = \phi A_e F_u. \qquad\qquad\qquad \text{(CSA 13.2(iii))}$$

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.

3.7.2 Factored Compressive Strength

Compression capacity is taken as the minimum of the two limit states: Flexural Buckling; and Torsional and Flexural-Torsional Buckling.

For the Flexural Buckling limit state, the factored axial compressive strength value, C_r, for Class 1, 2, or 3 sections depends on a factor, λ, which eventually depends on the slenderness ratio, KL/r, which is the larger of ($K_{33}L_{33}/r_{33}$) and ($K_{22}L_{22}/r_{22}$), and is defined as follows:

$$\lambda = \sqrt{\frac{F_y}{F_e}}, \qquad\qquad\qquad \text{(CSA 13.3.1)}$$

where:

$$F_e = \frac{\pi^2 E}{\left(\dfrac{kL}{r}\right)^2}, \qquad\qquad\qquad \text{(CSA 13.3.1)}$$

For single angles r_z is used in place of r_{33} and r_{22}. For members in compression, if KL/r is greater than 200, a message is printed (CSA 10.4.2).

Then the factored axial strength is evaluated as follows (CSA 13.3.1):

$$C_r = \phi A F_y (1 + \lambda^{2n})^{-\frac{1}{n}} \qquad \text{(CSA 13.3.1)}$$

where n is an exponent, and it takes three possible values to match the strengths related to three SSRC curves. The default n is 1.34, which is assigned to W-shapes rolled in Canada, fabricated Boxes and I-shapes, and cold-formed non-stress relieved (Class C) hollow structural sections (HSS) (CSA 13.3.1, CSA C13.3). The WWF sections produced in Canada from plates with flame-cut edges and hot-formed or cold-relieved (Class H) HSS are assigned to a favorable value of 2.24 (CSA 13.3.1, CSA C13.3). For heavy sections, a smaller value of n is considered appropriate (CSA C13.3). The program assumes the value of n as follows:

$$n = \begin{cases} 2.24, & \text{for WWF, HS, and HSS (Class H) sections,} \\ 1.34 & \text{for W, L, and 2L sections and normal HS and HSS sections,} \\ 1.34 & \text{for other sections with thickness less than 25.4 mm, except WWF sections,} \\ 1.00, & \text{for other sections with thickness larger than or equal to 25.4mm.} \end{cases}$$

The HSS sections in the current Canadian Section Database of the program are prefixed as HS instead of HSS. Also, to consider any HSS section as Class H, it is expected that the user would put a suffix to the HS or HSS section names.

The value of n can be overwritten in the steel design overwrites.

For the Torsional or Torsional-Flexural Buckling limit state, the factored compressive resistance, C_r, for Class 1, 2, or 3 sections is computed using the following equation, with a value of $n = 1.34$:

$$C_r = \phi A F_y (1 + \lambda^{2n})^{-\frac{1}{n}} \qquad \text{(CSA 13.3.1)}$$

where:

$$\lambda = \sqrt{\frac{F_y}{F_e}}, \qquad \text{(CSA 13.3.1)}$$

and the value of F_e is taken as:

a) For doubly symmetric and axisymmetric sections, the least of F_{ex}, F_{ey} and F_{ez};

b) For single symmetric sections, with the y axis taken as the axis of symmetry, the lesser of F_{ex} and F_{eyz},

where,

$$F_{ex} = \frac{\pi^2 E}{\left(\dfrac{K_x L_x}{r_x}\right)^2}.$$ (CSA 13.3.2)

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L_y}{r_y}\right)^2}.$$ (CSA 13.3.2)

$$F_{ez} = \left(\frac{\pi^2 E C_w}{\left(K_z L_z\right)^2} + GJ\right)\frac{1}{A\overline{r}_o^2}.$$ (CSA 13.3.2)

$$F_{eyz} = \frac{F_{ey} + F_{ez}}{2\Omega}\left[1 - \sqrt{1 - \frac{4 F_{ey} F_{ez}\Omega}{\left(F_{ey} + F_{ez}\right)^2}}\right].$$ (CSA 13.3.2)

L_z = Unbraced length factor for torsional buckling.

K_z = The effective length factor for torsional buckling, conservatively taken as 1.0. This can be overwritten in the design overwrites.

$$\overline{r}_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2 \text{, and}$$ (CSA 13.3.2)

$$\Omega = 1 - \left[\frac{x_o^2 + y_o^2}{\overline{r}_o^2}\right],$$ (CSA 13.3.2)

where x_o, y_o = the principal coordinates of the shear center to the centroid of the cross-section.

c) For angle sections, F_e is taken as the smallest root of:

$$\left(F_e - F_{ex}\right)\left(F_e - F_{ey}\right)\left(F_e - F_{ez}\right) - F_e^2\left(F_e - F_{ey}\right)\left(\frac{x_o}{\overline{r_o}}\right)^2 - F_e^2\left(F_e - F_{ex}\right)\left(\frac{y_o}{\overline{r_o}}\right)^2 = 0$$

(CSA 13.3.2.(c))

The factored compressive resistance, C_r, for Class 4 sections (CSA 11, Table 1, Table 2) is calculated using the preceding procedure as described for Class 1, 2, and 3 sections, with the exception that an effective area (CSA 13.3.5(a)) or an effective yield stress (CSA 13.3.5(b)) is used. The choice of using the effective area or effective yield stress can be made in the preferences. The effective area is calculated using a reduced element width that meets the maximum width-to-thickness ratio of a Class 3 section (CSA 13.3.5(a)). The effective yield stress is determined from the width-to-thickness ratio that meets the Class 3 limit (CSA 13.3.5(b)). The factored axial compressive resistance, C_r, is calculated using the requirements of Clause 13.3.1 or 13.3.2 as applicable. The slenderness ratios are calculated using gross-section properties.

The program uses the minimum of the factored axial compressive strength, C_r, computed from Cross-Sectional Strength, Overall Member Strength, and Torsional and Lateral-Torsional Buckling Strength (CSA 13.8.2, 13.8.3).

(a)   Cross-Sectional Strength: The axial compression capacity is based on the assumption that the number is fully braced in each direction ($L_{22} = 0$, $L_{33} = 0$), i.e., $\lambda = 0$. The axial compression capacity is based on the plastic limit state.

$$C_r = \phi A F_y \qquad \text{(CSA 13.8.2(a), 13.8.3(a), 13.3.1)}$$

(b)   Overall Member Strength: The axial compression capacity is based on the assumption that $K_{33} = 1$ and $K_{22} = 1$. For biaxial bending, C_r is taken as the minimum axial capacity for the flexural buckling limit state for $\dfrac{K_{22}L_{22}}{r_{22}}$ and $\dfrac{K_{33}L_{33}}{r_{33}}$. However for uniaxial strong axis bending, it is based on the flexural buckling limit state for $\dfrac{K_{22}L_{22}}{r_{22}}$ (CSA 13.8.2(b), 13.8.2(c), 13.3.1).

(c)   Lateral-Torsional Buckling Strength: The axial compression capacity is based on the minimum of capacities from the flexural buckling limit state

and the torsional and torsional-flexural buckling limit state (CSA 13.8.2(c), 13.8.3(c), 13.3.1, 13.3.2). For the flexural buckling limit state, the compression capacity is based on both the major and minor direction buckling using both $\dfrac{K_{22}L_{22}}{r_{22}}$ and $\dfrac{K_{33}L_{33}}{r_{33}}$ with the appropriate values for K_{22} and K_{33}. For the torsional and torsional-flexural buckling limit state, an additional slenderness parameter, $\dfrac{K_z L_z}{r_{min}}$, is used.

If the capacity C_r is overwritten, only the strength for the last case is modified, namely, the lateral torsional buckling strength. The capacity used for the cross-sectional strength and the overall member strength case remain unaffected by this overwrite.

3.7.3 Flexure Strength

The factored moment resistance in the major and minor directions is based on the geometric shape of the section, the section classification for compactness, and the unbraced length of the member. The bending strengths are evaluated in accordance with the code as follows (CSA 13.5, 13.6).

The program uses the minimum of the flexure strength, M_r, computed from Cross-Sectional Strength, Overall Member Strength, and Torsional and Lateral-Torsional Buckling Strength (CSA 13.8.2, 13.8.3).

Cross-Sectional Strength and Overall Member Strength: The M_{r33} and M_{r22} are calculated assuming that the member is laterally fully supported ($l_{22} = 0$ and $l_{33} = 0$) irrespective of its actual lateral bracing length (CSA 13.5, 13.8.2(a), 13.8.2(b), 13.8.3(a), 13.8.3(b)).

Lateral-Torsional Buckling Strength: M_{r33} and M_{r22} are calculated based on actual unbraced length (CSA 13.6, 13.8.2(c), 13.8.3(c)).

If the flexural capacities M_{r33} and M_{r22} are overwritten, only the strength for the last case is modified, namely, the lateral-torsional buckling strength. The capacities used for the cross-sectional strength and the overall member strength cases remain unaffected by the overwrites.

For Cross-Sectional Strength and Overall Member Strength cases, the moment capacities are considered to be as follows:

For Class 1 and 2 Sections:

$$M_r = \phi Z F_y = \phi M_p \qquad \text{(CSA 13.5(a))}$$

For Class 3 Sections:

$$M_r = \phi S F_y = \phi M_y \qquad \text{(CSA 13.5(b))}$$

For Class 4 Sections:

$$M_r = \begin{cases} \phi S_e F_y, & \text{when using effective section properties} \\ \phi S F_{y,\text{eff}}, & \text{when using effective yield stress} \end{cases}$$

where $\qquad$ (CSA 13.5(c))

S_e = the effective section modulus, and

$F_{y,\text{eff}}$ = the effective section yield stress.

The factored flexural resistance, M_r, for Class 4 sections (CSA 11, Table 1, Table 2) is calculated using the effective section modulus (CSA 13.5.c(iii)) or using an effective yield stress (CSA 13.5). The choice of using the effective section modulus or effective yield stress can be made in the preferences.

The effective section modulus is calculated using a reduced element width that meets the maximum width-to-thickness ratio of a Class 3 section (CSA 13.5.c(iii)). The use of effective section modulus in determining the factored moment resistance should be limited to members with Class 4 flanges and Class 3 or more compact webs. If the section has a web of Class 4, even though the program uses the preceding simplistic equation, a more detailed procedure needs to be followed by the user (CSA 13.5.c(i), 13.5.c(ii)). The user should do those checks independently or should use the effective yield stress method. The effective yield stress, $F_{y,\text{eff}}$, is determined from the width-to-thickness ratio that meets the Class 3 limit (CSA 13.5). The effective yield stress method can be used for both Class 4 flanges and Class 4 webs. The slenderness ratios are calculated using gross section properties.

For Lateral-Torsional Buckling Strength, the moment capacities are calculated using the procedure described in the following subsections (CSA 13.6, 13.8.2(c), 13.8.3(c)).

3.7.3.1   I-Shapes and Double Channels

3.7.3.1.1   Major Axis Bending

For Class 1 and 2 sections of I-Shapes and Double Channels bent about the major axis,

when $M_u > 0.67\ M_{p33}$,

$$M_{r33} = 1.15\phi M_{p33}\left[1 - 0.28\frac{M_{p33}}{M_u}\right] \le \phi M_{p33},\qquad \text{(CSA 13.6 (a)(i))}$$

when $M_u \le 0.67\ M_{p33}$,

$$M_{r33} = \phi M_u \qquad \text{(CSA 13.6 (a)(ii))}$$

where,

M_{r33} = Factored major bending resistance,
M_{p33} = Major plastic moment $= z_{33}F_y$,
M_u = Critical elastic moment,

$$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_{22}GJ + \left(\frac{\pi E}{L}\right)^2 I_{22}C_w} \qquad \text{(CISC 13.6 (a))}$$

L = Laterally unbraced length, l_{22},
C_w = Warping constant assumed as zero for Boxes and

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} \le 2.5. \qquad \text{(CSA 13.6 (a))}$$

where,

M_{max} = maximum factored bending moment in unbraced segment
M_a = factored bending moment at one-quarter point of unbraced segment

M_b = factored bending moment at midpoint of unbraced segment

M_c = factored bending moment at three-quarter point of unbraced segment

The program defaults ω_2 to 1.0 if the unbraced length, l, of the member is overwritten by the user (i.e., it is not equal to the length of the member). ω_2 should be taken as 1.0 for cantilevers. However, the program is unable to detect when the member is a cantilever. The user can overwrite the value of ω_2 for any member by specifying it.

