COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

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Integrated Building Design Software

Steel Frame Design Manual

IS 800:2007

CSi

Steel Frame Design Manual

IS 800:2007

For ETABS® 2016

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Contents

4   Special Seismic Provisions

5 Design Output

Bibliography

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 "Indian IS 800:2007." This option covers the "Indian Standard – General Construction in Steel – Code of Practice, Third Revision" (IS 2007).

The design codes supported under "Indian IS 800:2007" 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 "Indian IS 800:2007" steel frame design option. Chapter 2 addresses prerequisites related to modeling and analysis for a successful design in accordance with IS 800:2007. Chapter 3 provides detailed descriptions of the specific requirements as implemented in "Indian IS 800:2007." Chapter 4 provides detailed descriptions of the specific requirements for seismic loading as required by the specification IS 800:2007.

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 IS 800:2007 code is identified with the prefix "**IS**."

- Reference to the IS 1893 (Part 1):2002 code is identified with the prefix "**IS 1893**."

2.1   Check and Design Capability

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

To check adequacy of a section, the program checks the demand/capacity ("D/C") ratios at a predefined number of stations for each design load combination. It calculates the envelope of the D/C ratios. It also checks the other requirements on a pass or fail basis. If the capacity ratio remains less than or equal to the D/C ratio limit, which is a number close to 1.0, and if the section passes all the special requirements, the section is considered to be adequate;

otherwise the section is considered to have 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 the 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 sections will be changed.

To choose the optimal section, the user must first define a list of steel sections, the *Auto Select* sections list. The user must next assign this list, in the same manner as any other section assignment, to the frame members to be optimized. The program will use the median section by weight when doing the initial analysis. 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 an 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 run 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 calcu-

lated 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 design capacities are calculated for compression, tension, bending and shear based on the equations provided later in this manual. For flexure, the design capacities 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 design capacities 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 design capacities to determine D/C ratios. In either case, design codes typically require that the ratios not exceed a value of one. A D/C 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 load cases for which the structure needs to be checked. The program creates a number of default design load combinations for steel frame design. Users can add their own design combinations as well as modify or delete the program default design load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. The scale factors are applied to the forces and moments from the load cases to form the factored design forces and moments for each design load combination.

For normal loading conditions involving static dead load (DL), live load (LL), wind load (WL), earthquake load (EL), 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 combinations with negative factors for response spectrum load cases are not required because the program automatically takes the minima to be the negative of the maxima response when preparing the sub-combinations described previously.

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

2.6   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.
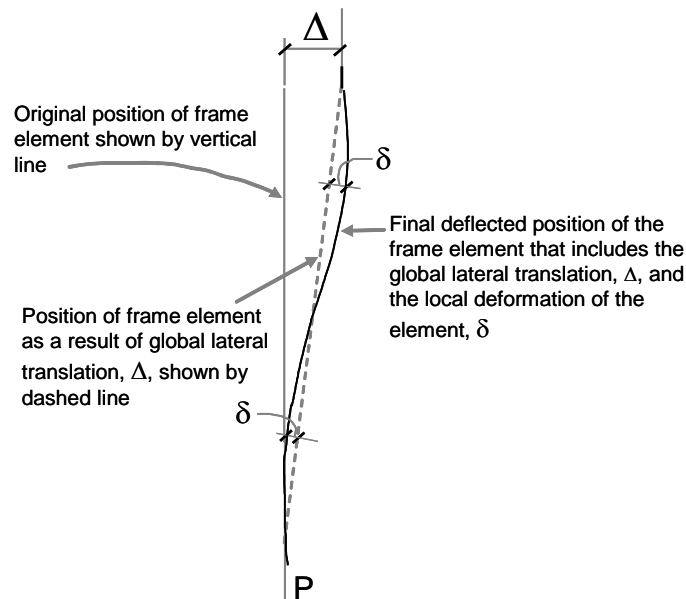
Figure 2-1 P-Δ and P-δ effects

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 strengths are common in current design codes and have the following general form:

$$M_{CAP} = \delta_b \left(M_{nt} + \delta_s M_{lt} \right)$$

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)

δ_s = Unitless amplification factor multiplying M_{lt}

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

A rigorous second order analysis (IS 4.4.2, 4.4.3.1) or the amplification of first order analysis results to estimate the effect of second order effects (IS 4.1.1, 4.4.3.1, Annex B) is required. The program has the capability of performing both. In the first case, the required strengths are determined directly from the analysis results without any amplification factor δ_s (i.e., δ_s is equal to 1). However, these amplification factors can always be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

To properly capture the *P-δ* effect in a finite element analysis, each element, especially column elements, must be broken into multiple finite elements, which is not really desired for other reasons. Although a single element per member can capture the *P-δ* effect to some extent, the program considers that

inadequate. The program thus uses the δ_b 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 (IS 4.3.6). 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 automatically included in the default design load combinations that include lateral loads. Moreover, 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.

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).
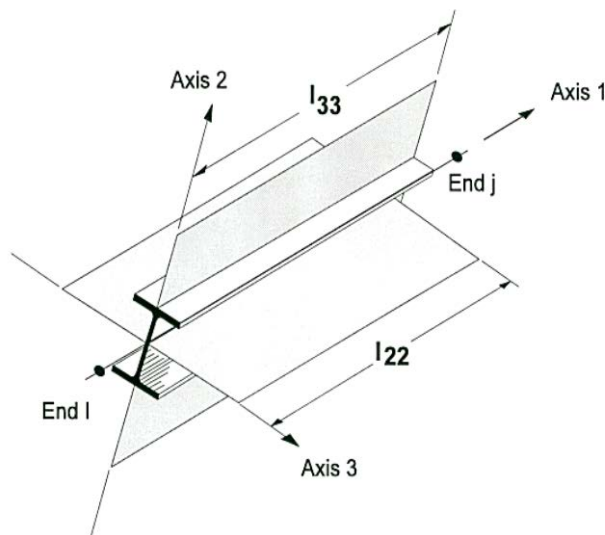

Figure 2-2 Unsupported lengths l_{33} and l_{22}

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

It is possible for the unsupported length of a frame object to be evaluated by the program as greater than the corresponding member length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In 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.

The unsupported length for minor direction bending for lateral-torsional buckling also can be defined more precisely by using "precise" bracing points in the Lateral Bracing option, which is accessed using the **Design menu > Steel Frame Design > Lateral Bracing** command. This allows the user to define the lateral bracing of the top, bottom, or both flanges. The bracing can be a point brace or continuous bracing.

The program calculates the unbraced length to determine axial capacity based on the limit state of flexural buckling from this definition. Any bracing at the top or bottom, or both, is considered enough for flexural buckling in the minor direction. While checking moment capacity for the limit state of lateral-torsional buckling (LTB) at a station the program dynamically calculates the bracing points on the compression flange at the left and at the right of the check station considering the sign of the moment diagram. This definition affects only the unbraced lengths for minor direction bending (L_{22}) and lateral-torsional buckling (L_{LTB}). This "exact" method of bracing definition does not allow the user to define unbraced lengths for major direction bending (L_{33}).

There are three sources of unbraced length ratio: (1) "automatic" calculation, (2) "precise" bracing definition, (3) overwrites, with increasing priority in considerations. "Automatic" calculation of the unbraced length is based on member connectivity considering only the members that have been entered into the model. This misses the tiny bracing members. However, such automatically calculated bracing lengths are load combo (moment diagram) independent.

This can be reported easily. Similarly, the overwritten values are load combo independent. This allows the program to report the overwritten unbraced length easily. However, if the member has a "precise" bracing definition, the unbraced length can be different at different stations of the member along the length. Also it can be load combo dependent. Thus, when the unbraced length is reported in the detailed design info, it is reported perfectly considering all three sources as needed. However, when reporting unbraced length on the model shown in the active window, the program-reported value comes from "automatic" calculation or from the overwrites if the user has overwritten it.

2.9   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, $N_{b,Rd}$, through an empirical column curve that accounts for geometric imperfections, distributed yielding, and residual stresses present in the cross-section.

There are two types of *K*-factors in the "Indian IS 800:2007" code. The first type of *K*-factor is used for calculating the Euler axial capacity assuming that all of the member joints are held in place, i.e., no lateral translation is allowed. The resulting axial capacity is used in calculation of the *k* factor. This *K*-factor is named as K_1 in this document. This K_1 factor is always less than 1 and is not calculated. By default the program uses the value of 1 for K_1. The program allows the user to overwrite K_1 on a member-by-member basis.

The other *K*-factor is used for calculating the Euler axial capacity assuming that all the member joints are free to sway, i.e., lateral translation is allowed. The resulting axial capacity is used in calculating P_d. This *K*-factor is named K_2 in this document. This K_2 is always greater than 1 if the frame is a sway frame. The program calculates the K_2 factor automatically based on sway condition. The program also allows the user to overwrite K_2 factors on a member-by-member basis. If the frame is not really a sway frame, the user should overwrite the K_2 factors.

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

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

The rest of this section is dedicated to the determination of K_2 factors.

The K-factor algorithm has been developed for building-type structures, where the columns are vertical and the beams are horizontal, and the behavior is basically that of a moment-resisting frame for which the K-factor calculation is relatively complex. For the purpose of calculating K-factors, the objects are identified as columns, beams, 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:

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

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

where the x and y subscripts correspond to the global X and Y directions and the c and b subscripts refer to column and beam. The local 2-2 and 3-3 terms EI_{22}/L_{22} and EI_{33}/L_{33} are rotated to give components along the global X and Y directions to form the $(EI/L)_x$ and $(EI/L)_y$ values. The correlation factor for effective flexural stiffness, c, is taken as 1, disregarding the code recommendation (IS Table 35). 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 β-values for END-I and the END-J of the member are calculated about the 2-2 and 3-3 directions as follows:

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

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

Finally, if β^I and β^J are known for a particular direction, the column K-factors for the corresponding direction is calculated from the following relationship for α:

$$K = \left[\frac{1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2}{1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2} \right]^{0.5}$$

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

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

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

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

The effective length factor K_1 of column in non-sway frames is given by

$$K = \frac{\left[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2\right]}{\left[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2\right]}$$

If the member is assigned with a framing type of sway frame, K_1 is used for K_z and K_y calculation, and K_2 is used for P_d calculation. If the member is assigned with a framing type of Braced frame, K_1 is used for both K_z and K_y calculation and also for P_d calculation.

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

4. C_1 factor: The logic is similar to that for the C_m factor.

5. K_z, K_y, and K_{LT} factors: This factor amplifies the factored moments for the P-δ effect. In its expression, there are the C_m factor and the Euler Buckling capacity P_e. If the user keeps the unbraced length ratios (L_{33} and L_{22}) and the K-factors (K_{33} and $K_{,22}$) correct, the K_z, K_y, and K_{LT} factors would be correct.

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 K_z, K_y, and K_{LT} factors will be picked up correctly. The factors C_m and C_1 will be picked up correctly if there is no intermediate bracing point. The calculated C_m and C_1 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_e-factor are correct, the effect of K_z, K_y, and K_{LT} factors will be picked up correctly. The factors C_m and C_1 will be picked up correctly if the member is broken at the bracing points. The calculated C_m and C_1 factors will be conservative if the member is not broken at the bracing points.

2.11   Supported Framing Types

The code (IS 800:2007) recognizes the following types of framing systems.

Framing Type	References
SMF (Special Moment Frame)	IS 12.11
OMF (Ordinary Moment Frame)	IS 12.10
SCBF (Special Concentrically Braced Frame)	IS 12.7
OCBF (Ordinary Concentrically Braced Frame)	IS 12.8
EBF (Eccentrically Braced Frame)*	IS 12.9
Secondary or Gravity only	IS 12.1

*No special provision for EBF is programmed.

2.12   Continuity Plates

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

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

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

- The steel beam frames in a direction that is at an angle to both of the principal axes.

To achieve a beam/column moment connection, continuity plates, such as shown in Figure 2-3, are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.