For Class 3 and Class 4 sections of I-Shapes and Double Channels bent about the major axis,

when $M_u > 0.67 M_{y33}$,

$$M_{r33} = 1.15 \phi M_{y33} \left[1 - 0.28 \frac{M_{y33}}{M_u} \right] \leq \phi M_{y33}, \qquad \text{(CSA 13.6 (b)(i))}$$

when $M_u \leq 0.67 M_{y33}$,

$$M_{r33} = \phi M_u, \qquad \text{(CSA 13.6 (b)(ii))}$$

M_{r33} and M_u are as defined earlier in this section, and M_{y33} is the major yield moment.

3.7.3.1.2 Minor Axis Bending

For I-Shapes and Double Channels bent about their minor axis, the moment capacities are considered to be as follows:

For Class 1 and Class 2 Sections:

$$M_{r22} = \phi Z_{22} F_y = \phi M_{p22} \qquad \text{(CSA 13.5 (a))}$$

For Class 3 sections:

$$M_{r22} = \phi S_{22} F_y = \phi M_{y22}. \qquad \text{(CSA 13.5 (b))}$$

For Class 4 Sections:

when using effective section properties.

when using effective yield stress.

$$M_{r22} = \begin{cases} \phi S_{e22} F_y, \\ \phi S_{e22} F_{y,eff}, \end{cases}$$

(CSA 13.5 (c))

3.7.3.2 Rectangular Bar

3.7.3.2.1 Major Axis Bending

For Class 2 sections of Rectangular bars bent about their major axis,

when $\quad M_u > 0.67 M_{p33}$,

$$M_{r33} = 1.15 \phi M_{p33} \left[1 - 0.28 \frac{M_{p33}}{M_u} \right] \le \phi M_{p33}, \qquad \text{(CSA 13.6 (b))}$$

when $\quad M_u \le 0.67 M_{y33}$,

$$M_{r33} = \phi M_u. \qquad \text{(CSA 13.6 (b))}$$

3.7.3.2.2 Minor Axis Bending

For Class 2 sections of Rectangular bars bent about their minor axis,

when $\quad M_u > 0.67 M_{p22}$,

$$M_{r22} = 1.15 \phi M_{p22} \left[1 - 0.28 \frac{M_{p22}}{M_u} \right] \le \phi M_{p22}, \qquad \text{(CSA 13.6 (b))}$$

when $\quad M_u \le 0.67 M_{y22}$,

$$M_{r22} = \phi M_u. \qquad \text{(CSA 13.6 (b))}$$

3.7.3.3 Boxes

3.7.3.3.1 Major Axis Bending

For Boxes bent about the major axis, the moment capacities are considered to be as follows:

For Class 1 and 2 sections:

$$M_{r33} = \phi Z_{33} F_y = \phi M_{p33} \qquad \text{(CSA 13.6(e), 13.5(a))}$$

For Class 3 sections:

$$M_{r33} = \phi S_{33} F_y = \phi M_{y33} \qquad \text{(CSA 13.6(c), 13.5(b))}$$

For Class 4 sections:

$$M_{r33} = \begin{cases} \phi S_{e33} F_y, & \text{when using effective section properties.} \\ \phi S33 F_{y,eff}, & \text{when using effective yield stress.} \end{cases}$$

$$\text{(CSA 13.6(c), 13.5(c))}$$

3.7.3.3.2 Minor Axis Bending

For boxes bent about their minor axis, the moment capacities are considered to be as follows:

For Class 1 and 2 sections:

$$M_{r22} = \phi Z_{22} F_y = \phi M_{p22} \qquad \text{(CSA 13.6(e), 13.5(a))}$$

For Class 3 sections:

$$M_{r22} = \phi S_{22} F_y = \phi M_{y22} \qquad \text{(CSA 13.6(c), 13.5(b))}$$

For Class 4 sections:

$$M_{r22} = \begin{cases} \phi S_{e22} F_y, & \text{when using effective section properties.} \\ \phi S_{22} F_{y,eff}, & \text{when using effective yield stress.} \end{cases}$$

(CSA 13.6(c), 13.5(c))

3.7.3.4   Pipes and Circular Rods

For pipes and circular rods bent about any axis,

$$M_{r33} = \phi Z f_y \text{ for Class 1 and 2}$$
(CSA 13.5, 13.6 (c))

$$M_{r33} = \phi S f_y \text{ for Class 3}$$
(CSA 13.5, 13.6 (c))

$$M_{r33} = \phi S_e f_y \text{ for Class 4}$$
(CSA 13.5, 13.6 (c))

3.7.3.5   Channel Section

3.7.3.5.1   Major Axis Bending

For Class 3 and 4 Channel sections bent about their major axis,

when $M_u > 0.67 M_{y33}$,

$$M_{r33} = 1.15 \phi M_{y33} \left[1 - 0.28 \frac{M_{y33}}{M_u} \right] \leq \phi M_{y33},$$
(CSA 13.6 (b))

when $M_u \leq 0.67 M_{y33}$,

$$M_{r33} = \phi M_u.$$
(CSA 13.6 (b))

3.7.3.5.2   Minor Axis Bending

For Class 3 and 4 Channel sections bent about their minor axis,

$$M_{r22} = M_{y22} = \phi Z_{22} F_y.$$

3.7.3.6   T-Shapes and Double Angles

3.7.3.6.1   Major Axis Bending

For T-Shape and Double Angle sections bent about their major axis,

when $M_u > M_{yr33}$,

$$M_{r33} = \phi \left[M_{p33} - \left(M_{p33} - M_{yr33} \right) \left(\frac{L - L_u}{L_{yr} - L_u} \right) \right] \le \phi M_{P33}, \quad (\text{CSA 13.6 (e)})$$

where,

$M_{yr33} = 0.7 S_{min} F_y$, S_{min} is the smaller section modulus

$L_{yr} = $ Length L obtained by setting $M_u > M_{yr33}$,

$L_u = 1.1 \gamma_t \sqrt{E/F_y}$,

$\gamma_t = \dfrac{b_c}{\sqrt{12 \left(1 + \dfrac{h_c w}{3 b_c t_c} \right)}}$,

where,

h_c = depth of the web in compression
b_c = width of compression flange
t_c = thickness of compression flange

when $M_u \le M_{yr33}$,

$M_{r33} = \phi M_u$. $\qquad\qquad\qquad\qquad$ (CSA 13.6 (e))

where,

$$M_u = \frac{\omega_3 \pi^2 E I_{33}}{2L^2} \left[\beta_x + \sqrt{\beta_x^2 + 4\left(\frac{GJL^2}{\pi^2 E I_{33}} + \frac{C_w}{I_{33}}\right)} \right] \qquad \text{(CISC 13.6 (e))}$$

L = Laterally unbraced length, l_{22},

C_w = Warping constant, taken from section database

$$\beta_x = 0.9(d-t)\left(\frac{2I_{yc}}{I_{33}} - 1\right)\left(1 - \left(\frac{I_{33}}{I_{22}}\right)^2\right)$$

β_x = asymmetry parameter for singly symmetric beams

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

I_{yt} = moment of inertia of the tension flange about the y-axis

and when singly symmetric beams are in single curvature

$\omega_3 = \omega_2$ for Double Angle Sections

$\omega_3 = 1.0$ for T-Sections

3.7.3.6.2 Minor Axis Bending

For Class 3 and 4 sections of T-Shapes and Double Angles, the factored minor bending strength is assumed as,

$$M_{r22} = \phi F_y S_{22}.$$

3.7.3.7 Angle Sections

The factored flexural strength for lateral-torsional buckling limit state of Single Angle sections is conservatively calculated based on the principal axes Bending.

The factored flexure strength for bending about the major principal axis for the limit state of lateral-torsional buckling, M_{r33}, is given as follows:

$$M_{r33} = \phi \begin{cases} \left(92 - \dfrac{0.17M_e}{M_y}\right)M_e, & \text{if } M_e \le M_{y,} \text{ and} \\[2em] \left(1.92 - 117\sqrt{\dfrac{M_y}{M_e}}\right)M_y \le 1.5M_y, & \text{if } M_e > M_y. \end{cases}$$

(CSA 13.6(e), AISC F10-2, F10-3)

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

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

(CSA 13.6(e), AISC F10-5.6)

where,

C_b = Lateral-torsional buckling modification factor for non-uniform 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, mm⁴.

r_t = Radius of gyration for the minor principal axis, mm.

t = Angle leg thickness, mm. It is taken as min(t_w, t_f).

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

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

β_w is positive for short legs in compression, negative for long legs 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 earlier equation, M_y is taken as the yield moment about the major principal axis Bending, considering the possibility of yielding at the heel and both of the leg tips.

The factored flexural strength for bending about the minor principal axis for the limit state of lateral-torsional buckling, M_{r22}, is taken as follows:

$$M_{r22} \quad = \quad \phi Z_{22} F_y \le 1.5 \; \phi S_{22} F_y \qquad \text{(CSA 13.6(e), AISC F10-2)}$$

3.7.3.8   General Sections

For General sections, the factored major and minor direction bending strengths are assumed as,

$$M_{r33} = \phi F_y S_{33}, \text{ and}$$

$$M_{r22} = \phi F_y S_{22}.$$

3.7.4   Shear Strength

The factored 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 factored strength for shear, V_r, it is assumed that there is no internal stiffness used to enhance shear strength of a section (CSA 13.4.1.1).

3.7.4.1   Shear in the Major Direction

The factored shear strength, V_{r2}, for major direction shears in I-Shapes, Boxes, Channels, Double-Channels, and T-Shapes is calculated as follows:

$$V_{r2} \quad = \quad \phi A_w F_s \qquad\qquad \text{(CSA 13.4.1.1)}$$

where A_w is the design shear area. It is taken as follows:

$$A_w = \begin{cases} dw & \text{for I-Shape sections,} \\ dw & \text{for Channel sections,} \\ dw & \text{for T-Shape sections,} \\ 2dw & \text{for Double Channel sections, and} \\ 2hw & \text{for Box sections.} \end{cases} \qquad \text{(CSA 13.4.1.1)}$$

F_s is the ultimate shear stress. It is taken as follows:

For unstiffened webs:

when $\dfrac{h}{w} \le \dfrac{1014}{\sqrt{F_y}}$

$$F_s = \ 0.66 F_y \qquad\qquad \text{(CSA 13.4.1.1(a)(i))}$$

when $\dfrac{1014}{\sqrt{F_y}} < \dfrac{h}{w} \le \dfrac{1435}{\sqrt{F_y}}$

$$F_s = \dfrac{670\sqrt{F_y}}{\left(h/w\right)} \qquad\qquad \text{(CSA 13.4.1.1(a)(ii))}$$

when $\quad \dfrac{h}{w} > \dfrac{1435}{\sqrt{F_y}}$

$$F_s = \dfrac{961200}{\left(h/w\right)^2} \qquad\qquad \text{(CSA 13.4.1.1(a)(iii))}$$

h/w is taken as follows:

$$\dfrac{h}{w} > \dfrac{h}{t_w}$$

The factored shear strength, V_{r2}, for major direction shears in Double Angle, Angle, Rectangular, Circular, Pipe, and General sections is calculated as follows:

$$V_{r2} = \phi A_w F_s \qquad\qquad \text{(CSA 13.4.1.1)}$$

where

$$F_s = 0.66 F_y \qquad\qquad \text{(CSA 13.4.1.1)}$$

3.7.4.2   Shear in the Minor Direction

The factored shear strength, V_{r3}, for minor direction shear in I-Shapes, Boxes, Channels, Double Channels, and T-Shapes is calculated as follows:

$$V_{r3} = \phi A_w F_s \qquad\qquad \text{(CSA 13.4.1.1)}$$

where, A_w is taken as follows:

$$A_w = \begin{cases} 2b_f t_f & \text{for I-Shape sections,} \\ 2b_f t_f & \text{for Channel sections,} \\ b_f t_f & \text{for T-Shape sections,} \\ 4b_f t_f & \text{for Double Channel sections, and} \\ 2b_f t_f & \text{for Box sections.} \end{cases}$$

F_s is calculated in the same way as that for major direction shear, except that h/w is taken as follows:

$$h/w = \begin{cases} b_f/(2t_f) & \text{for I-Shape sections,} \\ b_f/t_f & \text{for Channel sections,} \\ b_f/t_f & \text{for T-Shape sections,} \\ b_f/2t_f & \text{for Double Channel sections, and} \\ b_f/t_f & \text{for Box sections.} \end{cases}$$

The factored shear strength, V_{r3}, for minor direction shears in Double Angle, Angle, Rectangular, Circular, Pipe, and General sections is calculated as follows:

$$V_{r3} = \phi A_w F_s \qquad \text{(CSA 13.4.1.1)}$$

where,

$$F_s = 0.66F_y. \qquad \text{(CSA 13.4.1.1)}$$

3.8 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.

3.8.1   Axial and Bending Stresses

From the factored axial loads and bending moments at each station and the factored strengths for axial tension and compression and major and minor axis bending, a D/C ratio is produced for each of the load combinations as follows:

3.8.1.1   Axial Compressive and Bending — Class 1 and Class 2 I-Shapes

If the axial load is compressive, the D/C ratio for Class 1 and Class 2 sections of I-Shaped members is given by:

$$\frac{C_f}{C_r} + 0.85\frac{U_{13}M_{f33}}{M_{r33}} + \beta\frac{U_{12}M_{f22}}{M_{r22}} \qquad \text{(CSA 13.8.2)}$$

where,

C_f is the factored axial compressive loading, including $P\text{-}\Delta$ effects as defined in CSA clause 8.4,

M_{f33} and M_{f22} are the factored major and minor axis bending moment, including $P\text{-}\Delta$ effects as described in CSA clause 8.4,

C_r is the factored axial compression capacity for different limit states (CSA 13.3),

M_{r33} and M_{r22} are the factored major and minor axis flexural capacity for different limit states (CSA 13.5, 13.6),

U_{12} and U_{13} are two factors to account for the second-order effects due to the deformation of a member between its ends. They are different for different limit states.

β is a coefficient, and it is taken as follows:

$$\beta = 0.6 + 0.4\,\lambda_y \le 0.85 \qquad \text{(CSA 13.8.2)}$$

where,

$$\lambda_y = \frac{KL_{22}}{r_{22}}\sqrt{\frac{F_y}{\pi^2 E}} \qquad \text{(CSA 13.3.1)}$$

U_1 is taken as follows:

$$U_1 = \frac{\omega_1}{1 - \dfrac{C_f}{C_e}}, \text{ where,} \qquad\qquad \text{(CSA 13.8.4)}$$

$$C_e = \frac{\pi^2 EI}{L^2}. \qquad\qquad \text{(CSA 13.8.4)}$$

The factor U_1 must be a positive number. Therefore, C_f must be less than C_e. If this in not true, a failure condition is declared.

If the factor U_{12} or U_{13} is overwritten by the user, the calculated value is replaced with the user value.

ω_1 is the coefficient related to the distribution of moment over the span within the braced segment.

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

$$\omega_1 = 0.6 - 0.4 (M_a / M_b) \geq 0.4 \qquad\qquad \text{(CSA 13.8.5(a))}$$

where M_a and M_b, calculated from the analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane 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 distributed transverse loading between supports, the value of ω_1 is taken as 1.0 (CSA 13.8.5(b)).

(iii)   For beam-columns subjected to a single concentrated transverse loading between supports, the value of ω_1 is taken as 0.85 (CSA 13.8.5(c)).

When M_b is zero, ω_1 is taken as 1.0. The program defaults ω_1 to 1.0, if the unbraced length is more than the actual member length. The user can overwrite the value of ω_1 for any member. ω_1 can be expressed as follows:

$$\omega_1 = \begin{cases} 1.00, & \text{if length is more than actual length,} \\ 1.00, & \text{if tension member,} \\ 1.00, & \text{if both ends unrestrained} \\ 0.6 - 0.4\dfrac{M_a}{M_b}, & \text{if no transverse loading,} \\ 1.00, & \text{if transverse loading is present, and} \\ 0.85, & \text{if single concentrated load is present.} \end{cases}$$

(CSA 13.8.5)

The program checks the interaction equation (CSA 13.8.2) for three different limit states: (a) Cross-Sectional Strength; (b) Overall Member Strength; and (c) Lateral-Torsional Buckling Strength. In each case, β_1, U_{13}, U_{12}, C_r, Mr_{33}, and Mr_{22} may assume different values.

(a)   Cross-Section Strength (CSA 13.8.2(a))

- The axial compression capacity is based on the assumption that the member is fully braced in each direction ($L_{22} = 0$, $L_{33} = 0$), i.e., $\lambda = 0$. The axial compression capacity is based on plastic limit state.

    $$C_r = \phi A F_y \qquad \text{(CSA 13.8.2(a), 13.3.1)}$$

    If the axial capacity C_r is overwritten by the user, it is assumed to not apply to this limit state and so is ignored.

- The M_{r33} and M_{r22} are calculated assuming that the members are fully supported ($L_{22} = 0$, $L_{33} = 0$, $L_z = 0$) irrespective of their actual lateral lengths. The moment capacities are given by CSA Clause 13.5 (CSA 13.8.2(a), 13.5).

    If the capacities (M_{r33} and M_{r22}) are overwritten by the user, they are assumed to not apply for this limit state and so are ignored.

- U_{12} and U_{13} are calculated following the procedure described previously (CSA 13.8.4), but are taken to be not less than 1.

    $$U_{12} \geq 1.0 \qquad \text{(CSA 13.8.2(b), 13.8.4)}$$

    $$U_{13} \geq 1.0 \qquad \text{(CSA 13.8.2(a), 13.8.4)}$$

- The β factor is taken as 0.6

$$\beta = 0.6 \qquad\qquad\qquad \text{(CSA 13.8.2(a))}$$

(b) Overall Member Strength (CSA 13.8.2(b))

- The axial compression capacity is based on the assumption that $K_{33} = 1$ and $K_{22} = 1$. For biaxial bending, C_r is taken as the minimum axial capacity for the flexural buckling limit state for $\dfrac{K_{22}L_{22}}{r_{22}}$, and

  $\dfrac{K_{33}L_{33}}{r_{33}}$. However for uniaxial strong axis bending, it is based on the

  flexural buckling limit state for $\dfrac{K_{22}L_{22}}{r_{22}}$. (CSA 13.8.2(b), 13.3.1)

  If the axial capacity C_r is overwritten by the user, it is assumed to not apply in this limit state and so is ignored.

- M_{r33} and M_{r22} are calculated assuming that the members are fully supported ($L_{22} = 0$, $L_{33} = 0$, $L_z = 0$) irrespective of their actual lateral lengths. The moment capacities are given by CSA Clause 13.5 (CSA 13.8.2(b), 13.5).

  If the capacities (M_{r33} and M_{r22}) are overwritten by the user, they are assumed to not apply for this limit state and so are ignored.

- U_{12} and U_{13} for members of braced frames are calculated following the procedure described previously (CSA 13.8.4). But for members of unbraced frames, they are taken as 1.0 (CSA 13.8.2(b), 13.8.4).

- The β factor is calculated following the procedure described previously (CSA 13.8.2).

(c) Lateral-Torsional Buckling Strength (CSA 13.8.2(c))

- The axial compression capacity is taken as the minimum axial compression capacity considering flexural buckling (CSA 13.3.1) and torsional or lateral-torsional buckling (CSA 13.3.2). For the flexural buckling limit state, the compression capacity is based on both the major and minor direction buckling using both $\dfrac{K_{22}L_{22}}{r_{22}}$ and $\dfrac{K_{33}L_{33}}{r_{33}}$

  with the actual values of K_{22} and K_{33}. For the torsional and lateral-

torsional buckling limit state, additional slenderness parameter $\dfrac{K_z L_z}{r_{min}}$ is used.

If the axial capacity C_r is overwritten by the user, the overwritten value affects the C_r for this limit state.

- M_{r33} is calculated using the appropriate values of L_{22}, L_{33}, L_z, K_{22}, K_{33} and K_z (CSA 13.8.2(c), 13.5, 13.6).

    If the capacity M_{r33} is overwritten by the user, the overwritten value affects the M_{r33} for this limit state.

- M_{r22} is calculated assuming that the members are fully laterally supported ($L_{22} = 0$, $L_{33} = 0$, $L_z = 0$) irrespective of its actual lateral bracing length. The moment capacities are given by CSA Clause 13.5 (CSA 13.8.2(c), 13.5).

    If the capacity M_{r22} is overwritten by the user, it is assumed to not apply for this limit state and so is ignored.

- U_{12} and U_{13} are taken as 1 for unbraced frames (CSA 13.8.2(c)).

$$U_{13} = 1 \qquad \text{(for unbraced frame)} \qquad \text{(CSA 13.8.2(c))}$$

$$U_{12} = 1 \qquad \text{(for unbraced frame)} \qquad \text{(CSA 13.8.2(c))}$$

U_{12} for members of braced frames is taken as the calculated value following the procedure described previously (CSA 13.8.4).

U_{13} for members of braced frames is taken as the calculated value following the procedure described previously (CSA 13.8.4), except it is taken to be not less than 1 (CSA 13.8.2(c)).

$$U_{13} \geq 1.0 \qquad \qquad \text{(CSA 13.8.2(c), 13.8.4)}$$

In addition to the preceding interaction equation, for Class 1 and Class 2 I-Shaped members, the following D/C ratio also is checked.

$$\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}} \qquad \qquad \text{(CSA 13.8.2(c))}$$

where,

M_{f33} and M_{f22} are the factored major and minor axes bending moments, including P-Δ effect as described in CSA Clause 8.4, and

M_{r33} and M_{r22} are the factored major and minor axes flexural capacities considering both Clauses CSA 13.5 and 13.6, as appropriate.

3.8.1.2 Axial Compression and Bending — Other Sections

If the axial load is compressive, for all classes of sections except Class 1 and Class 2 I-Shaped sections, the D/C ratio is given by:

$$\frac{C_f}{C_r} + \frac{U_{13}M_{f33}}{M_{r33}} + \frac{U_{12}M_{f22}}{M_{r22}} \qquad \text{(CSA 13.8.3)}$$

where,

C_f is the factored axial compressive loading, including P-Δ effect as defined in CSA Clause 8.4,

M_{f33} and M_{f22} are the factored major and minor axis bending moment, including P-Δ effect as described in CSA Clause 8.9,

C_r is the factored axial compression capacity for different limit states (CAS 13.3),

M_{r33} and M_{r22} are the factored major and minor axis flexural capacity for different limit states (CSA 13.5, 13.6),

U_{12} and U_{13} are two factors to account for the second-order effects due to the deformation of a member between its ends. They are different for different limit states.

The program checks the interaction equation (CSA 13.8.3) for the three different limit states: (a) Cross-Sectional Strength; (b) Overall Member Strength; and (c) Lateral-Torsional Buckling Strength. In each case, U_{13}, U_{12}, C_r, M_{r33}, and M_{r22} may assume different values. The three limit states are handled in the same manner as those handled for the Class 1 and Class 2 I-Shaped sections.

Please read the previous section for the appropriate values of U_{13}, U_{12}, C_r, M_{r33}, and M_{r22} for all three limit states (CSA 13.8.3(a), 13.8.3(b), 13.8.3(c)).

3.8.2 Axial Tension and Bending

If the axial load is tensile, the D/C ratio is given by the larger of two ratios. In the first case, the ratio is calculated as

$$\left[\frac{T_f}{T_r}\right] + \left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right], \qquad \text{(CSA 13.9.1)}$$

assuming M_{r22} and M_{r33} are calculated based on a fully supported member ($L_{22} = 0$ and $L_{33} = 0$). If the capacities (T_r, M_{r22}, and M_{r33}) are overwritten by the user, the only overwritten capacity used in this case is T_r.. M_{r22} and M_{r33} overwrites are assumed to not apply to this case and are ignored.