For connection conditions described in the last two bullet items, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam.
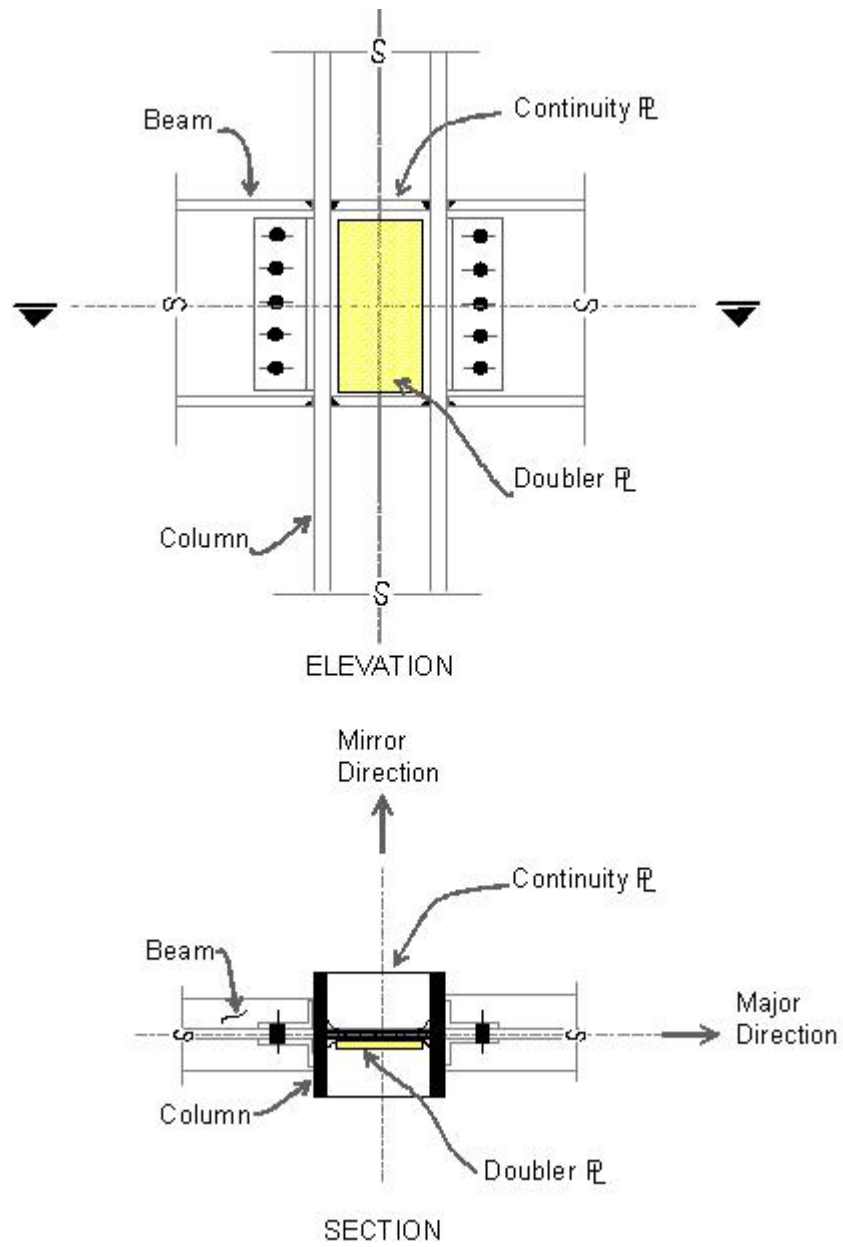
Figure 2-3 Doubler Plates and Continuity Plates

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

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

2.13 Doubler Plates

One aspect of the design of a steel framing system is an evaluation of the shear forces that exist in the region of the beam-column intersection known as the panel zone. Shear stresses seldom control the design of a beam or column member. However, in a moment resisting frame, the shear stress in the beam-column joint can be critical, especially in framing systems when the column is subjected to major direction bending and the web of the column resists the joint shear forces. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear stresses are seldom critical, and the program does therefore not investigate this condition.

Shear stresses in the panel zone due to major direction bending in the column may require additional plates to be welded onto the column web, depending 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. 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*. Overwriting the design procedure can be accessed from the **Design menu > Overwrite Frame Design Procedure** command.

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 procedure 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. Change the design procedure by selecting the member(s) and clicking the **Design menu > Overwrite Frame Design Procedure** command. 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, first use the **Design menu > Steel Frame Design > View/Revise Preference** 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 design of the structure, 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.

 Currently, this feature is absent in SAP2000.

2.17   Choice of Units

English as well as SI and MKS metric units can be used for input. The codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. For example, the IS 800:2007 code is published in millimeter-Newton-second units. By default, all equations and descriptions presented in this manual correspond to millimeter-Newton-second units. However, any system of units can be used to define and design a structure in the program.

Chapter 3
Design using IS 800:2007

This chapter provides a detailed description of the algorithms used by the programs in the design/check of structures in accordance with "IS 800:2007 – Indian Standard: General Construction in Steel – Code of Practice" (IS 2007). The implementation covers load combinations from "IS 800:2007," which is described in *Section 3.4 Design Loading Combinations* in this chapter. The loading based on "IS 1893:2002" and "IS 875:1987" has been described in a separate document entitled "Automated Lateral Loads Manual" (IS 2007; CSI 2007).

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

- Reference to the IS 800:2007 code is identified with the prefix "**IS**."

- Reference to the IS 1893 (Part 1):2002 code is identified with the prefix "**IS 1893**."

- Reference to the IS 875 (Part 3):1997 code is identified with the prefix "**IS 875**."

- Reference to the ANSI/AISC 360-05 code (specialist literature) is identified with the prefix "**AISC**."

3.1 Notations

The following table provides a list of the symbols used in this manual, along with a short description. Where possible, the symbol from the design code is used in this manual.

A	Cross-sectional area, mm^2
A_e	Effective section area, mm^2
A_g	Gross cross-sectional area, mm^2
A_n	Net effective area of cross-section, mm^2
A_v	Shear area, mm^2
A_{v2}, A_{v3}	Major and minor shear areas, mm^2
A_w	Web area, mm^2
b	Nominal dimension of plate in a section, mm
b_f	Flange width, mm
C_1	Modification factor for elastic critical moment
C_{mLT}	Equivalent uniform moment factor for lateral-torsional buckling
C_{my}	Equivalent uniform moment factor for minor bending
C_{mz}	Equivalent uniform moment factor for major bending
D	Outside diameter of pipes, mm
d	Overall depth or clear depth of member depending on section type, mm
E	Modulus of elasticity, N/mm^2
f_{cc}	Euler buckling stress $= \dfrac{\pi^2 E}{\left(KL/r \right)^2}$
f_{cd}	Design compressive stress, N/mm^2
f_u	Steel ultimate strength, N/mm^2

f_y	Steel yield strength, N/mm^2
f_{yw}	Steel yield strength of the web, N/mm^2
G	Shear modulus, N/mm^2
h	Overall depth of the section, mm
h_w	Web height, mm
I	Moment of inertia, mm^4
I_{22}, I_y	Minor moment of inertia, mm^4
I_{33}, I_z	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_{LT}	Moment magnification factor for lateral-torsional buckling
K_{LTB}	Effective length K-factor for lateral-torsional buckling
K_y	Moment magnification factor for minor axis bending moment
K_z	Moment magnification factor for major axis bending moment
L	Laterally unbraced length of member, mm
L_{22}, L_{33}	Laterally unbraced length of member for major and minor axes bending, mm
L_{LT}	Effective buckling length, mm
L_{LTB}	Unbraced length for lateral-torsional buckling, mm
M	Design factored bending moment, N-mm
M_d	Design bending resistance, N-mm
M_{dv}	Reduced design bending resistance accounting for shear, N-mm
M_y, M_{22}	Design bending moment about minor axis bending, N-mm
M_z, M_{33}	Design bending moment about major axis bending, N-mm
N	Factored applied axial force (tension, T, or compression, P)

N_{cr}	Elastic critical force, N
N_d	Design strength in tension, T_d, or compression strength due to yielding given by $N_d = A_g \, f_y / \gamma_{m0}$
P	Design axial compression force, N
P_d	Design compressive strength, N
r	Radius of gyration, mm
$r_{33}, \, r_{22}$	Radii of gyration in the major and minor directions, mm
r_z	Least radius of gyration for analysis sections, mm
t	Plate thickness, mm
T_d	Design tension resistance, N
T_{dg}	Design tensile strength due to yielding of cross-section, N
T_{dn}	Design ultimate tensil strength due to rupture, N
t_f	Flange thickness, mm
t_w	Web thickness, mm
V	Design factored shear force, N
V_{cr}	Design shear buckling resistance, N
V_d	Design shear strength, N
Z_e	Elastic section modulus, mm^3
Z_{e33}, Z_{e22}	Major and minor elastic section moduli, mm^3
Z_p	Plastic modulus, mm^3
Z_{p33}, Z_{p22}	Major and minor plastic section moduli, mm^3
ϕ	Value for calculating the reduction factor χ
ϕ_{LT}	Value for calculating the reduction factor χ_{LT}
α	Imperfection factor for axial compression
α_{LT}	Imperfection factor for lateral-torsional buckling
$\alpha_1, \, \alpha_2$	Coefficients used in an interaction equation (IS 9.3.1.1)

β	Reduction factor of moment capacity to account for shear forces
β_b	Moment capacity adjustment factor for appropriate class of section
χ	Stress reduction factor for buckling
χ_{LT}	Stress reduction factor for lateral-torsional buckling
ε	Coefficient dependent on f_y, $\sqrt{250/f_y}$
γ_{M0}	Partial factor for the resistance of cross-sections and buckling
γ_{M1}	Partial factor for the resistance governed by ultimate stress
λ	Non-dimensional slenderness, $\dfrac{KL}{r\pi}\sqrt{\dfrac{f_y}{E}}$ for flexural buckling
$\lambda_p, \lambda_r, \lambda_s$	Slenderness parameters for classification
λ_{LT}	Non-dimensional slenderness for lateral-torsional buckling
λ_w	Non-dimensional web slenderness ratio
ψ	Ratio of moments in a segment

3.2 Design Preferences

The steel frame design preferences are basic assignments that apply to all steel frame members. Table 3-1 lists steel frame design preferences for "Indian IS 800:2007." 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	AISC360-05/ IBC 2006	The selected design code. Subsequent design is based on this selected code.

Table 3-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
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 multivalued case is present in the combo.
Framing Type	SMF, OMF, SCBF, OCBF, EBF, Secondary	SMF	This item is used for ductility considerations in the design.
Importance Factor	> 0	1.0	This is related to seismic design. This depends on the functional use of the structure, hazardous consequences of its failure, post-earthquake functional needs, and so on (IS 1893 6.4.2).
Seismic Zone	Zone I, II, III, IV, V	Zone V	This affects seismic intensity. It depends on physical location of the structure (IS 1893 6.4.2, Table 2, Annex E).
Consider P-Delta Done?	Yes/No		It affects K-factor calculation. If P-Δ analysis is completed, K is taken as if the structure is braced.
GammaM0	> 0	1.1	Partial factor for the resistance of cross-sections and buckling.
GammaM1	> 0	1.25	Partial factor for the resistance of members to ultimate stress.
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).

Table 3-1: Steel Frame Design Preferences

Item	Possible Values	Default Value	Description
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.
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 "Indian IS 800:2007." 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.

Many of the items on the Overwrites form are similar to those found on the Preferences form.

Table 3-2 Steel Frame Design Overwrites for "Indian IS 800:2007"

Item	Possible Values	Default Value	Description
Current Design Section	Any defined steel section	Analysis section	The design section for the selected frame object. When this Overwrite is applied, any previous auto select section assigned to the frame object is removed.
Fame Type	SMF, OMF, SCBF, OCBF, EBF, Secondary	From Preferences	This item is used for ductility considerations in the design.
Section Class	Plastic, Compact, Semicompact, Slender	Program determined	Section class to be used. This can be "Class 1," "Class 2," "Class 3," or "Class 4." It determines the capacity of the sections and the interaction equations to be used. If not overwritten, it is calculated based on Table 2 of the code.
Column Buckling Curve (z-z)	a, b, c, or d	Program determined	Column buckling curve to be used for flexural buckling about the major axis. This determines the axial compression capacity for buckling about the major axis. It determines the imperfection factors for buckling curve. If not overwritten, it is taken from Table 10 of the code.
Column Buckling Curve (y-y)	a, b, c, or d	Program determined	Column buckling curve to be used for flexural buckling about the minor axis. This determines the axial compression capacity for buckling about the minor axis. It determines the imperfection factors for buckling curve. If not overwritten, it is taken from Table 10 of the code.
Is Rolled Section?	Yes./No	Program determined	It might affect the column buckling curve determination.
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), or 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.

Table 3-2 Steel Frame Design Overwrites for "Indian IS 800:2007"

Item	Possible Values	Default Value	Description
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.
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)	≥ 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.

Table 3-2 Steel Frame Design Overwrites for "Indian IS 800:2007"

Item	Possible Values	Default Value	Description
Effective Length Factor Braced (K1 Major)	≥ 0	1.0	Effective length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used calculating K_y, K_z and K_{LT} factors.
Effective Length Factor Braced (K1 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 calculating K_y, K_z and K_{LT} factors.
Effective Length Factor Sway (K2 Major)	≥ 0	Calculated	Effective length factor for buckling about the frame object major axis assuming that the frame is not braced at the joints against sidesway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always 1, regardless of any other values specified in the Overwrites. The factor is used for axial compression capacity.
Effective Length Factor Sway (K2 Minor)	≥ 0	Calculated	Effective length factor for buckling about the frame object minor axis assuming that the frame is not braced at the joints against sidesway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always 1, regardless of any other values specified in the Overwrites. The factor is used for axial compression capacity.
Effective Length Factor (K LTB)	≥ 0	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.
Bending Coefficient (C1)	≥ 0	Program Determined	Unitless factor; C_1 is used in determining the critical buckling moment, M_{cr}, for arbitrary moment diagram. Inputting zero means the value is program determined.