In the second case, the ratio is calculated as

$$\left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right] - \left[\frac{T_f\, Z_{33}}{M_{r33}\, A}\right] \quad \text{(for Class 1 and 2 Sections)} \quad \text{(CSA13.9.2 (a))}$$

$$\left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right] - \left[\frac{T_f\, S_{33}}{M_{r33}\, A}\right] \quad \text{(for Class 3 and 4 Sections)} \quad \text{(CSA13.9.2(b))}$$

If the capacities (M_{r22} and M_{r33}) are overwritten by the user, both of these overwritten capacities are used in this case.

For Circular and Pipe sections, an SRSS combination is first made of the two bending components before adding the axial load component instead of the simple algebraic addition implied by the preceding interaction formulas.

3.8.3 Shear Stresses

From the factored shear force values and the factored shear strength values at each station, for each of the load combinations, D/C ratios for shear in major and minor directions are produced as follows:

$$\frac{V_{f2}}{V_{r2}}, \text{ and}$$

$$\frac{V_{f3}}{V_{r3}}.$$

Chapter 4
Special Seismic Provisions

This chapter provides a detailed description of the algorithms related to seismic provisions in the design/check of structures in accordance with the "CSA S16-14, Design of Steel Structure" (CSA 2014). The loading based on "NBCC 2005" has been described in a separate document entitled "CSI Lateral Load Manual" (NBCC 2010; CSI 2012).

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

- Reference to the CSA S16-14 code is identified with the prefix "**CSA**."

- Reference to the NBCC 2010 code is identified with the prefix "**NBCC**."

4.1   Design Preferences

The steel frame design Preferences are basic assignments that apply to all of the steel frame members. The Preferences have been described previously in the relevant section of Chapter 3. Table 3-1 lists the steel frame design Preferences. The following steel frame design Preferences are relevant to the special seismic provisions.

- Framing Type

- Ductility Related Modification Factor, R_d

- Overstrength Related Modification Factor, R_o

- Spectral Acceleration Ratio, $I_eF_aS_a(0.2)$

- Ignore Seismic Code?

- Ignore Special Seismic Load?

- Is Doubler Plate Plug Welded?

4.2 Overwrites

The steel frame design Overwrites are basic assignments that apply only to those elements to which they are assigned. The overwrites have been described previously in the relevant section of Chapter 3. Table 3-2 lists the steel frame design Overwrites. The following steel frame design overwrites are relevant to the special seismic provisions.

- Frame Type

4.3 Supported Framing Types

The code now recognizes the types of framing systems identified in the table on the following page (CSA 27.2 through 27.11). With regard to those framing types, the program has implemented specifications for all of the types of framing systems listed.

By default in the program, the frame type is taken as Limited-Ductile Moment-Resisting Frame (Type LD MRF). However, the default frame type can be changed in the Preferences for all frames or in the Overwrites 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.

Framing Type	References
Type D MRF (Ductile Moment-Resisting Frame)	CSA 27.2
Type MD MRF (Moderately Ductile Moment-Resisting Frame)	CSA 27.3
Type LD MRF (Limited-Ductility Moment-Resisting Frame)	CSA 27.4
Type MD CBF(TC) (Moderately Ductile Concentrically Braced Frame – Tension-Compression)	CSA 27.5.2.3
Type MD CBF(V) (Moderately Ductile Concentrically Braced Frame – Chevron)	CSA 27.5.2.4
Type MD CBF(TO) (Moderately Ductile Concentrically Braced Frame – Tension only)	CSA 27.5.2.5
Type MD CBF(OT) (Moderately Ductile Concentrically Braced Frame – Others)	CSA 27.5.3
Type LD CBF(TC) (Limited-Ductility Concentrically Braced Frame – Tension-Compression)	CSA 27.6.2.1
Type LD CBF(V) (Limited-Ductility Concentrically Braced Frame – Chevron)	CSA 27.6.2.2
Type LD CBF(TO) (Limited-Ductility Concentrically Braced Frame – Tension only)	CSA 27.6.2.3
Type LD CBF(OT) (Limited-Ductility Concentrically Braced Frame – Others)	CSA 27.6.3
EBF (Ductile Eccentrically Braced Frame)	CSA 27.7
Conventional MF	CSA 27.11.1
Conventional BF	CSA 27.11.1
Cantilever Column	CSA 27.11.2

4.4   Member Design

This section describes the special requirements for designing a member. The section has been divided into subsections for each framing type.

4.4.1   Type (Ductile) Moment-Resisting Frames (D MRF)

For this framing system ($R_d = 5.0$, $R_o = 1.5$), the following additional requirements are checked or reported (CSA 27.2).

- All beams are required to be Class 1 sections (CSA 27.2.2(a)).

All beams are required to be laterally braced in accordance with the requirements of CSA 13.7(b) (CSA 27.2.2(b)).

$$\frac{L_{z2}}{r_{22}} \leq \frac{17\,250 + 15\,500\kappa}{F_y} \qquad \text{(CSA 13.7(b), 27.2.2(b))}$$

The value of κ is computed from the bending moment distribution for combined gravity and seismic loads.

If any of these criteria are not satisfied, the program issues an error message.

▪ All columns are required to be Class 1 or Class 2 (CSA 27.2.3.1(a).

All columns are required to be laterally braced in accordance with the requirements of CSA Clause 13.7(b) (CSA 27.2.3.1(b)).

$$\frac{L_{22}}{r_{22}} \leq \frac{17\,250 + 15\,500\kappa}{F_y} \qquad \text{(CSA 13.7(b), 27.2.3.1(b))}$$

using $\kappa = 0$ (CSA 27.2.3.1(b)).

The factored axial load shall not exceed $0.3\phi AF_y$ when the specified one-second spectral acceleration ratio (I_E, F_v, S_a (1.0)) is greater than 0.30.

If any of these criteria are not satisfied, the program issues an error message.

▪ Columns need to be designed to resist gravity loads together with the forces induced by plastic hinging of the beams at each beam-to-column intersection:

$$\sum M'_{rc} \geq \sum \left(1.1 R_y M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right) \qquad \text{(CSA 27.2.3.2)}$$

where

$\sum M'_{rc}$ = sum of the column factored flexural resistance at the intersection of the beam and column centerlines.

$$\sum M'_{rc} = 1.18\phi \left(1 - \frac{C_f}{\phi C_y} \right) M_{pc} \leq \phi M_{pc} \qquad \text{(CSA 27.2.3.2)}$$

M_{pc} = nominal plastic moment resistance of the column

C_f = results from summation of V_h acting at this level and above

M_{pb} = nominal plastic moment resistance of the beam

x = distance from the center of a beam plastic hinge to the column face

The program calculates only the beam-column capacity ratio. The program does not design the column for this load yet.

4.4.2 Type (Moderately Ductile) Moment-Resisting Frames (MD MRF)

For this framing system ($R_d = 3.5$, $R_o = 1.5$), the following additional requirements are checked or reported (CSA 27.3).

- All beams are required to be Class 1 or Class 2 sections (CSA 27.3(a)(1)).

All beams need to be designed to be laterally braces in accordance with the requirements of CSA 13.7(a) (CSA 27.3, 27.2.2).

$$\frac{L_{cr}}{r_y} = \frac{25\,000 + 15\,000\kappa}{F_y} \qquad \text{(CSA 13.7(a), 27.3(a)(ii), 27.2.2(a))}$$

The value of κ is computed from the bending moment distribution for combined gravity and seismic loads (CSA 27.3(a)).

If any of these criteria are not satisfied, the program issues an error message.

- All columns are required to be Class 1 or Class 2 sections (CSA 27.3, 27.2.3.1).

All columns need to be designed to be laterally braced in accordance with the requirements of CSA Clause 13.7(b) (CSA 27.3, 27.2.3.1(b)).

$$\frac{L_{cr}}{r_y} = \frac{17\,250 + 15\,500\kappa}{F_y} \qquad \text{(CSA 13.7(a), 27.3(a)(ii), 27.2.2(a))}$$

using $\kappa = 0.0$.

The factored axial loads shall not exceed $0.5\phi AF_y$ when the specified one-second spectral acceleration ratio ($I_E F_V S_a(1.0)$) is greater than 0.5 (CSA 27.3(b), 27.2.3.1(b)).

If any of these criteria are not satisfied, the program issues an error message.

- Columns need to be designed to resist gravity load together with the forces induced by plastic hinging of the beams at each beam-to-column intersection.

$$\sum M_{rc}' \geq \sum \left(1.1 R_y M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right)$$ (CSA 27.2.3.2, 27.3)

where

$\sum M_{rc}'$ = sum of the column factored flexural resistance at the intersection of the beam and column centerlines.

$$\sum M_{rc}' = 1.18\phi \left(1 - \frac{C_f}{\phi C_y} \right) M_{pc} \leq \phi M_{pc}$$ (CSA 27.2.3.2)

M_{pc} = nominal plastic moment resistance of the column

C_f = results from summation of V_h acting at this level and above

M_{pb} = nominal plastic moment resistance of the beam

x = distance from the center of a beam plastic hinge to the column face

The program calculates only the beam-column capacity ratio. The program does not design the column for this load yet.

4.4.3 Type (Limited-Ductility) Moment-Resisting Frames (LD MRF)

For this framing system ($R_d = 2.0$, $R_o = 1.3$), the following additional requirements are checked or reported (CSA 27.4).

- All beams are required to be Class 1 or Class 2 sections. If this criterion is not satisfied, the program issues an error message (CSA 27.4.2.1).

- All columns are required to be Class 1 (CSA 27.4.2.1) I-Shaped sections (CSA 27.4.4.3).

- When the specified short-period spectral acceleration ratio ($I_E F_V S_a(0.2)$) is greater than 0.55 or the building height is greater than 60 m (CSA 27.4.2.2), columns shall satisfy the requirements of CSA 27.2.3.2 with modification of the term $1.1 R_y M_{pb}$ to M_{pb} (CSA 27.4.2.2).

$$\sum M'_{rc} \geq \sum \left(M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right) \tag{CSA 27.4.2.2}$$

where,

$\sum M'_{rc}$ = sum of the column factored flexural resistance at the intersection of the beam and column centerlines.

$$\sum M'_{rc} = 1.18\phi \left(1 - \frac{C_f}{\phi C_y} \right) M_{pc} \leq \phi M_{pc} \tag{CSA 27.2.3.2}$$

M_{pc} = nominal plastic moment resistance of the column

C_f = results from summation of V_h acting at this level and above

M_{pb} = nominal plastic moment resistance of the beam

x = distance from the center of a beam plastic hinge to the column face

If this criterion is not satisfied, the program issues an error message.

4.4.4 Type (Moderately Ductile) Concentrically Braced Frames (MD CBF)

For this framing system ($R_d = 3.0$, $R_o = 1.3$), the following additional requirements are checked or reported (CSA 27.5).

Moderately ductile concentrically braced frames include the following (CSA 27.5.2.1):

(a) tension-compression bracing system (TC)
(b) chevron braced system (V)
(c) tension only bracing system (TO)
(d) systems other than those in items (a), (b), and (c) (OT).

▪ The slenderness ratio, KL/r, of diagonal bracing members shall not exceed 200 (CSA 27.5.3.1).

▪ For buildings with specified short-period spectral acceleration, $I_e F_u S_a(0.2)$, equal to or greater than 0.35, the width-to-thickness ratio of the flanges of

diagonal bracing members shall not exceed the following limits (CSA 27.5.3.2).

(a) when $KL/r \leq 100$,

  (i)   for rectangular and square HSS: $330/\sqrt{F_y}$; for circular HSS: $10000/F_y$;

  (ii) for legs of angles and flanges of channels: $145/\sqrt{F_y}$;

  (iii) for other elements: Class 1;

(b) when $KL/r = 200$

  (i)   for HSS members: Class 1;

  (ii) for legs of angles: $170/\sqrt{F_y}$;

  (iii) for other elements: Class 2; and

(c) when $100 < KL/r < 200$, linear interpolation is used.