Table 3-2 Steel Frame Design Overwrites for "Indian IS 800:2007"

Item	Possible Values	Default Value	Description
Uniform Moment Factor (Cmz)	≥ 0	Calculated	Unitless factor; Cm for major axis bending is used in determining the interaction ratio. Inputting zero means the value is program determined.
Uniform Moment Factor (Cmy)	≥ 0	Calculated	Unitless factor; Cm for minor axis bending is used in determining the interaction ratio. Inputting zero means the value is program determined.
Uniform Moment Factor (CmLT)	≥ 0	Program Determined	Unitless factor; CmLT for lateral torsional buckling is used in determining the interaction ratio. Inputting zero means the value is program determined.
Moment Coefficient (Kz)	≥ 0	Program Determined	Unitless moment magnification factor for major axis bending moment. Specifying zero means the value is program determined.
Moment Coefficient (Ky)	≥ 0	Program Determined	Unitless moment magnification factor for minor axis bending moment. Specifying zero means the value is program determined.
Moment Coefficient (K_LT)	≥ 0	Program Determined	Unitless moment magnification factor for lateral-torsional buckling. Specifying zero means the value is program determined.
Yield Stress, Fy	≥ 0	From Material	Material yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Compressive Capacity, Pd	≥ 0	Calculated	Design axial compressive capacity. Specifying zero means the value is program determined.
Tensile Capacity, Td	≥ 0	Calculated	Design axial tensile capacity. Specifying zero means the value is program determined.
Major Bending Capacity, Mdz	≥ 0	Calculated	Design bending moment capacity in major axis bending. Specifying zero means the value is program determined.
Minor Bending Capacity, Mdy	≥ 0	Calculated	Design bending moment capacity in minor axis bending. Specifying zero means the value is program determined.
Critical Buckling Moment, Mcr			Elastic critical moment calculated in accordance with Annex E.
Major Shear Capacity, Vdy	≥ 0	Calculated	Design shear capacity force for major direction shear. Specifying zero means the value is program determined.
Minor Shear Capacity, Vdz	≥ 0	Calculated	Design shear capacity force for minor direction shear. Specifying zero means the value is program determined.

Table 3-2 Steel Frame Design Overwrites for "Indian IS 800:2007"

Item	Possible Values	Default Value	Description
Demand/Capacity Ratio Limit	≥ 0	Calculated	The demand/capacity ratio limit to be used for acceptability. Demand/capacity ratios that are less than or equal to this value are considered acceptable. 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 IS 800:2007 code, the following default design combinations are generated by the program (IS 3.5.1, 5.3.3, Table 4):

1.5DL	(IS 3.5.1, 5.3.3, Table 4, Line 1)
1.5DL + 1.5L	(IS 3.5.1, 5.3.3, Table 4, Line 2)
1.5DL $\pm$ 1.5WL	(IS 3.5.1, 5.3.3, Table 4, Line 4)
0.9DL $\pm$ 1.5WL	(IS 3.5.1, 5.3.3, Table 4, Line 4)
1.2D + 1.2WL + 1.2W	(IS 3.5.1, 5.3.3, Table 4, Line 3)
1.2D + 1.2L $\pm$ 0.6W	(IS 3.5.1, 5.3.3, Table 4, Line 2)
1.5D $\pm$ 1.5E	(IS 3.5.1, 5.3.3, Table 4, Line 4)
0.9D $\pm$ 1.5E	(IS 3.5.1, 5.3.3, Table 4, Line 4)
1.2D + 1.2L $\pm$ 1.2E	(IS 3.5.1, 5.3.3, Table 4, Line 3)

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.

Live load reduction factors can be applied to the member forces of the live load case on a member-by-member basis to reduce the contribution of the live load to the factored loading. However, such a live load case must be specified as type Reducible Live Load.

3.5 Design Strength

The design strength for steel shapes is obtained by dividing the ultimate strength of the shape by a partial factor of safety, γ_m. The values of γ_m used in the program are as follows:

Partial safety factor for buckling and yielding, $\gamma_{m0} = 1.10$ (IS 5.4.1, Table 5)

Partial safety factor for ultimate stress $\gamma_{m1} = 1.25$.   (IS 5.4.1, Table 5)

Although not recommended, the program allows them to be overwritten in the preferences. If they are overwritten, the program uses them consistently.

3.6 Classification of Sections for Local Buckling

For the determination of the design strengths for axial compression and flexure, the sections are classified as Plastic (Class 1), Compact (Class 2), Semi-Compact (Class 3), or Slender (Class 4). The program classifies the individual sections in accordance with Table 3-3 (IS 3.7.2, Table 2). As specified in that table, a section is classified as Class 1, Class 2, Class 3 or Class 4 as applicable.

For a section to qualify as Plastic (Class 1), its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression

elements must not exceed the limiting width-thickness ratios λ_p from Table 2 of the code. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r, the section is Compact (Class 2). If the width-thickness ratio of any element exceeds λ_r but does not exceed λ_s, the section is Semi-Compact (Class 3). If the width-thickness ratio of any element exceed λ_s, the section is considered Slender (Class 4). The expressions of λ_p, λ_r, and λ_s, as implemented in the program, are reported in Table 3-1 (IS 3.7.2, Table 2).

The table uses the variables h, b_f, t_f, t_w, b, t, D, d, and so on. The variables b, d, D and t are explained in the respective figures inside the table. The variables b_f, t_f, h, and t_w are explained in Figure 3-1.

For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows (IS Table 2, Note 3):

(a) For flanges of I-shaped members and tees, the width b is the clear distance between the lateral support and the free edge.

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

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

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

Refer to Table 3-3 (IS Table 2) for the graphic representation of unstiffened element dimensions.

For stiffened (internal) elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows (IS Table 2):

(a) For webs of rolled or formed sections, d is the clear distance between flanges less the fillet or corner radius at each flange.

(b) For webs of built-up sections, d is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used.

(c) For flanges of rectangular hollow structural sections (HSS), the width b is the clear distance between webs. For webs of rectangular HSS, d is the clear distance between the flanges.

Table 3-3   Limiting Width-Thickness Ratios of Compression Elements for Classification Sections

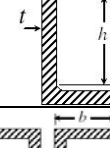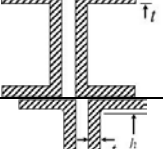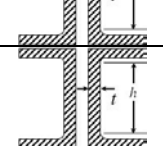| Section Type	Description of Element	Example	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element		
				Plastic Class 1 (λ_p)	Compact Class 2 (λ_r)	Semi-Compact Class 3 (λ_s)
Doubly Symmetric I-Shape	Flexural compression of flanges of rolled I-Shapes		b/t_f	9.4ε	10.5ε	15.7ε
	Flexural compression in flanges of built-up I-Shapes		b/t_f	8.4ε	9.4ε	13.6ε
	Axial only compression in flanges of rolled I-Shapes		b/t_f	9.4ε	10.5ε	15.7ε
	Axial only compression in flanges of built-up I-Shapes		b/t_f	8.4ε	9.4ε	13.6ε
	Flexure in web		d/t_w	$\dfrac{84\varepsilon}{1+r_1} \geq 42\varepsilon$	if $r \leq 0$ $\dfrac{105\varepsilon}{1+r_1}$ if $r > 0$ $\dfrac{105\varepsilon}{1+1.5r_1} \geq 42\varepsilon$	$\dfrac{126\varepsilon}{1+2r_2} \geq 42\varepsilon$
	Web in axial only compression		d/t_w	NA	NA	42ε

Table 3-3 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections
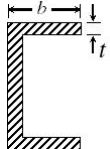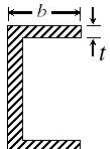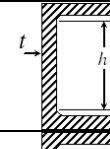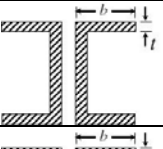
Section Type	Description of Element	Example	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element		
				Plastic Class 1 (λ_p)	Compact Class 2 (λ_r)	Semi-Compact Class 3 (λ_s)
Channel	Flexural compression in flanges		b/t_f	9.4ε	10.5ε	15.7ε
	Axial only compression in flanges		b/t_f	9.4ε	10.5ε	15.7ε
	Flexure in web		d/t_w	42ε	42ε	42ε
	Web in axial only compression		d/t_w	42ε	42ε	42ε
Double Channel	Flexural compression in flanges		b/t_f	9.4ε	10.5ε	15.7ε
	Axial only compression in flanges		b/t_f	9.4ε	10.5ε	15.7ε
	Flexure in web		d/t_w	42ε	42ε	42ε
	Web in axial only compression		d/t_w	42ε	42ε	42ε

Table 3-3   Limiting Width-Thickness Ratios of Compression Elements for Classification Sections
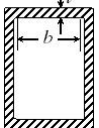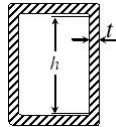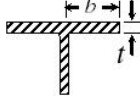
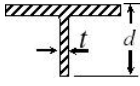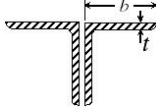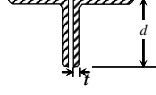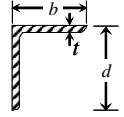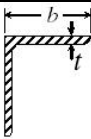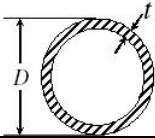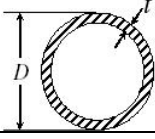| Section Type	Description of Element	Example	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element		
				Plastic Class 1 (λ_p)	Compact Class 2 (λ_r)	Semi-Compact Class 3 (λ_s)
Box	Flexural or axial compression of flanges under major axis bending		b/t_f	29.3ε	33.5ε	42ε
	Flexure in web		d/t_w	$\dfrac{84\varepsilon}{1+r_1} \geq 42\varepsilon$	if $r \leq 0$ $\dfrac{105\varepsilon}{1+r_1}$ if $r > 0$ $\dfrac{105\varepsilon}{1+1.5r_1} \geq 42\varepsilon$	$\dfrac{126\varepsilon}{1+2r_2} \geq 42\varepsilon$
T-Shape	Flexural or axial compression in flanges		b/t_f	9.4ε	10.5ε	15.7ε
	Compression in stems		d/t_w	8.4ε	9.4ε	18.9ε
Double Angle	Any type of compression in leg		b/t	9.4ε	10.5ε	15.7ε
	Any type of compression in leg		b/t	9.4ε	10.5ε	15.7ε
			$(b+d)/t$	———	———	25ε
Angle	Flexural compression in any leg		b/t	9.4ε	10.5ε	15.7ε
	Axial only compression in any leg		b/t	9.4ε	10.5ε	15.7ε
			$(b+d)/t$	———	———	25ε

Table 3-3   Limiting Width-Thickness Ratios of Compression Elements for Classification Sections

Section Type	Description of Element	Example	Width-Thickness Ratio, (λ)	Limiting Width-Thickness Ratios for Compression Element		
				Plastic Class 1 (λ_p)	Compact Class 2 (λ_r)	Semi-Compact Class 3 (λ_s)
Pipe	Flexural compression		D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
	Axial only compression		D/t	——	——	$88\varepsilon^2$
Round Bar	——	——	——	Assumed Compact		
Rectangular	——	——	——	Assumed Compact		
General	——	——	——	Assumed Compact		
SD Section	——	——	——	Assumed Compact		

Refer to Table 3-3 (IS Table 2) for the graphic representation of stiffened element dimensions.

The various parameters used in calculating the width-to-thickness ratio limits are defined as:

$$\varepsilon = \sqrt{250/f_y} \qquad \text{(IS Table 2 Notes)}$$

$$r_1 = -\frac{P}{\left(dt_w\right)f_y/\gamma_0} \qquad \text{(IS Table 2 Notes)}$$

$$r_2 = -\frac{P}{Af_y/\gamma_0} \qquad \text{(IS Table 2 Notes)}$$

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

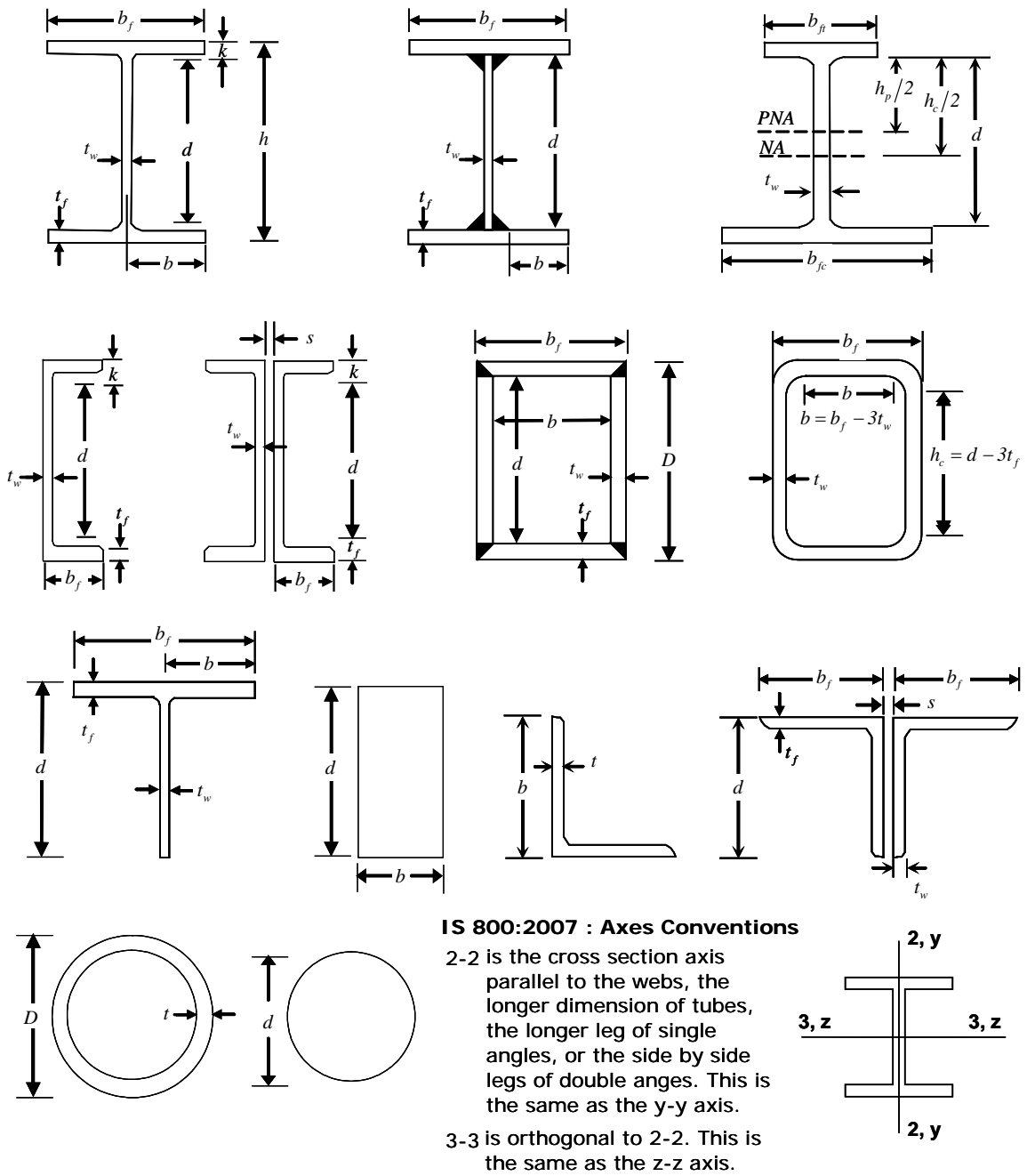**Stress check of Slender sections is beyond the scope of this program.**