When $I_E F_a S_a(0.2)$ is less than 0.35, sections shall be Class 1 or 2, with the exception of HSS, which shall be Class 1. The width-to-thickness ratio for legs of angles shall not exceed $170/\sqrt{F_y}$ (CSA 27.5.3.2).

In all of the preceding cases, for back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$.

▪ Columns in braced bays shall be Class 1 or Class 2 sections (CSA 27.5.5.2) and shall have a bending resistance of not less than $0.2\,ZF_y$ in combination with the computed axial loads (CSA 27.5.5.2). If any of these criteria are not satisfied, the program issues an error message.

▪ Columns shall never have a slender section (CSA 27.5.5.2).

4.4.4.1   Tension-Compression (TC)

▪ Tension-compression concentrically braced system shall not exceed 40 m in height, except when the specified short-period spectral acceleration is less than 0.35 (CSA 27.5.2.3). The factored seismic force is increased by 3% per meter of height above 32 m when the building height exceeds 32 m. This

clause has not been implemented in the program yet. The user is required to check this clause independently.

4.4.4.2 Chevron(V)

▪ Chevron concentrically braced system shall not exceed 40 m in height, except when the specified short-period spectral acceleration is less than 0.35 (CSA 27.5.2.4). The factored seismic force is increased by 3% per meter of height above 32 m when the building height exceeds 32 m. This clause has not been implemented in the program yet. The user is required to check this clause independently.

▪ The beams to which chevron bracing is attached shall be continuous and shall resist bending moments due to gravity loads (assuming no vertical support is provided by the bracing members), in conjunction with bending moments and axial forces induced by forces of $A_g R_y F_y$ and $0.2 A_g R_y F_y$ in the tension and compression bracing members, respectively (CSA 27.5.2.4).

While calculating the bending moment, if the braces are connected to the beam from above, the brace compression force shall be taken as 1.2 times the probable compressive resistance of the bracing member (CSA 27.5.2.4).

In the case of buildings up to four stories, the tension brace force may be taken as $0.6 A_g R_y F_y$, provided that the beam is a Class 1 section (CSA 27.5.2.4).

4.4.4.3 Tension-Only (TO)

▪ Tension-only concentrically braced systems shall not exceed 20 m in height. The factored seismic force is increased by 3% per meter of height above 16 m when the building height exceeds 16 m (CSA 27.5.2.5(a)). This clause has not been implemented in the program yet. The user is required to check this clause independently.

4.4.5 Type (Limited-Ductility) Concentrically Braced Frames (LD CBF)

For this framing system ($R_d = 2.0$, $R_o = 1.3$), the following additional requirements are checked or reported (CSA 27.6).

Limited ductility concentrically braced frames include the following (CSA 27.6.2.1 to 27.6.2.3):

(a) tension-compression bracing system (TC)
(b) chevron braced system (V)
(c) tension only bracing system (TO)
(d) systems other than those in items (a), (b), and (c) (OT).

▪ For buildings with specified short-period spectral acceleration equal to or greater than 0.45, the width-to-thickness ratio of flanges of diagonal bracing members shall not exceed the following limits (CSA 27.5.3.2, 27.6.1).

(a) when $KL/r \leq 100$,

  (i)　for rectangular and square HSS: $330/\sqrt{F_y}$; for circular HSS: $10000/\sqrt{F_y}$;

  (ii)　for legs of angles and flanges of channels: $145/\sqrt{F_y}$;

  (iii)　for other elements: Class 1;

(b) when $KL/r = 200$,

  (i)　for HSS members: Class 1;

  (ii)　for legs of angles: $170/\sqrt{F_y}$;

  (iii)　for other elements: Class 2; and

(c) when $100 < KL/r < 200$, linear interpolation is used.

(d) when $KL/r > 200$ as permitted in clause 27.6.3.1, width-to-thickness limits do not apply (CSA 27.6.3.2(a)).

   When $I_z F_a S_a (0.2)$ is less than 0.45, sections shall be Class 1 or Class 2. The width-to-thickness ratio for legs of angles shall not exceed $170/\sqrt{F_y}$ (CSA 27.5.3.2, 27.6.1).

  In all of the preceding cases, for back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$.

▪ Columns in braced bays shall be Class 1 or Class 2 sections (CSA 27.5.5.2) and shall have a bending resistance of not less than $0.2 Z F_y$ in combination

with the computed axial loads (CSA 27.5.5.2). If these criteria are not satisfied, the program issues an error message.

4.4.5.1 Tension-Compression (TC)

▪ Tension-compression concentrically braced systems shall not exceed 60 m in height, except where the specified short-period spectral acceleration is less than 0.35 (CSA 27.6.2.1). The factored seismic force is increased by 2% per meter of height above 48 m when the building height exceeds 48 m. This clause has not been implemented in the program yet. The user is required to check this clause independently.

4.4.5.2 Chevron(V)

▪ Chevron concentrically braced systems shall not exceed 60 m in height (CSA 27.6.2.2). The factored seismic force is increased by 2% per meter of height above 48 m when the building height exceeds 48 m (CSA 27.6.2.1). This clause has not been implemented in the program yet. The user is required to check this clause independently.

▪ The beams to which chevron bracing is attached shall be continuous and shall resist bending moments due to gravity loads (assuming no vertical support is provided by the bracing members), in conjunction with bending moments and axial forces induced by forces of $A_gR_yF_y$ and $0.2\,A_gR_yF_y$ in the tension and compression bracing members, respectively (CSA 27.6.1, 27.5.2.4).

When braces are connected to the beam from above, the brace compression force shall be taken as 1.2 times the probable compressive resistance of the bracing member (CSA 27.6.1, 27.5.2.4).

In the case of buildings up to four stories, the tension brace force may be taken as $0.6\,A_gR_yF_y$, provided that the beam is a Class 1 section (CSA 27.6.1, 27.5.2.4).

The exception clause in this section is not used by the program.

4.4.5.3   Tension-Only (TO)

- Tension-only concentrically braced systems shall not exceed 40 m in height. The factored seismic force is increased by 3% per meter of height above 32 m when the building height exceeds 32 m (CSA 27.6.2.3(a)).

- The slenderness ratio, KL/r, of diagonal bracing member shall not exceed 200 (CSA 27.6.1, 27.5.3.1) with the following exception. In single-story and two-story structures, the slenderness ratio of diagonal bracing members that are connected and designed as tension only bracing members (CSA 27.5.2.5) shall not exceed 300 (CSA 27.6.3.1).

4.4.6   Eccentrically Braced Frames (EBF)

For this framing system ($R_d = 4.0$, $R_o = 1.5$), the following additional requirements are checked or reported (CSA 27.7).

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.

- When $e \leq 1.6\, M_p/V_p$, the link beams may have Class 2 flanges and Class 2 web (CSA 27.7.2.4); otherwise, link beams are checked to be Class 1 (CSA 27.7.2.4).

  where,

  e = length of link

  $$V_p = 0.55 t_w d F_y \qquad\qquad \text{(CSA 27.7.2.4)}$$

  If this criterion is not satisfied, the program issues an error message.
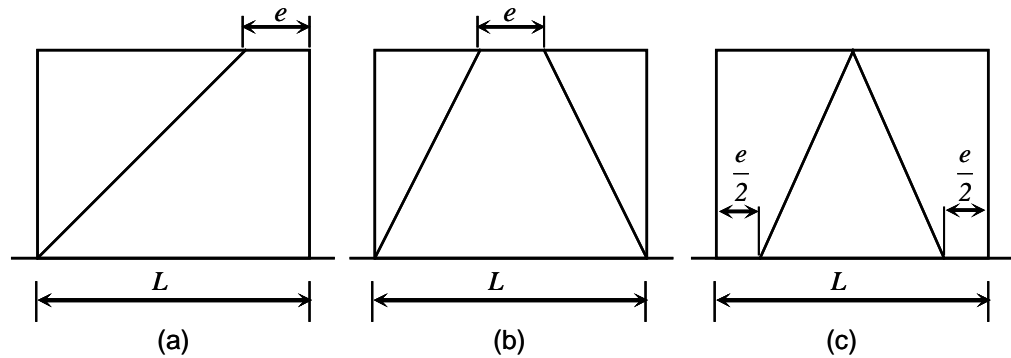
Figure 4-1. Eccentrically Braced Frame Configurations

- The shear resistance for link beams is taken as follows (CSA 27.7.3):

$$V_r \leq \min\left(\phi V_p,\ 2\phi M_p \big/ e\right) \tag{CSA 27.7.3}$$

where,

$$V'_p = V_p \sqrt{\left(1 - \frac{P_f}{AF_y}\right)^2},\tag{CSA 27.7.3}$$

$$M'_p = 1.18 M_p \left[1 - \frac{P_f}{AF_y}\right] \leq M_p,\tag{CSA 27.7.3}$$

$$V_p = 0.55 t_\omega d\, F_y,\tag{CSA 27.7.3}$$

$$P_f = \text{Axial force in link } (C_f \text{ or } T_f),\tag{CSA 27.7.3}$$

$$e\ = \text{length of link},\tag{CSA 27.7.3}$$

$$A\ = \text{gross area of link beam}.\tag{CSA 27.7.3}$$

- If $P_f > 0.15 AF_y,$ the link beam length, $e,$ is checked not to exceed the following:

  - if $\dfrac{A_w}{A} \geq 0.3 \dfrac{V_f}{P_f}$

$$e \leq \left[1.15 - 0.5 \frac{P_f}{V_f} \frac{A_w}{A} \right] \left[1.6 \frac{M_p}{V_p} \right] \qquad \text{(CSA 27.7.4)}$$

– if $\dfrac{A_w}{A} < 0.3 \dfrac{V_f}{P_f}$

$$e \leq \left[1.6 \frac{M_p}{V_p} \right] \qquad \text{(CSA 27.7.4)}$$

where,

$A_w = (d - 2t_f) t_w$ for I-Shaped, Channel and T-Shaped sections

(CSA 27.7.4)

    $= 2(d - 2t_f)t_w$ for Box and Double Channel section

    $= \dfrac{A_{v2}}{A}$ for all other sections.

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 (CSA 27.7.5):

$$\theta = \frac{\Delta L}{e}$$

– $\theta \leq 0.08$ radian, where link beam clear length, $e \leq 1.6 M_p / V_p$

– $\theta \leq 0.02$ radian, where link beam clear length, $e \geq 2.6 M_p / V_p$

– $\theta \leq$ 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

$$\Delta = 3\Delta_s,$$

(CSA 27.7.5)

1 = bay length

- The beam strength outside the link is checked to be at least $1.30\,R_y$ times the beam force corresponding to the controlling link beam shear strength (CSA 27.7.9.2). The controlling link beam nominal shear strength is taken as follows:

$$V_r \le \left(\phi V_p' \min 2\phi M_p' / e \right)$$

(CSA 27.7.9.2)

The values of V_p' and M_p' are calculated following the procedure described previously (CSA 27.7.3). The correspondence between brace force and link beam force is obtained from the associated load cases that has the highest link beam force.

- Each diagonal brace and its end connections shall have a factored resistance to support axial force and moment produced by the strain-hardened link. The forces developed in the link shall be taken as $1.30 R_y$ times the nominal strength of the link.

- Columns shall be designed to resist the cumulative effect of yielding links, together with the gravity loads. The link forces shall be taken as $1.15 R_y$ times the nominal strength of the link, except that in the top two stories, the force shall be taken as $1.30 R_y$ times the nominal value. Column resistances shall satisfy the requirements of Clause 13.8, except that the interaction value shall not exceed 0.65 for the top column tier in the braced bay and 0.85 for all other columns in the braced bay. Column sections shall be Class 1 or Class 2 (CSA 27.7.12.2).

- All braces are checked to be at least Class 1 or Class 2 (CSA 27.7.10.1). If this criterion is not satisfied, the program issues an error message.

- All column members are checked to be at least Class 1 or 2. If these criteria are not satisfied, the program issues an error message (CSA 27.7.13.1).

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 dia-

phragm 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.

4.4.7 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 (CSA 27.9).

4.4.8 Conventional Moment Frame (MF)

For this framing system, the following additional requirements are checked or reported (CSA 27.11).

- This framing system is checked to be designed using a force reduction factor $R_d = 1.5$, $R_o = 1.3$.

- The primary framing members of the seismic load resisting system are designed to resist gravity loads combined with seismic loads multiplied by R_d when the specified short-period spectral acceleration ratio is greater than 0.45. This clause has not been implemented in the program yet. The user is required to check this clause independently.

4.4.9 Conventional Braced Frame (BF)

For this framing system, the following additional requirements are checked or reported (CSA 27.11).

- This framing system is checked to be designed using a force reduction factor $R_d = 1.5$, $R_o = 1.3$.

- The primary framing members of the seismic load resisting system are designed to resist gravity loads combined with seismic loads multiplied by R_d when the specified short-period spectral acceleration ratio is greater than 0.45. This clause has not been implemented in the program yet. The user is required to check this clause independently.

4.4.10   Cantilever Column

For this framing system, the following additional requirements are checked or reported (CSA 27.11.2).

- This framing system is checked to be designed using a force reduction factor $R_d = 1.0$, $R_o = 1.0$. Class 1 sections are to be designed using a force reduction factor $R_d = 1.5$, $R_o = 1.3$ (CSA 27.11.2(a)). This clause has not been implemented in the program yet. The user is required to check this clause independently.

- U_2 shall be less than or equal to 1.25. If this criterion is not satisfied, the program issues an error message.

4.5   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

4.5.1   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 (LD MRF, MD MRF and D MRF) only. No check is made for braced frames (LD CBF, MD CBF, EBF).

The program first evaluates the need for continuity plates. When the required strength P_{bf} exceeds the bearing resistance of the web B_r as appropriate, a continuity plate will be required. The program checks the following limit states.

- The factored bearing resistance of the column web against local yielding at the toe of the fillet is given as follows (CSA 14.3.2):

$$B_r = \phi_{bi}(t_{fb} + 10t_{fc})F_y t_w \qquad \text{End column} \qquad \text{(CSA 14.3.2(a)(i))}$$

$$B_r = \phi_{be}(t_{fb} + 4t_{fc})F_y t_w \qquad \text{Interior column} \qquad \text{(CSA 14.3.2(b)(i))}$$

where

$$\phi_{bi} = 0.80$$

$$\phi_{be} = 0.75$$

- The available strength of the column web against crippling is given as follows:

$$B_r = 1.45\phi_{bi}t_{wc}^2\sqrt{F_y E} \qquad\qquad \text{End column} \qquad\qquad \text{(CSA 14.3.2(a)(ii))}$$

$$B_r = 0.60\phi_{be}t_{wc}^2\sqrt{F_y E} \qquad\qquad \text{Interior column} \qquad\qquad \text{(CSA 14.3.2(b)(ii))}$$

where

$$\phi_{bi} = 0.80$$

$$\phi_{be} = 0.75$$

If any of the preceding conditions are not met, the program calculates the required continuity plate area as follows (CSA 14.4.2).

$$A_{cp} = \frac{\left(P_{bf} - B_r\right)}{\phi F_{cr}}, \qquad\qquad \text{End column}$$

$$A_{cp} = \frac{\left(P_{bf} - B_r\right)}{\phi F_{cr}}, \qquad\qquad \text{Interior column}$$

In the preceding expressions, F_{cr} is the factored axial compressive stress of the equivalent column related to the beam-column joint. F_{cr} is taken as follows:

$$F_{cr} = \phi F_y \left(1 + \lambda^{2n}\right)^{-\frac{1}{n}}, \qquad\qquad \text{(CSA 13.3.1, 14.4.2)}$$

where,

$$\phi \quad = \quad 0.90, \qquad\qquad\qquad\qquad\qquad\qquad\qquad\quad \text{(CSA 13.1)}$$

$$n \quad = \quad 1.0 \text{ (assumed conservatively)}, \qquad\qquad\qquad \text{(CSA 13.3.1)}$$

$$\lambda \quad = \quad \frac{Kl}{r\pi}\sqrt{\frac{f_y}{E}}, \qquad\qquad\qquad\qquad\qquad\qquad \text{(CSA 13.3.1)}$$

$$K \quad = \quad 0.75, \qquad\qquad\qquad\qquad\qquad\qquad\qquad\quad \text{(CSA 14.4.2)}$$

$$l \quad = \quad d_c - 2t_{fc}, \text{ and} \qquad\qquad\qquad\qquad\qquad\quad \text{(CSA 14.4.2)}$$

$$r \;=\; \min\{r_{33}, r_{22}\} \text{ of the equivalent column.}$$

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 column (CSA 14.4.2).

▪ The effective length is taken as $0.75h$, i.e., $K = 0.75$ and $l = h = d_c - 2t_{fc}$ (CSA 14.4.2).

▪ $\dfrac{Kl}{r}$ 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 (CSA 14.4.2). This is similar to a column capacity check. For this condition, the program calculates the required continuity plate area as follows:

$$A_{cp} = \frac{P_{bf}}{\phi F_{cr}} - 25t_{wc}^2, \qquad \text{if not at top story} \qquad \text{(CSA 14.4.2)}$$

$$A_{cp} = \frac{P_{bf}}{\phi F_{cr}} - 12t_{wc}^2, \qquad \text{if at top story} \qquad \text{(CSA 14.4.2)}$$

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 $A_{cp} \le 0$, 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:

$$t_{cp}^{\min} = 0.5t_{fb} \qquad \text{(borrowed from AISC J10.8)}$$

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

$$b_{cp}^{\min} = 2\left(\frac{b_{fp}}{3} - \frac{t_{wc}}{2}\right)$$ (borrowed from AISC J10.8)

▪ So that the minimum area is given by

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

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

In the preceding equations,

A_{cp} = Required continuity plate area

F_y = 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 $M_u/(d_b - t_{tb})$

B_r = Factored bearing resistance of web

t_{fb} = Beam flange thickness

t_{fc} = Column flange thickness

t_{wc} = Column web thickness

ϕ = Resistance factor

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 (LD MRF, MD MRF and D MRF) only. No checks are performed for braced frames (LD CBF, MD CBF, EBF).

Note that the code insists on designing the continuity plate to match with tested connection.

For Seismic cases,

$$P_{of} = 1.1R_y \cdot b_{tb} t_{fb} f_{yb} \text{ D MRF} \tag{CSA 27.2.5.2}$$

$$P_{fb} = 1.1R_y \cdot b_{fb} t_{fb} f_y \text{ MD MRF} \tag{CSA 27.3, 27.2.5.2}$$

$$P_{bf} = 1.1R_y \cdot b_{fb} t_{fb} f_y \text{ LD MRF} \tag{CSA 27.4.4.5, 27.4.4.1}$$

4.5.2 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 (LD MRF, MD MRF and D MRF) only. No check is made for braced frames (LD CBF, MD CBF, 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 resistance is determined as V_r as appropriate. Assuming that the effect of panel zone deformation on frame stability has not been considered in analysis, the shear resistance, V_r is determined as follows:

(a) When detailed in accordance to the following conditions:

    i. When the doubler plate is Plug welded.

    ii. When the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is equal to or greater than 0.55, and when the joint panel zone is designed such that the sum of panel zone depth and the width divided by panel zone thickness does not exceed 90 (CSA 27.2.4.3(a)).

    iii. The joint panel zone is designed to satisfy the width-to-thickness limit of CSA Clause 13.4.1.1(a), i.e., $d_{wc}/t_{wc} = 439\sqrt{k_v/F_y}$ (CSA 27.2.4.3(b)).

$$V_r = 0.55\phi_v d_{wc} t_p F_{yc} \left[1 + \frac{3 b_{fc} t_{fc}^2}{d_c d_{wb} d_{wc}} \right] \le 0.66\phi_v d_{wc} t_p F_y,$$

$$\text{(CSA 27.2.4.2.1)}$$

By using $V_p = V_r$, the required column panel zone thickness t_p is found as follows.

$$t_p = \frac{V_p}{0.55\phi_v d_{wc} t_p F_y \left[1 + \dfrac{3 b_{fc} t_{fc}^2}{d_c d_{wb} d_{wc}}\right]} \leq \frac{V_p}{0.66\phi_v d_{wc} t_p F_y}.$$

(b) When detailed in accordance to the following conditions:

i. Joint panel zone is designed to satisfy the width-to-thickness limit of CSA Clause 13.4.1.1(a) i.e., $d_{wc}/t_{wc} = 439\sqrt{k_v/F_y}$ (CSA27.2.4.3(b)).

$$V_r = 0.55\phi_v d_{wc} t_p F_y. \qquad\qquad \text{(CSA 27.2.4.2.2)}$$

By using $V_p = V_r$, the required column panel zone thickness t_p is found as follows.

$$t_p = \frac{V_p}{0.55\phi_v d_{wc} t_p F_y}$$

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 the 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

V_c = Column shear in the column above

n_b = Number of beams connecting to a column

d_n = Overall depth of the n-th beam connecting to a column

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

d_c = Overall depth of the column

M_{bn} = Factored beam moment from corresponding loading combination

V_r = Shear resistance of a panel

P_r = Required axial resistance

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 D MRF (CSA 27.2.4.2), MD MRF (CSA 27.3, 27.2.4.2), LD MRF (CSA 27.4.3, 27.2.4.2) only. No further check/design is performed for other types of frames.

The doubler plate and the column web should satisfy the slenderness criteria (CSA 27.2.4.3(a)). 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 the 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.

4.5.3　Weak Beam/Strong Column Measure

Only for Ductile Moment Resisting Frames (D MRF) and Moderately Ductile Moment Resisting Frames (MD MRF) with seismic design, the code requires that the sum of column flexure strengths at a joint be more than the sum of beam flexure strengths (CSA 27.2.3.2, 27.3). The column flexure strength should reflect the presence of the axial force present in the column. The beam flexural strength should reflect the 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.

When the specified short-period spectral acceleration ratio is greater than 0.55 or the building is greater than 60 m in height, columns shall satisfy the requirements of Class 27.2.3.2; however, when applying Clause 27.2.3.2, the term $1.1R_yM_{pb}$ may be replaced by M_{pb}. In addition, the beams shall be designed so that for each story, the story shear resistance is not less than that of the story above.

The preceding equation can be written as follows:

When $I_eF_aS_a(0.2) > 0.55$ or when the building height is greater than 60 m, columns are designed for:

$$\sum M_{rc}' \geq \sum \left(M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right)$$
(CSA 27.2.3.2)

where

$\sum M_{rc}'$ = sum of the column factored flexural resistance at the intersection of the beam and column centerlines.

$$\sum M_{rc}' = 1.18\phi M_{pc} \left(1 - \frac{C_f}{\phi C_y} \right) \leq \phi M_{pc}$$
(CSA 27.2.3.2)

M_{pc} =　nominal plastic moment resistance of the column

C_f =　results from summation of V_h acting at this level and above

M_{pb} =　nominal plastic moment resistance of the beam

V_h =   shear acting at that plastic hinge location where $1.1 R_y M_{pb}$ is reached at the beam hinge locations

x =   distance from the center of a beam plastic hinge to the column face, which shall correspond to that of the assembly used to demonstrate performance in accordance with Clause 27.2.5.1.

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

$$R_{\text{maj}} = \frac{\sum\limits_{n=1}^{n_b} M_{pbn}^* \cos\theta_n}{M_{pcax}^* + M_{pcbx}^*}.$$

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

$$R_{\text{min}} = \frac{\sum\limits_{n=1}^{n_b} M_{pbn}^* \sin\theta_n}{M_{pcay}^* + M_{pcby}^*},$$

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 the column above the story level

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

$$M^*_{pc} = M'_{rc} = 1.18\phi\left(1 - \frac{C_f}{\phi C_y}\right)M_{pc} \leq \phi M_{pc}$$ (CSA 27.2.3.2, 27.3)

The plastic moment capacities of the beams are amplified for potential increase in capacity for strain hardening as follows:

For D MRF and MD MRI

$$M^*_{pb} = M'_{rb} = 1.1R_y M_{pb} + V_h\left(x + \frac{d_c}{2}\right)$$ (CSA 27.2.3.2, 27.3)

For LD MRF

$$M^*_{pb} = M_b + V_h\left(x + a_c\right)$$ (CSA 27.4.2.2)

M'_{rc} = Factored plastic moment resistance of the column

C_f = Results from summation of V_h acting at this level and above

M'_{rb} = Amplified plastic moment resistance of the beam

x = Distance from the center of a beam plastic hinge to the column face

V_h = Shear acting at plastic hinge location when $1.1R_y M_{pb}$ is reached at the hinge location.