IS 800:2007 : Axes Conventions

2-2 is the cross section axis parallel to the webs, the longer dimension of tubes, the longer leg of single angles, or the side by side legs of double anges. This is the same as the y-y axis.

3-3 is orthogonal to 2-2. This is the same as the z-z axis.

Figure 3-1 IS 800:2007 Definition of Geometric Properties

3.7   Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are P, M_z, M_y, V_y, and V_z, corresponding to factored values of the axial force, the major and minor moments, and shears respectively. These factored loads are calculated at each of the previously defined stations.

For slender compression members, the code recommends the use of a second order frame analysis, also called a P-Δ analysis, which includes the effect of sway deflections on the axial loads and moments in a frame. For an adequate and rational analysis, realistic moment curvature or moment rotation relationships should be used to provide accurate values of deflections and forces. The analysis also should include the effect of foundation rotation and sustained loads. Because of the complexity in the general second order analysis of frames, the code provides an approximate design method that takes into account the "amplification" due to lateral deflections in columns (IS 4.4, 9.3 Annex B).

Hence, when using the "Indian IS 800:2007" code, it is recommended that the user include P-Delta analysis, including consideration of method, tolerances, P-Delta load combinations and so on. With this analysis, the program can capture the lateral drift effect, i.e., the global effect or P-Δ effect, very nicely. But the program does not capture the local effect (P-δ effect) to its entirety because most often the column members are not meshed. To capture the local effects in columns, the program uses the approximate formula for amplified design moments as specified in the code (IS 9.3). Two major parameters in calculating the amplified design moments are the effective length factors for major and minor axis bending. The effective length factors for columns are computed using a code-specified procedure (IS 7.2.2, Annex D). If P-Δ analysis is not selected, the program calculates effective length factors, K, assuming the frame is a sway frame (sway unrestrained) (IS Annex D, Figure 28). However, if the P-Δ analysis is selected, the program assumes the member is prevented from further sway and assumes that the frame can be considered non-sway where $K < 1$ (IS Annex D, Figure 27). For more information on P-Δ and P-δ effects, please refer to Chapter 2 "Second Order P-Delta Effects." See "Effective Length Factors (K)" also in Chapter 2 concerning the determination of K factors.

3.8 Calculation of Factored Strengths

The design strengths in compression, tension, bending, and shear are computed for Plastic (Class 1), Compact (Class 2), and Semi-Compact (Class 3) sections in accordance with the text in the following sections. The design flexural strength for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I-Shape, Box, Channel, Double Channel, Circular, Pipe, T-Shape, and Double Angle sections, the principal axes coincide with their geometric axes. For the Single Angle sections, the principal axes are determined and all computations except shear are based on that.

For *all* sections, the design 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 design strengths for one or more of the members on the Steel Frame Overwrites form, those values will override the calculated values for those members. The specified capacities should be based on the principal axes of bending for flexure and on the geometric axes for shear.

3.8.1 Design Tensile Strength

This section applies to the members subject to axial tension.

The factored design tension, T, in the members shall satisfy the following requirements:

$$\frac{T}{T_d} \leq 1.0 \tag{IS 6.1}$$

where

T_d = design strength of the member.

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of the gross section, T_{dg} and the rupture strength of the critical section T_{dn}.

$$T_d = \min(T_{dg}, T_{dn}) \tag{IS 6.1}$$

The design strength of members under axial tension, T_{dg} as governed by the yielding gross section, is given by:

$$T_g = A_g f_y / \gamma_{m0} \qquad \text{(IS 6.2)}$$

f_y = yield stress of the material,

A_g = gross area of the cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding (IS Table 5)

The design strength in tension of the section, T_{dn}, as governed by rupture of net cross-sectional area, A_n, is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1} \qquad \text{(IS 6.3)}$$

where

A_n = net effective area of the member

f_u = ultimate stress of the material, and

γ_{m1} = partial safety factor for failure at ultimate stress (IS Table 5)

The values of A_g and A_n are defined as follows:

The gross cross-section area, A, is based on nominal dimensions, ignoring fastener holes and splice materials, and accounting for larger openings (IS 6.2).

The net effective cross-section area, A_n, is defined as the gross cross-section area, A, minus fastener holes and other openings. The net effective area, A_n, 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_n / A_g ratio must be modified using the steel frame design Overwrites to account for the effective area.

3.8.2 Design Compressive Strength

The design compressive strength is taken as the minimum of the two limit states: Flexural Buckling; and Torsional and Flexural-Torsional Buckling.

3.8.2.1 Flexural Buckling Limit State

The design compressive strength, P_d, of a member is given by:

$$P_d = A_e f_{cd} \qquad \text{(IS 7.1.2)}$$

where,

A_e = effective section area; for Class I, Class 2, and Class 3 cross-sections, it is taken as the gross sectional area (IS 7.3.2) and

f_{cd} = design compressive stress.

The design compressive stress, f_{cd}, or axially loaded compression members is calculated using the following equation:

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y/\gamma_{m0} \le f_y/\gamma_{m0} \qquad \text{(IS 7.1.2.1)}$$

where

$$\phi \;=\; 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] \qquad \text{(IS 7.1.2.1)}$$

λ = non-dimensional effective slenderness ratio

$$=\; \sqrt{f_y/f_{cc}} = \sqrt{f_y\left(KL\big/r\right)^2\big/\pi^2 E} \qquad \text{(IS 7.1.2.1)}$$

$$=\; \frac{KL}{r\pi}\sqrt{\frac{f_y}{E}} \qquad \text{(IS 7.1.2.1)}$$

f_{cc} = Euler buckling stress = $\dfrac{\pi^2 E}{\left(KL\big/r\right)^2}$ $\qquad \text{(IS 7.1.2.1)}$

where

KL/r = effective slenderness ratio or ratio of effective length, KL, to appropriate radius of gyration, r,

α = imperfection factor given in IS Table 7

$$\chi \quad = \quad \text{stress reduction factor}$$

$$= \quad \frac{1}{\left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right]} \qquad \text{(IS 7.1.2)}$$

$\gamma_{m0} \quad = \quad$ partial safety factor for material strength.

The effective slenderness ratio, KL/r, is given by:

$$\frac{KL}{r} = \max\left\{\frac{K_{33}L_{33}}{r_{33}}, \frac{K_{22}L_{22}}{r_{22}}\right\}$$

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

K is the effective length factor for flexural buckling. It can assume two values: K_{22} for buckling about the minor axis (y-y) and K_{33} for buckling about the major axis (z-z). L is the unbraced length of the member. It also can assume two values, L_{22} and L_{33}, for buckling about minor axis (y-y) and major axis (z-z), respectively. Please refer to Chapter 2 "Member Unsupported Length" and "Effective Length Factors (K)" for more details on L and K.

The imperfection factor, α, is defined in Table 3-4 (IS 7.1.1, 7.1.2.1, Table 7) based on the buckling class. The buckling class is defined in Table 3-5 (IS 7.1.2.2, Table 10) based on shapes, dimensional properties, and bending axis.

Table 3-4: Imperfection Factors (IS 7.1.1, 7.1.2.1, Table 7)

Buckling Class	a	b	c	d
Imperfection Factor, α	0.21	0.34	0.49	0.76

Table 3-5: Buckling Class of Cross-Section (IS 7.1.2.2, Table 10)

Section Shape	Limits		Axis	Buckling Class
Rolled I-Sections	$h/b > 1.2$	$t_f \leq 40$ mm	Major	a
			Minor	b
		$40 < t_f \leq 100$ mm	Major	b
			Minor	c
	$h/b \leq 1.2$	$t_f \leq 100$ mm	Major	b
			Minor	c
		$t_f > 100$ mm	Major	d
			Minor	d
Welded I-Sections	$t_f \leq 40$ mm		Major	b
			Minor	c
	$t_f > 40$ mm		Major	c
			Minor	d
Hollow Tube and Pipe Sections	hot rolled		any	a
Welded Box	$b/t_f > 30$ or $h/t_w > 30$		any	b
	$b/t_f < 30$ and $h/t_w < 30$		any	c
Channel, Tee, Double Channel, General, Solid Sections, Section Designer	none		any	c
Angle and Double Angle Sections	none		any	c

3.8.2.2 Torsional and Flexural-Torsional Buckling

Torsion and Flexural-Torsional Buckling are currently not covered in Indian IS 800:2007 code. However, program incorporates the relevant clauses from AISC 360-05 code to cover the torsion and flexural torsion buckling which are important part of design.

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

The design compressive strength, P_d, of a member with I, Box, Channel, Double Channel, Double Angle, and Angle sections is given by:

$$P_d = A_e f_{cd} \tag{IS 7.1.2}$$

where,

A_e = effective sectional area

$$f_{cd} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0} \qquad \text{(IS 7.1.2.1)}$$

$$\chi = \frac{1}{\left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right]} \qquad \text{(IS 7.1.2.1)}$$

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] \qquad \text{(IS 7.1.2.1)}$$

$$\lambda = \sqrt{f_y / F_e}$$

α = imperfection factor for minor axis bending

F_e is calculated from the following equations:

I-Shapes and Double Channel Sections

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

Channel Sections

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

Double Angle and T-Shapes

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

Single Angle Sections with Equal Legs

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

Single Angle Sections with Unequal Legs

F_e is the lowest root of the cubic equation.

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

(AISC E4-6)

In the preceding equations,

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

x_0, y_0 are the coordinates of the shear center with respect to the centroid, $x_0 = 0$ for Double Angle and T-Shaped members (y-axis symmetry)

$$\overline{r}_0 = \sqrt{x_o^2 + y_o^2 + \frac{I_{22} + I_{33}}{A_g}} = \text{polar radius of gyration about the shear center}$$

(AISC E4-7)

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\overline{r}^2}\right)$$

(AISC E4-8)

$$F_{e33} = \frac{\pi^2 E}{\left(K_{33}L_{33}/r_{33}\right)^2}$$

(AISC E4-9)

$$F_{e22} = \frac{\pi^2 E}{\left(K_{22}L_{22}/r_{22}\right)^2}$$

(AISC E4-10)

$$F_{ez} = \left[\frac{\pi^2 E C_w}{\left(K_z/L_{LT}\right)^2} + GJ\right]\frac{1}{A\overline{r}_0^2}$$

(AISC E 4-11)

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

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

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

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

L_{LT} is the effective length for torsional buckling, but it can be overwritten.

For Angle sections, the principal moment of inertia and radii of gyration are used for computing F_e. Also, the maximum value of KL, i.e., max ($K_{22}L_{22}$, $K_{33}L_{33}$), is used in place of $K_{22}L_{22}$ or $K_{33}L_{33}$ in calculating F_{e22} and F_{e33} in this case. The principal maximum value r_{max} is used for calculating F_{e33}, and the principal minimum value r_{min} is used in calculating F_{e22}.

3.8.3   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 (IS 800:2007).

The program uses the minimum of the flexure strength, M_d, computed from Cross-Section Strength and Lateral-Torsional Buckling Strength (IS 8.2).

3.8.3.1   Cross-Sectional Strength

The cross-sectional flexural strengths of the member are calculated assuming the member is laterally fully supported.

When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section, the design bending strength, M_d is taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0} \qquad\qquad \text{(IS 8.2.1.2)}$$

where

β_b = 1.0 for plastic and compact sections,

   = Z_e / Z_p for semi-compact sections,

Z_p = plastic section modulus of the cross-section,

Z_e = elastic section modulus of the cross-section,

f_y =   yield stress of the material,

γ_{m0} =   partial safety factor.

When the design shear force (factored), V exceeds $0.6V_d$, where V_d is the design shear strength of the cross-section, the design bending strength, M_d is taken as

$$M_d = M_{dv} \qquad\qquad\qquad\text{(IS 8.2.1.3)}$$

where

    M_{dv} = design bending strength under high shear.

When the factored shear force is less than or equal to 0.6 times the design shear strength ($V \leq 0.6\ V_d$), there is no reduction in moment capacity of the section necessary (IS 9.2.1). When the factored shear force is high ($V > 0.6\ V_d$), the moment capacity of the section is reduced to M_{dv} which is calculated as follows (IS 9.2.2):

    (a)   Plastic (Class 1) and Compact (Class 2) Sections

$$M_{dv} = (1-\beta)M_d + \beta M_{fd} \leq 1.2 Z_e f_y / \gamma_{m0} \qquad\text{(IS 9.2.2(a))}$$

where

$$\beta = \begin{cases} 0 & \text{if } V/V_d \leq 0.6 \\ \left(2V/V_d - 1\right)^2, & \text{if } V/V_d > 0.6 \end{cases} \qquad\text{(IS 9.2.2(a))}$$

     M_d =   plastic design moment of the whole section disregarding high shear force effect (IS 8.2.1.2) considering web buckling effects (IS 8.2.1.1),

     V =   factored applied shear force,

     V_d =   design shear strength as governed by web yielding or web buckling (IS 8.4.1, 8.4.2),

     M_{fd} =   plastic design strength of the area of cross-section excluding the shear area, considering partial safety factor γ_{m0},

     Z_e =   elastic section modulus of the entire section.

    M_{fd} is considered for I, Box, Channel, and Double Channel sec-

tions about the major axis bending and for only Box sections about the minor axis bending. For all other cases, M_{fd} is conservatively taken as zero.

(b)   Semi-Compact (Class 3) Section

$$M_{dv} = (1-\beta)M_d + \beta M_{fd} \leq Z_e f_y / \gamma_{m0} \qquad \text{(IS 9.2.2(b))}$$

where β, M_d, V, V_d, M_{fd}, and Z_e are the same as defined previously.

3.8.3.2   Lateral-Torsional Buckling Strength

The design bending strength of a laterally unsupported beam as governed by lateral-torsional buckling is given by

$$M_d = \beta_b Z_p f_{bd} \qquad \text{(IS 8.2.2)}$$

where

β_b $= 1.0$ for Plastic (Class1) and Compact (Class 2) sections

   $= Z_e / Z_p$ for Semi-Compact (Class 3) sections,

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fiber

f_{bd} = design bending compressive stress, obtained as given in the following

   $$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$$

χ_{LT} = bending stress reduction factor to account for lateral-torsional buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{ \phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2 \right]^{0.5} \right\}} \leq 1 \qquad \text{(IS 8.2.2)}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2 \right) + \lambda_{LT}^2 \right] \qquad \text{(IS 8.2.2)}$$

α_{LT} = the imperfection parameter. It is given by:

   $= 0.21$ for rolled steel section, and

= 0.49 for welded steel section.

The non-dimensional slenderness ratio, λ_{LT}, is given by:

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 Z_e f_y / M_{cr}} \qquad \text{(IS 8.2.2)}$$

where,

$M_{cr} = $ elastic critical moment calculated in accordance with Annex E.

The elastic critical moment, M_{cr}, is based on gross cross-section properties and taken as:

$$M_{cr} = C_1 \frac{\pi^2 E I_y}{L_{LT}} \left(\frac{I_w}{I_y} + \frac{L_{LT}^2 G I_t}{\pi^2 E I_y} \right)^{0.5} \qquad \text{(IS Annex E, 8.2.2.1)}$$

where I_y, I_w, and I_t are the minor axis inertia, warping constant, and torsion constant, respectively; L_{LT} is the effective unbraced length for the lateral-torsional buckling mode, and C_1 is defined as:

$$C_1 = 1.88 - 1.40\psi + 0.52\psi^2 \leq 2.7 \qquad \text{(EC3-1993 F1.1(6))}$$

where ψ is the ratio of the smaller to the larger end moments. The value of C_1 is also taken as 1.0 if the unbraced length is overwritten. The value of C_1 can be overwritten on a member-by-member basis.

C_1 should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. **The user should overwrite C_1 for cantilevers.** The program also defaults C_1 to 1.0 if the minor unbraced length, l_{22}, is redefined to be more than the length of the member by the user or the program, i.e., if the unbraced length is longer than the member length. The Overwrites can be used to change the value of C_1 for any member.

Here L_{LT} is the effective unbraced length for the lateral-torsional buckling mode.

$$L_{LT} = K_{LTB} L_{LTB}$$

where K_{LTB} is the effective length factor for the lateral-torsional buckling mode, and L_{LTB} is the unbraced length for the lateral-torsional buckling mode. For more details on these two factors, please refer to Chapter 2 of this manual.

The lateral-torsional buckling resistance of Channels, Double Channels, Tees, Angles, Double Angles, and I-sections is calculated as described previously.

Lateral-torsional buckling is not considered for Tubular, Box, or Solid sections. For General or Section Designer sections, the lateral-torsional buckling resistance is taken as the design elastic moment resistance.

3.8.4   Design 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_d, it is assumed that there is no intermediate stiffness used to enhance shear strength of a section (IS 8.4.2.2).

The design shear strength, V_d, is taken as

$$V_d = V_n / \gamma_{m0} \tag{IS 8.4}$$

where the nominal shear strength of a cross section, V_n, may be governed by plastic shear resistance (IS 8.4.1) or strength of the web, as governed by shear buckling (IS 8.4.2).

3.8.4.1   Plastic Shear Resistance

The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p \tag{IS 8.4.1}$$

where

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}} \tag{IS 8.4.1}$$

A_v = shear area, and

f_{yw} = yield strength of the web. $\hspace{2cm}$ (IS 8.4.1)

The shear area is calculated as given in the following (IS 8.4.1):

- I and Channel sections:

    Major Axis Bending

      Hot-Rolled — ht_w

      Welded — dt_w

    Minor Axis Bending

      Hot-Rolled or Welded — $2bt_f$

- Rectangular Hollow Sections of Uniform Thickness

    Loaded parallel to depth (h) — $Ah/(b+h)$

    Loaded parallel to width (b) — $Ab/(b+h)$

- Circular Hollow Tubes of Uniform Thickness — $2A/\pi$

- Plates and Solid Bars — A

where,

A	=	cross-section area,
b	=	overall breadth of a tubular section, breadth of I-section flanges,
d	=	clear depth of the web between flanges,
h	=	overall depth of the section,
t_f	=	thickness of the flange, and
t_w	=	thickness of the web.

For all other sections, the equivalent elastic shear area is used.

3.8.4.2 Resistance to Shear Buckling

Resistance to shear buckling is verified when $d/t_w > 67\varepsilon$, where $\varepsilon = \sqrt{250/f_y}$ (IS 8.4.2.1).

The nominal shear strength, V_n, of webs as governed by buckling is evaluated using the "simple post-critical method" assuming that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}, \qquad\qquad\qquad \text{(IS 8.4.2.2)}$$

where,

V_{cr} = shear force corresponding to web buckling,

    = $A_v \tau_b$,

τ_b = shear stress corresponding to web buckling, determined as follows:

- when $\lambda_w \leq 0.8$

$$\tau_b = f_{yw}/\sqrt{3}, \qquad\qquad \text{(IS 8.4.2.2)}$$

- when $0.8 < \lambda_w < 1.2$

$$\tau_b = \left[1 - 0.8\left(\lambda_w - 0.8\right)\right]\left(f_{yw}/\sqrt{3}\right), \qquad\qquad \text{(IS 8.4.2.2)}$$

- when $\lambda_w \geq 1.2$

$$\tau_b = f_{yw}/\left(\sqrt{3}\lambda_w^2\right), \qquad\qquad \text{(IS 8.4.2.2)}$$

λ_w = non-dimensional web slenderness ratio for shear buckling stress,

    = $\sqrt{f_{yw}/\left(\sqrt{3}\tau_{cr,e}\right)}, \qquad\qquad \text{(IS 8.4.2.2)}$

$\tau_{cr,e}$ = the elastic critical shear stress of the web,

$$= \frac{K_v \pi^2 E}{12\left(1 - \mu^2\right)\left[d/t_w\right]^2}, \qquad\qquad \text{(IS 8.4.2.2)}$$

μ = Poisson's ratio, and

K_v = 5.35 when transverse stiffeners are provided only at supports

$$= 4.0 + 5.35/(c/d)^2 \text{ for } c/d < 1.0$$

$$= 5.35 + 4.0/(c/d)^2 \text{ for } c/d \geq 1.0$$

where c, d are the spacing of the transverse stiffeners and the depth of the web, respectively. It is assumed that c is very large.

3.9   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.

3.9.1   Combined Axial Force and Bending Moment

Under combined axial force and bending moment, section strength as governed by material failure and member strength as governed by buckling failure shall be checked in accordance with IS 9.3.1 and IS 9.3.2, respectively.

3.9.1.1 Design for Cross-Section Strength

The combined effect of axial force and bending moments is checked in the same way whether the axial force is a tensile force or a compression force. There are minor exceptions that are noted in the relevant sections.

3.9.1.1.1 Plastic (Class 1) and Compact (Class 2) Cross-Sections

(a) For I, Rectangular Hollow, Circular Tube, and Solid Rectangular sections, the combined axial force (tension or compression) and bending moment is checked by taking the following summation of the utilization ratios for each force component as follows:

$$\left[\frac{M_y}{M_{ndy}}\right]^{\alpha_1} + \left[\frac{M_z}{M_{ndz}}\right]^{\alpha_2} \leq 1 \qquad\qquad \text{(IS 9.3.1.1(a))}$$

In the equation 9.3.1.1(a), there are two basic ratios — major axis bending (z) and minor axis bending (y) — and two factors α_1 and α_2 which determine the overall PMM ratio. The factors α_1 and α_2 in tern depend on the axial (n) ratio. Since α_1 and α_2 are often more than one, the overall ratio can be smaller than the individual component ratios when the component ratios are less than one. In this case the program reports either of the pure bending moment ratios or the axial ratio whichever governs as the PMM ratio as follows:

$$\frac{M_y}{M_{ndy}} \leq 1 \qquad\qquad \text{(IS 9.3.1.1(ay))}$$

$$\frac{M_z}{M_{ndz}} \leq 1 \qquad\qquad \text{(IS 9.3.1.1(az))}$$

$$\frac{N}{N_d} \leq 1 \qquad\qquad \text{(IS 9.3.1.1(an))}$$

where,

M_y, M_z = factored applied moments about the minor and major axis of the cross-section, respectively,

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone (IS 9.3.1.2),

N = factored applied axial force (tension, T, or compression, P),

N_d = design strength in tension, T_d, as obtained from IS 6 or in compression due to yielding given by $N_d = A_g\, f_y / \gamma_{m0}$,

M_{dy}, M_{dz} = design strength under corresponding moment acting alone (IS 8.2),

A_g = gross area of the cross-section,

γ_{m0} = partial factor of safety in yielding, and

α_1, α_2 = constants as given in IS 9.3.1.1, Table 17, and taken as follows:

– For I-sections and channels

$$\alpha_1 = 5n \geq 1 \qquad \text{(IS 9.3.1.1, Table 17)}$$

$$\alpha_2 = 2 \qquad \text{(IS 9.3.1.1, Table 17)}$$

– For Rectangular Hollow sections

$$\alpha_1 = \frac{1.66}{1 - 1.13n^2} \leq 6 \qquad \text{(IS 9.3.1.1, Table 17)}$$

$$\alpha_2 = \frac{1.66}{1 - 1.13n^2} \leq 6 \qquad \text{(IS 9.3.1.1, Table 17)}$$

– For Circular Tube sections

$$\alpha_1 = 2 \qquad \text{(IS 9.3.1.1, Table 17)}$$

$$\alpha_2 = 2 \qquad \text{(IS 9.3.1.1, Table 17)}$$

– For Solid Rectangular sections

$$\alpha_1 = 1.73 + 1.8n^3 \qquad \text{(IS 9.3.1.1, Table 17)}$$

$$\alpha_2 = 1.73 + 1.8n^3 \qquad \text{(IS 9.3.1.1, Table 17)}$$

In all of the preceding cases, n is taken as

$$n = \frac{N}{N_d} \qquad \text{(IS 9.3.1.1, Table 17)}$$

M_{ndy} and M_{ndz} are computed as follows:

– For Solid Rectangular sections

$$M_{ndz} = M_{dz}\left(1 - n^2\right) \qquad \text{(IS 9.3.1.2(a))}$$

$$M_{ndy} = M_{dy}\left(1 - n^2\right) \qquad \text{(IS 9.3.1.2(a))}$$

– For Welded I-sections

$$M_{ndz} = M_{dz}\left(\frac{1 - n}{1 - 0.5a}\right) \qquad \text{(IS 9.3.1.2 (b))}$$

$$M_{ndy} = \begin{cases} M_{dy}, & \text{for } n \le a, \\ M_{dy}\left[1 - \left(\dfrac{n - a}{1 - a}\right)^2\right], & \text{for } n > a \end{cases} \qquad \text{(IS 9.3.1.2 (b))}$$

where,

$$a = \frac{A - 2b_f t_f}{A} \le 0.5 \qquad \text{(IS 9.3.1.2 (b))}$$

– For Rolled I-sections

$$M_{ndz} = 1.11 M_{dz}\left(1 - n\right) \le M_{dz} \qquad \text{(IS 9.3.1.2 (c))}$$

$$M_{ndy} = \begin{cases} M_{dy}, & \text{if } n \le 0.2, \\ 1.56 M_{dy}\left(1 - n\right)\left(n + 0.6\right) \le M_{dy}, & \text{if } n > 0.2 \end{cases} \qquad \text{(IS 9.3.1.2 (c))}$$

– For Hollow Rectangular sections:

$$M_{ndz} = M_{dz}\left(\frac{1-n}{1-0.5a_w}\right) \leq M_{dz} \qquad \text{(IS 9.3.1.2(d))}$$

$$M_{ndy} = M_{dy}\left(\frac{1-n}{1-0.5a_f}\right) \leq M_{dy} \qquad \text{(IS 9.3.1.2(d))}$$

where

$$a_w = \frac{A - 2bt_f}{A} \leq 0.5 \qquad \text{(IS 9.3.1.2(d))}$$

$$a_f = \frac{A - 2ht_w}{A} \leq 0.5 \qquad \text{(IS 9.3.1.2(d))}$$

– For Hollow Circular Tube sections:

$$M_{ndz} = 1.04 M_{dz}\left(1 - n^{1.7}\right) \leq M_{dz} \qquad \text{(IS 9.3.1.2(e))}$$

$$M_{ndy} = 1.04 M_{dy}\left(1 - n^{1.7}\right) \leq M_{dy} \qquad \text{(IS 9.3.1.2(e))}$$

In all of the preceding cases, n is taken as

$$n = \frac{N}{N_d} = \begin{cases} \dfrac{T}{T_d}, & \text{if } N \text{ is tensile} \\[2mm] \dfrac{P}{P_d}, & \text{if } N \text{ is compressive} \end{cases} \qquad \text{(IS 9.3.1.2)}$$

(b) For Tee sections, combined axial force and bending is conservatively checked by taking a linear summation of the utilization ratios for each force component as:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.1.1(b))}$$

However, for this case the maximum longitudinal stress at three extreme points of the section are added with appropriate sign. That means that at the two extreme points on the flange, all three terms are added algebrai-

cally, whereas at the tip of the web, the minor axis bending term becomes zero.

(c)   For Channel, Double Channel, Double Angle, Angle, General, and Section Designer sections, combined axial force and bending is conservatively checked by taking a linear summation of the utilization ratios for each force component as:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.1.1(b))}$$

The equation number reported in the output for the interaction equations in this section is really the section number 9.3.1.1. There are two interaction equations in this section of the code of which none of them are numbered in the code itself. So the program named the equations as 9.3.1.1(a) and 9.3.1.1(b) to distinguish between them. Moreover, the program named the interaction equations 9.3.1.1(az), 9.3.1.1(ay) and 9.3.1.1(an) while checking the component ratios.

3.9.1.1.2   Semi-Compact (Class 3) Cross-Sections

For all shapes, with the exception noted in the following text, the combined axial force and bending is conservatively checked by taking the linear summation of the utilization ratios for each force component:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.1.3)}$$

For Doubly Symmetric sections, the preceding equation is a representation of the code-specified equation given here:

$$f_x \leq \frac{f_y}{\gamma_{M0}} \qquad \text{(IS 9.3.1.3)}$$

As an exception for Circular and Pipe sections, an SRSS (Square Root of Sum of Squares) combination is made first of the two bending components before adding the axial load component, instead of the single algebraic addition as implied by the interaction equations given by IS 9.3.1.3. The resulting interaction equation is given by the following:

$$\frac{N}{N_d} + \sqrt{\left(\frac{M_y}{M_{dy}}\right)^2 + \left(\frac{M_z}{M_{dz}}\right)^2} \leq 1.0 \qquad \text{(IS 9.3.1.3)}$$

As an exception, for Tee sections, the terms are algebraically added for three extreme points of the section. See the previous section of this manual for details.

3.9.1.2   Design for Overall Member Strength

The combined effect of axial compression and bending with special emphasis to flexural and lateral-torsional buckling is checked using Section IS 9.3.2.2. The combined effect of axial tension and bending is checked in accordance with Sections IS 9.3.2.1 and IS 9.3.2.2, with the exception that the axial term is ignored while checking in accordance with Section 9.3.2.2.

The program checks these equations assuming the section is prismatic. For nonprismatic sections the same equations are used. However the cross-section properties used are based on the section being checked. The user is advised to check the appropriateness of this method.

3.9.1.2.1   Sections Under Flexure and Axial Compression

The combined effect of axial compression and bending with special emphasis to flexural and lateral-torsional buckling is checked by calculating the utilization ratios based on the following two interaction equations:

$$\frac{P}{P_{dy}} + K_y \frac{C_{my}M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.2.2(a))}$$

$$\frac{P}{P_{dz}} + 0.6K_y \frac{C_{my}M_y}{M_{dy}} + K_z \frac{C_{mz}M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.2.2(b))}$$

where,

C_{my}, C_{mz} = equivalent uniform moment factor in accordance with IS Table 18,

P = applied axial compression under factored load,

M_y, M_z = maximum factored applied bending moments about the y- and z-axis of the member, respectively,

P_{dy}, P_{dz} = design strength under axial compression as governed by buckling about the minor (y) and major (z) axis, respectively,

M_{dy}, M_{dz} = design bending strength about the y (minor) or z (major) axis considering laterally unsupported length of the cross-section (IS Section 8),

$$K_y = 1 + (\lambda_y - 0.2) n_y \leq 1 + 0.8 n_y \quad\quad \text{(IS 9.3.2.2)}$$

$$K_z = 1 + (\lambda_z - 0.2) n_z \leq 1 + 0.8 n_z \quad\quad \text{(IS 9.3.2.2)}$$

$$K_{LT} = 1 - \frac{0.1 \lambda_{LT} n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1 n_y}{(C_{mLT} - 0.25)} \quad\quad \text{(IS 9.3.2.2)}$$

where,

n_y, n_z = ratio of actual applied axial force to the design axial strength for buckling about the y and z axis, respectively, and

$$\lambda_y = \frac{KL_{22}}{r_{22} \pi} \sqrt{\frac{f_y}{E}} \quad\quad \text{(IS 7.1.2.1)}$$

$$\lambda_z = \frac{KL_{33}}{r_{33} \pi} \sqrt{\frac{f_y}{E}} \quad\quad \text{(IS 7.1.2.1)}$$
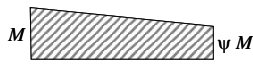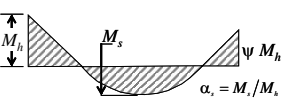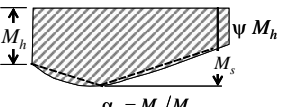
C_{mLT} = equivalent uniform moment factor for lateral-torsional buckling in accordance with IS Table 18 corresponding to the actual moment gradient between lateral supports against torsional deformation in the critical region under consideration.

The equation number reported in the output for the interaction equations in this section is really the section number 9.3.2.2. There are two interaction equations in this section of the code of which none of them are numbered in the code itself. So the program named the equations as 9.3.2.2(a) and 9.3.2.2(b) to distinguish between them.

As an exception, for Circular and Pipe sections, an SRSS (Square Root of Sum of Squares) combination is made first of the two bending components before addition of the axial load component instead of simple algebraic addition as implied by the equation given previously.

The C_{my}, C_{mz}, and C_{mLT} factors are the equivalent uniform moment factor for minor bending, major bending, and lateral-torsional buckling mode and are related to the distribution of moment over the braced segment. They are calculated depending on the loading condition and moment diagram based on Table 3-6 (IS Table 18, 9.3.2.2). All of the factors are calculated in the same way, except that appropriate bending axis and bracing direction are used. See Table 3-6 for details.

Table 3-6 Equivalent Uniform Moment Factor (IS Table 18, 9.3.2.2)

Bending Moment Diagram	Range		Case	C_{my}, C_{mz}, C_{mLT}	
				Two or More Concentrated Loads or Uniform Load	Single Concentrated Load On Segment
M ▱ ψM — Linear moment diagram	$-1 \le \psi \le 1$		(a)	$0.6 + 0.4\psi \ge 0.4$	
M_h M_s ψM_h $\alpha_s = M_s/M_h$ — End moment dominating $\lvert M_h \rvert \ge \lvert M_s \rvert$	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	(b)	$0.2 + 0.8\alpha_s \ge 0.4$	$0.2 + 0.8\alpha_s \ge 0.4$
	$-1 \le \alpha_s \le 0$	$0 \le \psi \le 1$	(c)	$0.1 - 0.8\alpha_s \ge 0.4$	$-0.8\alpha_s \ge 0.4$
		$-1 \le \psi \le 0$	(d)	$0.1(1-\psi) - 0.8\alpha_s \ge 0.4$	$0.2(-\psi) - 0.8\alpha_s \ge 0.4$
M_h ψM_h M_s $\alpha_h = M_h/M_s$ — Span moment dominating $\lvert M_s \rvert \ge \lvert M_h \rvert$	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	(e)	$0.95 - 0.05\alpha_h$	$0.90 + 0.10\alpha_h$
	$-1 \le \alpha_h \le 0$	$0 \le \psi \le 1$	(f)	$0.95 + 0.05\alpha_h$	$0.90 + 0.10\alpha_h$
		$-1 \le \psi \le 0$	(g)	$0.95 + 0.05\alpha_h(1+2\psi)$	$0.90 + 0.1\alpha_h(1+2\psi)$

For members with sway buckling mode, the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$

C_{my}, C_{mz}, C_{mLT} are obtained according to the bending moment diagram between the relevant braced point.

Moment Factor	Bending Axis	Points Braced in Direction
C_{mz}	z-z	y-y
C_{my}	y-y	z-z
C_{mLT}	z-z	z-z

3.9.1.2.2 Sections Under Flexure and Axial Tension

The combined effect of axial tension and bending with special emphasis to flexural and lateral-torsional buckling is checked by calculating the utilization ratios based on the following three interaction equations:

$$-\psi \frac{T/A}{M_d/Z_{ec}} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.2.1)}$$

$$K_y \frac{C_{my}M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.2.2(a))}$$

$$0.6K_y \frac{C_{my}M_y}{M_{dy}} + K_z \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{(IS 9.3.2.2(b))}$$

where,

T = factored tensile force,

A = area of the cross-section,

Z_{ec} = elastic section modulus of the section with respect to the extreme compression fiber,

ψ = 0.8, taken conservatively assuming M can vary independently,

and all other parameters have been described in the previous section.

The equation number reported in the output for the last two of the interaction equations in this section is really the section number 9.3.2.2. There are two interaction equations in this section of the code of which none of them are numbered in the code itself. So the program named the equations as 9.3.2.2(a) and 9.3.2.2(b) to distinguish between them. These two equations strictly applies to cases with axial compression and bending. However, they are still used in the case of axial tension and flexure to check flexural buckling mode. In this case the advantage of axial tension is ignored.

As an exception, for Circular and Pipe sections, an SRSS (Square Root of Sum of Squares) combination is made first of the two bending components before addition of the axial load component instead of simple algebraic addition as implied by the equation given previously.

3.9.2 Combined Axial Force, Bending Moment, and Shear Force

The combined effect of axial force, shear force, and bending moment is checked in the same way as described in Section 3.9.1 "Combined Axial Force and Bending Moments," with one exception. The exception is that moment capacity may be reduced depending on the value of shear force (IS 9.2.1, 9.2.2). This includes checking for cross-section strength (IS 9.3.1) only. The overall member strength (IS 9.3.2) is checked without consideration of the shear force.

When the factored shear force is less than or equal to 0.6 times the design shear strength ($V \leq 0.6\ V_d$), there is no reduction in moment capacity of the section necessary (IS 9.2.1). When the factored shear force is high ($V > 0.6\ V_d$), the moment capacity of the section is reduced to M_{dv}, which is calculated as described in Section 3.8.3.1 (IS 9.2.2).

3.9.3 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:

$$\frac{V_y}{V_{dy}} \leq 1.0 \tag{IS 8.4}$$

$$\frac{V_z}{V_{dz}} \leq 1.0 \tag{IS 8.4}$$

3.10 Checking the Maximum Effective Slenderness Ratio of a Member

The maximum effective slenderness ratio, KL/r values of a beam, strut or tension member shall not exceed those given in Table 3-7 (IS 3.8 Table 3). KL is the effective length of the member, and r is the appropriate radius of gyration.

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

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

Table 3-7 Maximum Values of Effective Slenderness Ratios (IS 3.8, Table 3)

SI No.	Member	Maximum Effective Slenderness Ratio (KL/r)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A member subjected to compression forces resulting from a combination with wind/earthquake actions only, provided the deformation of such member does not adversely affect the stress in any part of the structure.	250
iii)	Compression flange of a beam against lateral-torsional buckling.	300
iv)	Members always under tension	400

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 "IS 800:2007 – Indian Standard: General Construction in Steel – Code of Practice" (IS 2007). The implementation covers load combinations from "IS 800:2007," which are described in Section 4.4 *Design Special Load Combinations* in this chapter. The loading based on "IS 1893:2002" and "IS 875:1987" has been described in a separate document entitled "CSI Lateral Loads Manual" (IS 2007; CSI 2007).