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.

4.5.4 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.

▪ For D MRF, MD MRF and LD MRF, 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, i.e., $1.1R_yZF_y$ (CSA 27.2.5.2, 27.3, 27.4.4.2(a)). The connection shear for the development of the full plastic moment capacity of beam is as follows:

$$V_f = \frac{R_dR_o}{1.3}EL + V_{DL} + V_{LL} \qquad \text{(CSA 27.2.5.2, 27.3, 27.4.4.2(b))}$$

where,

$\quad V_f \quad = \quad$ Shear force corresponding to END I or END J of the beam

$\quad EL \quad = \quad$ Maximum anticipated seismic load

$\quad L_h \quad = \quad$ Clear length of the beam

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

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

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.

4.5.5   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.

Brace axial forces for seismic designs are evaluated for braced frames (LD CBF, MD CBF, EBF) only. No special checks are performed for moment frames (DMRF, MD MRF, LD MRF).

▪ For MD CBF and LD CBF, the bracing connection force is taken as the minimum of the two values (CSA 27.5.4.2(a)):

a. The expected yield strength in tension of the bracing member, determined as $T_y F_y A_g$ (CSA 27.5.4.2(a)) and 1.2 times the probable compressive resistance of the bracing members, determined as C_r/ϕ.

▪ 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 $1.30R_y$ times the nominal strength of the link (CSA 27.7.9.2).

The maximum connection force from the load combinations is determined for all of the regular load combinations.

Chapter 5
Design Output

5.1   Overview

The program has the capacity to create design output in four major ways – graphical display, file output, tabular display, and member specific detailed design information.

The graphical display includes input and output design information for members visible in the active window; the display can be sent directly to a printer or saved to a file. The file output includes both summary and detail design data that can be saved in RTF, HTML and plain text formats. The tabular display output includes both summary and detail design data that can be displayed or saved in many formats, including Excel, Access, RTF, HTML and plain text. The member specific detailed design information shows the details of the calculation.

The following sections describe some of the typical graphical display, file output, tabular display output, and member specific detailed design information. Some of the design information is very specific to the chosen steel design code. This manual addresses "CSA S16-14" design code related output information only.

5.2 Display Design Information on the Model

The graphical display of design output includes input and output design information for all steel frame members that are visible in the active window. The graphical output can be produced in color or in gray-scaled screen display. The active screen display can be sent directly to the printer or saved to a file in several formats.

Input and output design information for the CSA S16-14code includes the following.

Table 5-1 Graphical Display of Design Information

Design Input Information	Design Output Information
▪ Design sections	▪ P-M stress ratio values with members color-coded based on the ratio
▪ Design type	
▪ Live load reduction factors	▪ P-M colors and shear stress ratio values
▪ Unbraced length ratios, *L*-factors, for major and minor direction of bending, and for lateral-torsional buckling	▪ P-M ratio colors and no values
▪ Effective length factor for sway condition, *K*-factors, for major and minor directions of bending	▪ Identify the P-M failure
	▪ Identify the shear failure
▪ Effective length factors, K_z, for lateral-torsional buckling	▪ Identify all failures
▪ ω_1 factors for major and minor directions of bending	
▪ ω_2 factors for major and minor directions of bending	
▪ U_1 factors for major and minor directions of bending	
▪ U_2 factors for major and minor directions of bending	
▪ R_d factors	
▪ R_o factors	
▪ Yield stress, F_y	
▪ Nominal axial capacities	
▪ Nominal bending capacities	
▪ Nominal shear capacities	

Note that only one of the listed items can be displayed on the model at a time.

Use the **Design menu > Steel Frame Design > Display Design Info** command in SAP2000/ETABS and **Advanced > Frame Design > Steel > Display Design Info** command in CSiBridge to plot design input and output values directly on the model. The Display Steel Design Results form shown in Figures 5-1 and 5-2 will display. Choose the Design Output or Design Input option. One item can be selected from the drop-down list. For example, the P-M interaction ratios can be displayed by choosing the Design Output option and selecting P-M Ratio Colors & Values from the drop-down list. Click the **OK** button to display the longitudinal reinforcing in the active window. A typical graphical display of longitudinal reinforcing is shown in Figure 5-3.
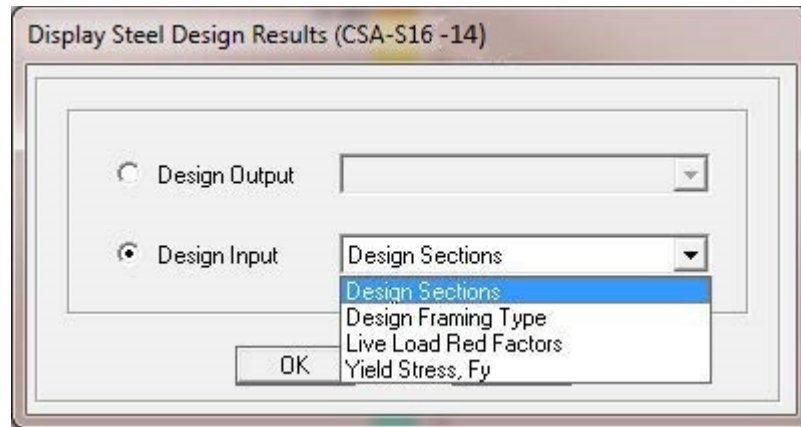
*Figure 5-1 Choice of design input data for display on the model
in the active window*
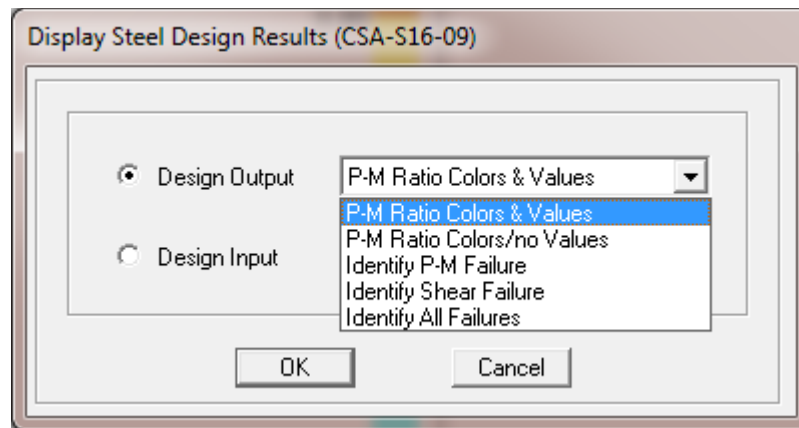
Figure 5-2 Choice of design output data for display on the model in the active window
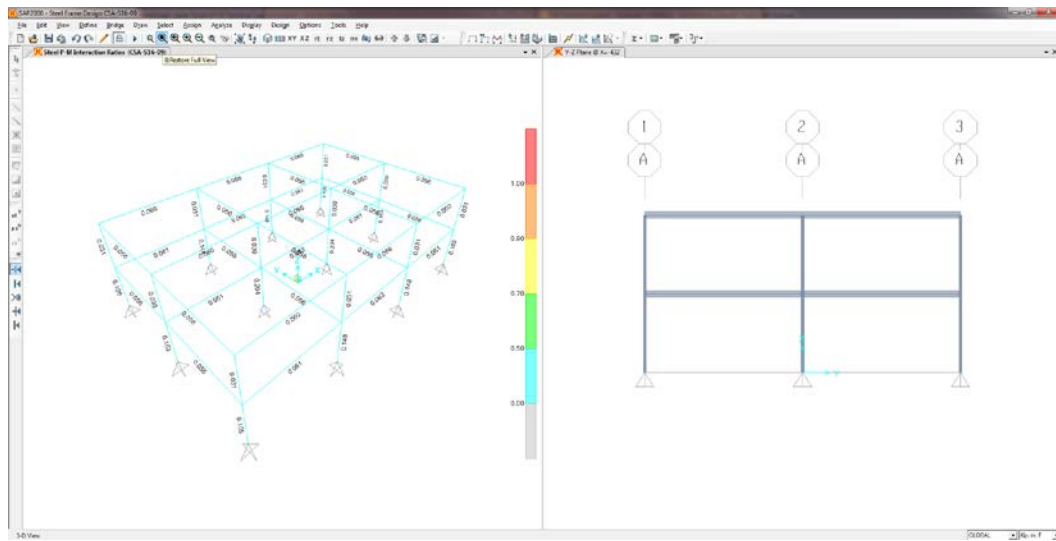
Figure 5-3 A typical graphical display

The graphics can be displayed in 3D or 2D mode. The standard view transformations are available for all steel design information displays. Several buttons on the toolbar can be used to switch between 3D and 2D views. Alternatively, click the **View menu** > **Set 3D View** or **Set 2D View** commands in SAP2000/ETABS and the **Home > View > Set 3D View** or **Set 2D View** commands in CSiBridge.

The onscreen graphical display can be sent to a printer using any of the following commands. Use the **File menu > Print Graphics** command in SAP2000/ETABS and the **Orb > Print > Print Graphics** command in CSiBridge to print the active window. To capture the graphical display in a file for printing through another application, use the **File menu > Capture Enhanced Metafile** command in SAP2000/ETABS and the **Orb > Pictures > Metafile** command in CSiBridge to create an .emf file, or use the **File menu > Capture Picture** command in SAP2000/ETABS and the **Orb > Pictures > Bitmap*** command in CSiBridge to create a bitmap (.bmp) file. Create a screen capture of the active window using the **Alt+ Print Screen** keyboard keys or create a screen capture of the entire window using the **Ctrl** + **Print Screen** keyboard keys. Then use the **Ctrl+V** keyboard keys to paste the saved image into Paint or other graphical program.

By default the graphical displays are in color. It may be advantageous to view or present the display in gray-scale graphics or using a white background. Use the **Options menu > Color** command to set these options.

5.3   Display Design Information in Tables

In addition to model definition and analysis results, the design information for all steel frame members or for only selected members can be displayed in tabular spreadsheet format. Currently, the program generates design summary data, PMM design details and shear design details. The tabular spreadsheet output can be displayed by selecting the **Display menu > Show Tables** command in SAP2000/ETABS and the **Home > Display > Show Tables** command in CSiBridge to access the **Choose Tables for Display** form, an example of which is shown in Figure 5-4. That form can be used to choose which tables or sets of tables are to be displayed.

The names of the tables are displayed in a tree structure, which can be collapsed or expanded by clicking on an item in the tree. Click on the small check boxes preceding the items to select those tables for display. If a branch of the tree is selected, all of the tables under that branch are selected. The selected set of tables can be saved as a Named Set using the **Save Named Set** button. This named set can be used in the future for quick selection. If one or more frame members are selected on the structural model before accessing the Choose Tables for Display form, the Selection Only check box will be checked when the form

displays, and the program will display information for the selected members only; uncheck the check box to display information for all applicable "unselected" members in the model. If the Show Unformatted check box is checked, the numbers will be displayed unformatted instead of being displayed using a limited number of decimal digits. The unformatted option provides higher precision output that can then be copied into other programs.