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

- Reference to the IS 800:2007 code is identified with the prefix "**IS**."

- Reference to the IS 1893 (Part 1):2002 code is identified with the prefix "**IS 1893**."

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.

- Importance Factor
- Seismic Zone
- 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 (IS 12). With regard to those framing types, the program has implemented specifications for all of the types of framing systems listed, except EBF.

By default in the program, the frame type is taken as SMF. 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
SMF (Special Moment Frame)	IS 12.11
OMF (Ordinary Moment Frame)	IS 12.10
SCBF (Special Concentrically Braced Frame)	IS 12.7
OCBF (Ordinary Concentrically Braced Frame)	IS 12.8
EBF (Eccentrically Braced Frame)*	IS 12.9
Secondary or Gravity only	IS 12.1

*No special provision for EBF is programmed.

4.4 Design Special Load Combinations

The program creates the default design load combinations based on the specification of the code. The default combinations have been described in "Design Loading Combinations" in Chapter 3. Additional information may be found in "Design Load Combinations" in Chapter 2. The user may change the default load combinations if the governing code is different.

The program automatically considers seismic load effects, including overstrength factors (IS 12.2.3), as special load combinations that are created automatically from each load combination involving seismic loads. In that case, the earthquake force is represented by 2.5E.

The special seismic combinations are

$$1.2DL + 0.5LL \pm 2.5EL \qquad \text{(IS 12.2.3(a))}$$

$$0.9DL \pm 2.5EL \qquad \text{(IS 12.2.3(b))}$$

4.5 Special Check for Column Strength

When P / P_d in columns due to a prescribed loading combination is greater than 0.4, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the amplified seismic load combinations (IS 12.5.1.1).

$$1.2DL + 1.0LL \pm 2.5EL \hspace{3cm} \text{(IS 12.2.3(a))}$$

$$0.9DL \pm 2.5EL \hspace{3cm} \text{(IS 12.2.3(b))}$$

The preceding load combinations are used to calculate axial force only because other forces and moments are ignored. This axial capacity check is in addition to regular strength checks for the regularly specified load combinations (IS 12.5).

Those combinations are internal to the program. The user does NOT need to create additional load combinations for such load combinations. The special circumstances for which these load combinations are additionally checked are described later in this chapter, as appropriate. The special loading combination factors are applied directly to the load cases. It is assumed that any required scaling (such as may be required to scale response spectra results) has already been applied to the load cases.

If the overwrite item "Ignore Special Seismic Load?" is set to yes, the preceding check will not be performed.

4.6 Member Design

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

4.6.1 Ordinary Concentrically Braced Frames (OCBF)

For this framing system, the following additional requirements are checked or reported (IS 12.7).

- OCBF shall not be used in Seismic Zones IV and V and for buildings with an Importance Factor greater than 1 (I > 1) in Seismic Zone III.

- When P / P_d in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load combinations* as described

previously in the "Special Check for Column Strength" section of this manual (IS 12.5.1).

- All brace members are checked to be Plastic (Class 1), Compact (Class 2) or Semi-Compact (Class 3), but not Slender (Class 4) in accordance with Table 3-3 (IS 12.7.2.4; 3.7.2, Table 2). If this criterion is not satisfied, the program issues an error message.

- The maximum Kl/r ratio of the braces is checked not to exceed 120 (IS 12.7.2.1). If this criterion is not met, an error message is reported in the output.

- The required compressive strength of braces shall not exceed 0.8 times P_d (IS 12.7.2.2). If this criterion is not met, an error message is reported in the output.

4.6.2 Special Concentrically Braced Frames (SCBF)

For this framing system, the following additional requirements are checked or reported (IS 12.8).

- SCBF may be used in any seismic zone and for any bending irrespective of the value of the importance factor (IS 12.8.1.1).

- When P/P_d in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load combinations* as described previously in the "Special Check for Column Strength" section of this manual (IS 12.5.1).

- All column and brace members are checked to be Plastic (Class 1) in accordance with to Table 3-3 (IS 12.8.4.1, 12.8.2.5). If this criterion is not satisfied, the program issues an error message.

- The bracing member is designed such that gross area yielding governs over the net area rupture. This puts a limit on the net area of the gross area ratio (IS 12.8.2.7). If this criterion is not satisfied, the program issues an error message.

- The maximum Kl/r ratio of the braces is checked not to exceed 160 (IS 12.8.2.2). If this criterion is not satisfied, the program issues an error message.

4.6.3 Eccentrically Braced Frames (EBF)

The code does not give any direct specification for this framing type. It rather refers to specialist literature. The program does not check any special requirement for the framing type.

4.6.4 Ordinary Moment Frames (OMF)

For this framing system, the following additional requirement is checked and reported (AISC SEISMIC 11).

- OMF can not be used in seismic zones IV and V irrespective of the value of the importance factor and in seismic zone III for buildings with an importance factor greater than unity (I > 1) (IS 12.10.1.1).

- When P/P_d in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in the absence of any applied moment and shear for the *special seismic load combinations* as described previously in the "Special Check for Column Strength" section of this chapter (IS 12.5.1).

4.6.5 Special Moment Frames (SMF)

For this framing system, the following additional requirements are checked or reported (IS 12.11).

- This framing is applicable to any seismic zone and for any importance factor (IS 12.11.1.2).

- When P/P_d in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment

and shear for the *special seismic load combination* as described previously in the "Special Check for Column Strength" section of this chapter (IS 12.5.1).

▪ All beams and columns are required to be Plastic (Class 1) or Compact (Class 2) (IS 12.11.3.1). If this criterion is not satisfied, the program issues an error message.

4.6.6   Secondary (Non-Seismic)

No special consideration for this type of framing system is needed.

4.7   Joint Design

When checking for seismic load, 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

4.7.1   Design of Continuity Plates

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

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

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

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

To achieve a proper beam-column moment connection strength, continuity plates such as shown in Figure 2.3 of Chapter 2 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column. For connection conditions described by the first bullet, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends on the magnitude of the beam-flange force and the properties of the column. This is the condition that the program investigates. Columns of I Sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e., parallel to the column major direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames (OMF, SMF) only. No check is made for braced frames (OCBE, SCBF, EBF).

4.7.2   Design of Doubler Plates

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

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

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending on the loading and the geometry of the steel beams that frame into the column, either along the column major direction or at an angle so that the beams have components along the column major direction. See Figure 2-3 of Chapter 2. The program investigates such situations and reports the thickness of any

required doubler plates. Only columns with I Shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames (OMF, SMF) only. No check is made for braced frames (OCBF, SCBF, EBF).

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

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

The available strength of the web panel zone for the limit state of shear yielding is determined as V_d using the same procedure as described in Section 3.8.4 of this manual "Design Shear Strength." There is an exception that K_v is based on the assumption that the transverse stiffener spacing, c, is equal to the center-to-center flange distance of the beam, and the panel depth is equal to the depth of the column.

$$c = d_b - t_f$$

$$d = d_c$$

With this assumption, K_v, $\tau_{cr,e}$, λ_w, and τ_b are calculated. Then V_d is taken as

$$V_n = t_b \, d_c t_{wc} \qquad \qquad \text{(IS 8.4.2.2)}$$

If shear buckling does not control, τ_b becomes equal to $f_y / \sqrt{3}$ (IS 8.4.1).

By using $V_d = V_n / \gamma_{m0}$, the required column panel zone thickness t_p is found as follows.

$$t_p = \frac{V_p \, \gamma_{m0}}{\tau_b d_c}$$

The extra thickness, or the required thickness of the doubler plate is given as follows:

$$t_{dp} = t_p - t_w,$$

where,

f_y = Column and doubler plate yield stress

t_p = Required column panel zone thickness

t_{fn} = Flange thickness of n-th beam

t_{dp} = Required doubler plate thickness

t_{fc} = Column flange thickness

t_w = Column web thickness

V_p = Required panel zone shear capacity

V_c = Column shear in column above

n_b = Number of beams connecting to column

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

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

d_c = Overall depth of column

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

V_n = Nominal shear strength of panel

The largest calculated value of t_{dp}, calculated for any of the load combinations based on the factored beam moments and factored column axial loads, is reported.

Doubler plate requirements for seismic design are evaluated for SMF only. No further check/design is performed for other types of frames.

▪ For SMF, the panel zone doubler plate requirements that are reported will develop at least the beam moments equal to the expected plastic moment capacity of the beam $(1.2M_p)$, or beam moments due to specified load combinations involving seismic load (IS 12.11.2.2). For seismic design, V_p

is calculated using the equation given previously, except that M_{pb} is taken as $1.2 f_y Z_{p33}$ (IS 12.11.2.1). All other calculations follow the same procedure as given previously.

▪ For SMF, the program checks the following panel zone column web thickness requirement:

$$t \geq \frac{\left(d_c - 2t_{fc}\right) + \left(d_b - 2t_{fb}\right)}{90}. \qquad \text{(IS 12.11.2.4)}$$

Here, t is taken as $t_{wc} + t_{dp}$ when the doubler plate is plug welded to prevent local buckling. In such cases, t_{dp} is increased if necessary to meet this criterion. If the doubler plate is not plug welded to the web, t is taken as t_{wc} and also as t_{dp} for checking both plates. If t_{wc} cannot satisfy the criteria, then a failure condition is declared. If t_{dp} does not satisfy this criterion, then its value is increased to meet the criterion. If the check is not satisfied, it is noted in the output.

4.7.3   Weak Beam Strong Column Measure

Only for Special Moment Frames (SMF) the code requires that the sum of column flexure strengths at a joint should be more than 1.2 times the sum of beam flexure strengths (IS 12.11.3.2).

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2 \qquad \text{(IS 12.11.3.2)}$$

where

$\sum M_{pc}$ = sum of the moment capacity in the column above and below the beam centerline

$\sum M_{pb}$ = sum of the moment capacity in the beam at the intersection of the beam and the column centerlines

To facilitate the review of the strong column weak beam criterion, the program will report a beam-column plastic moment capacity ratio for every joint in the structure.

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

$$R_{maj} = \frac{1.2\sum_{n=1}^{n_b} M^*_{pbn} \cos\theta_n}{M^*_{pcax} + M^*_{pcbx}}.$$ (IS 12.11.3.2)

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

$$R_{maj} = \frac{1.2\sum_{n=1}^{n_b} M^*_{pbn} \sin\theta_n}{M^*_{pcay} + M^*_{pcby}},$$ (IS 12.11.3.2)

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 the column

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

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

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

n_b = Number of beams connecting to the column

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

$$M_{pc}^* = Z_c \left(F_{yc} - \left| \frac{P_{uc}}{A_g} \right| \right)$$

The plastic moment capacities of the beams are amplified for taking the beam moment capacity at the centerline of the column, rather than at the face of the column, as

$$M_{pb}^* = F_{yb} Z_b f_{mv}$$

where,

Z_b = Plastic modulus of beam

Z_c = Plastic modulus of column

F_{yb} = Yield stress of beam material

F_{yc} = Yield stress of column material

P_{uc} = Axial compression force in a column for a given load combination

A_g = Gross area of the column

f_{mv} = The moment amplification factor. It is taken as the ratio of beam moment at the centerline of column to the moment of the column face. Factor f_{mv} is taken as follows:

$$f_{mv} = 1 + \frac{d_c}{L_b} \text{ where,}$$

d_c = Depth of column section, and

L_b = clear span length of the beam.

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

4.7.4 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 SMF, the beam connection shear is taken as the maximum of those from regular load combinations and those required for the development of full plastic moment capacity of the beam. The shear strength need not exceed the required value corresponding to the special load combination in IS 12.2.3 with amplified earthquake load. The connection shear for the development of the full plastic moment capacity of the beam is as follows:

$$V = \frac{1.2CM_{pb}}{L_h} + 1.2V_{DL} + 0.5V_{LL} \qquad \text{(IS 12.11.2.2)}$$

where,

V =   Shear force corresponding to END I or END J of the beam

C =   0 if the beam ends are pinned, or for a cantilever beam
    =   1 if one end of the beam is pinned
    =   2 if no ends of the beam are pinned

M_{pb} =   Plastic moment capacity of beam

L_h =   Clear length of the beam

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

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

- For OMF, the beam connection shear is taken as the maximum of the two values: (a) maximum shear from the load combinations and (b) maximum shear based on beam moment capacity of the beam.

The maximum shear from the load combinations is determined from all of the regular load combinations but not from the *amplified seismic load combinations* (IS 12.2.3).

The maximum shear based on beam moment capacity is calculated for any load combination involving seismic load by replacing the seismic effect on shear force with the capacity shear as follows:

$$V_u = \frac{1.2 CM_{pb}}{L_h} + V_{DL} + V_{LL} \qquad \text{(IS 12.10.2.1)}$$

All parameters in the preceding equation have been described previously in this section. The moment connection is assumed to be rigid.

- For SCBF the beam connection shear is taken as the maximum of those from the regular load combination.

- For OCBF the beam connection shear is taken as the maximum of those from the regular load combinations.