Use the other buttons in the form to tailor the data display. For example, click the **Select Load Cases** button to specify which load cases are to be included in the display of model definition data; click the **Select Analysis Case** and **Modify/Show Options** to specify which analysis cases are to be included and how analysis results are displayed.
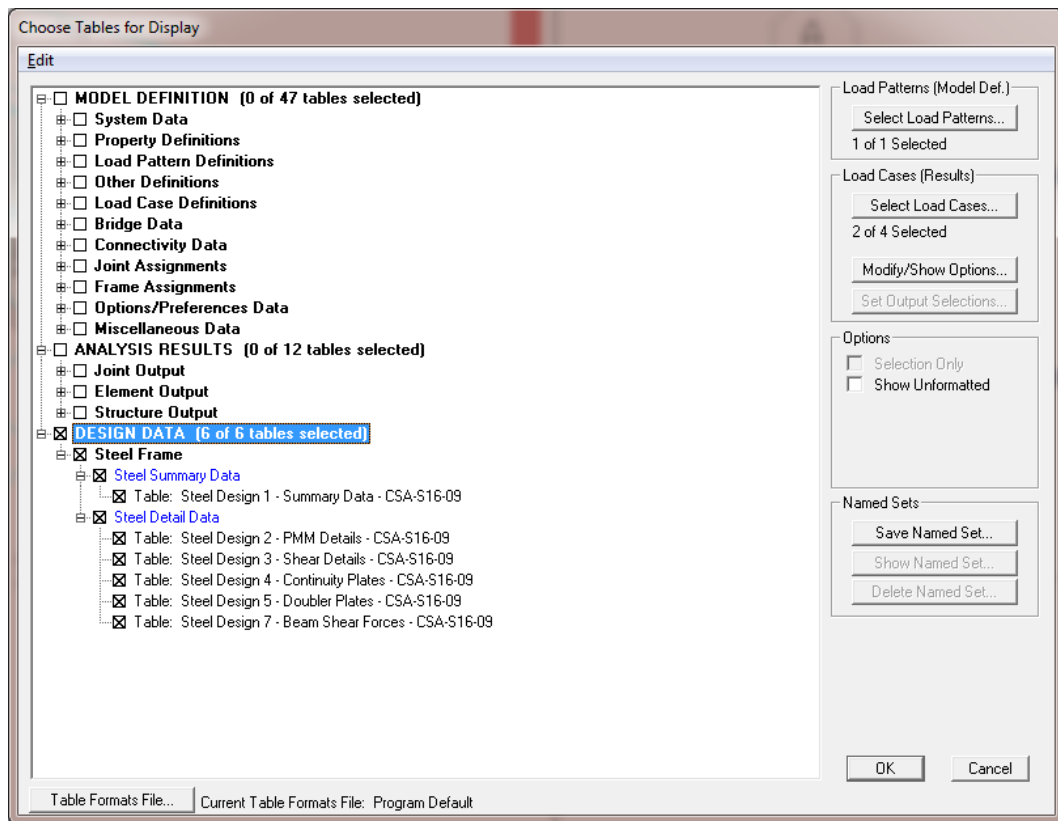
Figure 5-4 Choice of design data tables for tabular display

After selecting all of the tables for steel frame design and the display options, click the **OK** button to display a form showing one of the selected design tables,

with a drop-down list in the upper right-hand corner of the form that can be used to select other tables for display. A typical design table is shown in Figure 5-5.

Use the scroll bars on the bottom and right side of the tables to scroll right and left or up and down if portions of the data table can not be displayed in the form's display area. The columns can be resized by clicking the left mouse button on the separator of the headers, holding down the left mouse button and then dragging the mouse to the left or right. Reset the column widths to their default values by selecting the **View menu > Reset Default Column Widths** command on the form. The table can be split into two or more tables by clicking on the small black rectangular area near the bottom-left corner of the table, holding down the left mouse button, and then dragging the mouse button to the left or right. Repeat this process to add more splits. Use the split and horizontal scroll bar to put two columns side by side for easier comparison. The splits can be removed by selecting the **View menu > Remove Splits** command on the form. Alternatively, remove the split by clicking, holding and dragging the left mouse button to merge the split key to its original location.

Figure 5-5 A typical tabular display of design data

Select multiple consecutive columns by putting the cursor on the header, holding down the mouse button, and then dragging the mouse button left or right. Alternatively, depress the Shift key and click the left mouse button to select a range of columns.

The current table (i.e., the table in the active window) can be copied to the Window clipboard and them pasted into popular programs such as Excel, Access, .rtf., .html, and plain text formats. Many other features of the design tables are left for the user to discover by using the program.

5.4 Display Detailed Member Specific Information

The program has the capability to display the design details for a specific member. The information includes member identification, shape name, section properties, design combination name, design combination forces, and other design input data to check the design results. The information also includes stress ratios for P-M-M and other interactions, demand/capacity ratios from shear, nominal strengths, design factors such as ω_1, ω_2, U_1, U_2, and so forth. The design details are displayed in a summary form and also are displayed for a specific load combination and for a specific station of a frame member.

When the design results are displayed on the model in the active window, the detailed design information can be accessed by **right clicking** on the desired frame member to display the Steel Stress Check Information form. Alternatively, click the **Design menu > Steel Frame Design > Interactive Steel Frame Design** command in SAP2000/ETABS and the **Advanced > Frame Design > Steel > Interactive Steel Frame Design** command in CSiBridge and then right click on the frame member. An example of that form is shown in Figure 5-6.

Figure 5-6 A typical member specific steel stress check information summary

> Note: It should be noted that two design processes are available in CSiBridge: superstructure design (on the Design/Rating tab, including the Optimize command that can be used to interactively design a steel I-beam superstructure) and design of the individual elements comprising the structure (the **Advanced > Frame Design** commands). This manual addresses the second design process.

The Steel Stress Check Information form identifies the frame members and the analysis and design section, and includes a display area of mostly demand/capacity ratio data and buttons that access forms that provide further details about the selected frame member. The display area reports the load combinations, the stress check stations, the P-M-M interaction ratio along with its axial and flexural components, and the shear stress ratios. The load combination is reported by its name, while the station is reported by its location, which is measured from the I-end of the column. The number of reported line items in the text box is equal to the number of design combinations multiplied by the number of stations. Only one line item is highlighted in blue when the form first displays. That item highlights the largest demand/capacity ratio from P-M-M,

major and minor shear or any other considered interaction ratio, unless a line item(s) has design overstress or an error. In that case, the item with the overstress or error will be selected and highlighted. If many line items are overstressed or have an error, the last among all such line items will be selected and highlighted.

The stress check information is always reported for the design section. If the member is assigned an individual section, the analysis and design section are always the same. If the member is assigned an Auto Select Section (a list of sections), the analysis and design section can be different, unless the design has converged.

The **Overwrites** and **Details** buttons near the bottom of the Steel Stress Check Information form can be used to access the Steel Frame Design Overwrites form, and the Steel Stress Check Data form, which displays detailed information about the selected frame element. While the latter form displays information in a non-editable format, the Overwrites form display the overwrite data in editable format. This allows the user to enter an interactive mode of design.

▪ **Overwrites** button. Click this button to access the Steel Frame Design Overwrites form. Use that form to make revisions to the steel frame design overwrites and then immediately review the new design results as a summary using the Steel Stress Check Information form, or in detail by clicking the **Details** button to access the Steel Stress Check Data form. Clicking the **OK** button on the Steel Frame Design Overwrites form temporarily saves any changes. To make the changes permanent, click the **OK** button on the Steel Stress Check Information form. To disregard the changes, click the **Cancel** button on the Steel Stress Check Information form. An example of an Overwrites form is shown in Figure 5-7.

Figure 5-7 A typical member specific Steel Frame Design Overwrites form

▪ **Details** button. Click this button to access the Steel Stress Check Data form. Use the form to review all of the design details for the highlighted item. An example of a Steel Stress Check Data form is shown in Figure 5-8. The information includes the member ID, load combo and station identifications, steel design sections, section properties, design combination forces, stress ratios for P-M-M and other interactions, stress ratios for shear, nominal strengths, and design factors such as, ω_1, ω_2, U_1, U_2, and so forth. Values that are not applicable are reported as N/A. Similarly, N/C and N/N indicate an item is "Not Calculated" and "Not Needed."

Before clicking the Details button on the form shown in Figure 5-6, highlight an item for the desired design station and design load combination in the Steel Stress Check Information display area by clicking on the line. The data subsequently displayed will relate to the highlighted item. By default, the most

critical line item is selected when the form first displays, as described previously.

To increase or decrease the width of the Steel Stress Check Information form (Figure 5-8), put the cursor near the right edge of the form, click the left mouse button, and drag the mouse cursor towards the left or right. Similarly, the height of the form can be increased or decreased.
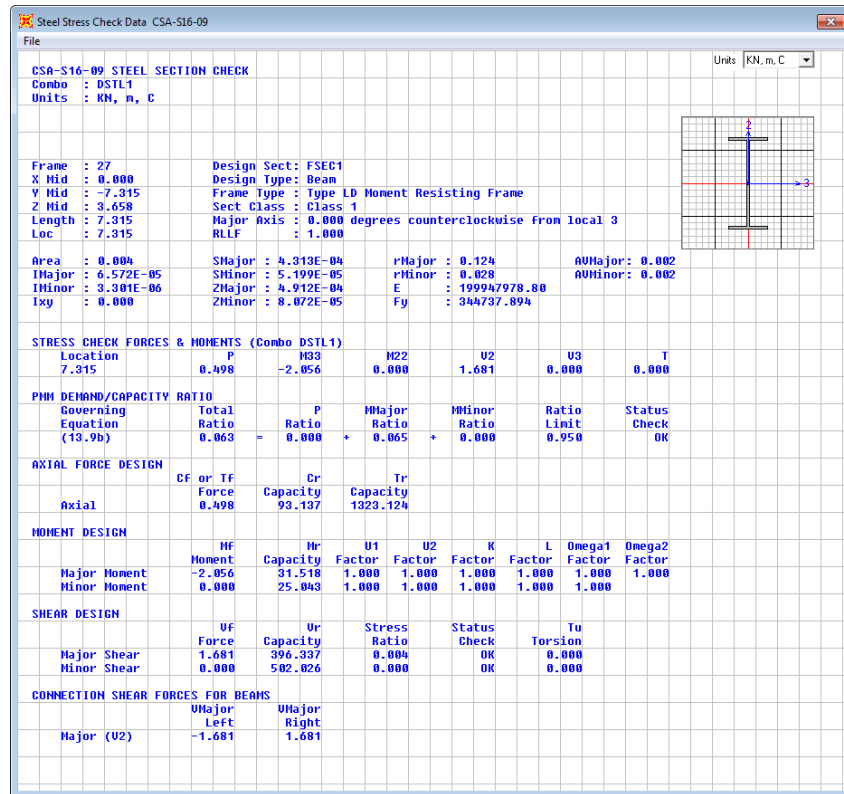

Figure 5-8 A typical Steel Stress Check Information form

The text in the form can be dragged in any direction by positioning the cursor in the middle of the form, and then clicking the left mouse button and dragging the text in the desired direction. Similarly, the graphical display of the column section can also be dragged in any direction.

Use the Units drop-down list in the upper right-hand corner of the form to change the units used to display the data. Data displayed on the form can be

sent directly to the printer by selecting the **File menu > Print** command on the form. The program allows limited page setup options using the **Print Setup** command on the **File** menu on the form.

5.5 Save or Print Design Information as Tables

In addition to model definition and analysis results, the design information for all steel frame members or for selected frames only can be saved in tabular format. Currently for CSA S16-14code, the program saves design summary data, PMM design details, and shear design details.

Save the file output by selecting the **File menu > Print Tables** command in SAP2000/ETABS and the **Orb > Print > Print Tables** command in CSiBridge and selecting the type of information (e.g., Input, Analysis Output, Summary Report and so on); the Print Design Tables form will then display. If one or more structural members were selected before the Print Design form is accessed, the Selection Only check box will be checked and the program will save the data for the selected members only; uncheck the check box to save the data for all appropriate "unselected" members. After the design output is saved in a text file, the file can be opened using any text file editor/viewer. However, SAP2000 and ETABS provide the **File menu > Display Input/Output Text Files** command that can be used to display the text file in Window's Wordpad.

Using the preceding process without checking the Print to File check box will send the specified information directly to the printer.

5.6 Error Messages and Warnings

Error messages and warnings may be displayed in the steel frame design output. Those messages and warnings are assumed to be self explanatory.

Bibliography

CSA, 2014. CSA S16-14 – Design of Steel Structures; Canadian Standards Association, 5060 Spectrum Way, Suite 100, Mississauga, Ontario, Canada L4W5N6.

CSI, 2014. CSI Lateral Load Manual. Computers and Structures, Inc., 2014 1646 N. California Blvd., Walnut Creek, CA 94596.

NBCC 2010. National Building Code of Canada, National Research Council of Canada, Ottawa, Ontario.