- For EBF, the beam connection shear is taken as the maximum of those from the regular load combinations.

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.

4.7.5 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 (SCBF, OCBF) only. No special checks are performed for moment frames (OMF, SMF).

- For SCBF, the bracing connection force is taken as the following value (IS 12.8.3.1):

    a. The expected yield strength in tension of the bracing member, determined as $1.1 f_y A_g$ (IS 12.8.3.1(a)).

- For OCBF, the bracing connection force is taken as the minimum of the two values (IS 12.7.3):

    a. The expected yield strength in tension of the bracing member, determined as $1.2 f_y A_g$ (IS 12.7.3.1(a)).

    b. The maximum load effect of the amplified seismic load combination (IS 12.2.3, 12.7.3.1(b)).

- For EBF, no special requirements are checked. The user should refer to specialist literature.

Chapter 5
Design Output

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 "Indian IS 800:2007" design code related output information only.

5.1 Graphical Display of Design Information

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 "Indian IS 800:2007" code 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 framing type	▪ P-M colors and shear stress ratio values
▪ Live load reduction factors	
▪ 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 factors for braced condition, K_1, for major and minor directions of bending	▪ Continuity plate and doubler plate thickness
▪ Effective length factors for sway condition, K_2 factors, for major and minor directions of bending	▪ Beam/column capacity ratios
	▪ Beam connection shear
▪ Effective length factor, K_{LTB}, for lateral-torsional buckling	▪ Brace connection axial force
	▪ Identify the P-M failure
▪ C_m factors for major and minor directions of bending, and for lateral-torsional buckling (C_{mz}, C_{my}, C_{mLT})	▪ Identify the shear failure
	▪ Identify all failures
▪ Amplification factors (K_z, K_y, K_{LT}) for major and minor directions of bending	
▪ C_1 factors	
▪ Yield stress, f_y	
▪ Design axial capacities (P_d, T_d)	
▪ Design bending capacities (M_{dz}, M_{dy}, M_{cr})	
▪ Design shear capacities (V_{dy}, V_{dz})	

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 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.
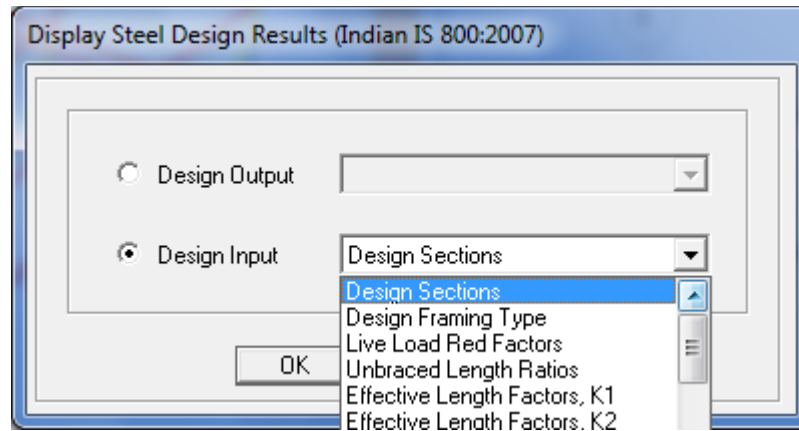
Figure 5-1 Choice of design input data for display on the model
in the active window
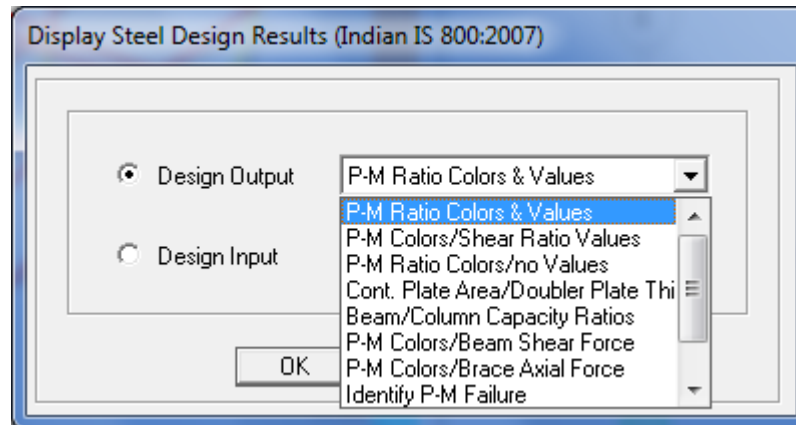
Figure 5-2 Choice of design output data for display on the model
in the active window

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 PM-Ratio in the active window. A typical graphical display is shown in Figure 5-3.
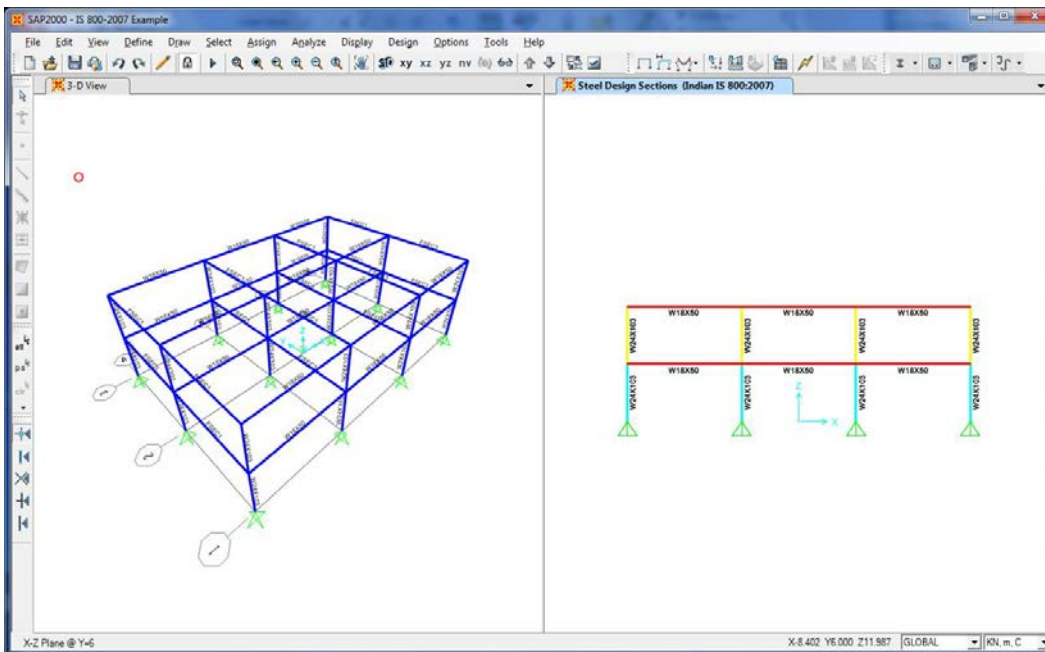
Figure 5-3 A typical graphical display

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** and the **Set 3D View** or **Set 2D View** commands.

The onscreen graphical display can be sent to printer using any of the following commands. Use the **File menu > Print Graphics** command 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 to create an .emf file, or use the **File menu > Capture Picture** command 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.

5.2 Tabular Display of Design Information

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 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.
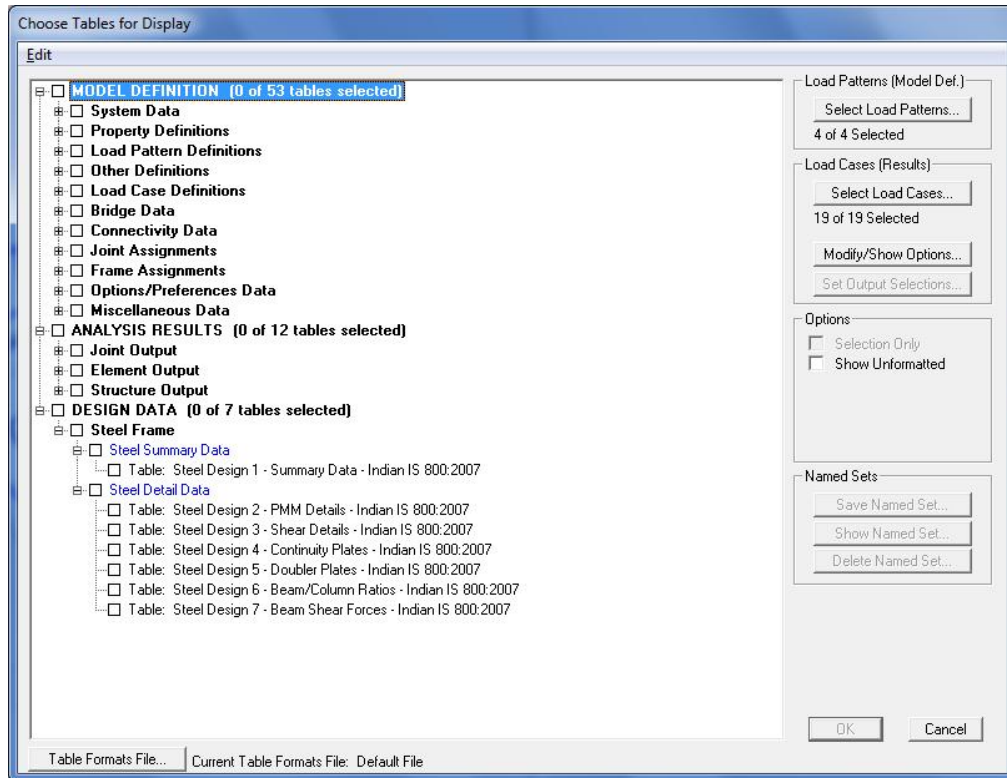
Figure 5-4 Choice of design data tables for tabular display

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 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 Patterns** button to specify which load patterns are to be included in the display of model definition data; click the **Select Load Case** and **Modify/Show Options** to specify which load cases are to be included and how analysis results are displayed.

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 **Format-Filter-Sort 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 **Format-Filter-Sort 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.

Each of the individual fields (columns) can be formatted. Fields with text information can be set for specific types of alignment (center, left, right) and to specific widths. In addition to setting the alignment and column widths, fields with numeric data can be set for the maximum number of characters, zero tolerance, number of decimal digits and so forth. The tables can be formatted, filtered and sorted using multiple criteria accessed using the **Format-Filter-Sort** menu on the form. Filtering and sorting features can be useful in identifying critical cases. After specifying the table formatting information, save it to a format file that can be used in the future by selecting the appropriate feature available under the **Format-Filter-Sort** menu.

The current table (i.e., the table in the active window) can be exported to Excel, Access, .rtf., .html, and plain text formats. In both Excel spreadsheet and Access database formats, the tables can have many columns, making the tables very

wide. In .rft and .html formats, the tables are split at convenient points to make a reasonably printable document. In plain text format, the tables can be saved with or without the splits. To export a table to Excel, for example, select the **File menu > Export Current Table > To Excel** command. The program will export the entire table to an Excel spreadsheet workbook and will automatically open the file in MS Excel. Similarly, if the table is exported to other formats, the default editor specific to that format will open the document, provided that the editor is available in the computer. The table will be saved in that format only if the relevant editor is not available. Instead of exporting or displaying the currently displayed table, the entire set of available tables can be exported and displayed in the afore-mentioned formats by selecting the File menu and the appropriate submenu command. With these exporting and display features, the tables can be saved in many required formats and can be printed. Many other features of the design tables are left for the user to discover by using the program.

The preceding description is for SAP2000. ETABS differs slightly.

5.3   Detailed Display of 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 K_1 (braced), K_2 (sway), C_{mz}, C_{my}, C_{mLT}, K_z, K_y, K_{LT}, 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 and then right click on the frame member. An example of that form is shown in Figure 5-6.

```
Steel Stress Check Information (Indian IS 800:2007)

  Frame ID       1                      Analysis Section    W24X103
  Design Code    Indian IS 800:2007     Design Section      W24X103

  COMBO     STATION /----MOMENT INTERACTION CHECK-----//-MAJ-SHR---MIN-SHR-/
  ID           LOC  RATIO   =   AXL + B-MAJ + B-MIN      RATIO      RATIO
  DSTL12       0.00  0.004(C) = 0.002 + 0.002 + 0.000    0.001      0.000
  DSTL12       1.50  0.004(C) = 0.001 + 0.002 + 0.000    0.001      0.000
  DSTL12       3.00  0.004(C) = 0.001 + 0.002 + 0.000    0.001      0.000
  DSTL13       0.00  0.006(C) = 0.003 + 0.002 + 0.000    0.001      0.000
  DSTL13       1.50  0.005(C) = 0.002 + 0.002 + 0.000    0.001      0.000
  DSTL13       3.00  0.005(C) = 0.002 + 0.002 + 0.000    0.001      0.000
```

Modify/Show Overwrites — Overwrites

Display Details for Selected Item — Summary · Flexure · Envelope

Display Complete Details — Tabular Data

Stylesheet: Table Format File

◉ Strength   ○ Deflection   OK   Cancel

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

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 a set of 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.

- **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, K_1 (braced), K_2 (sway), C_{mz}, C_{my}, C_{mLT}, K_z, K_y, K_{LT}, 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."

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

Figure 5-8 A typical Steel Stress Check Data form

Before clicking the button, 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 Data form, 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.

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.

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.

The Steel Stress Check Information form also includes a **Tabular Data** button that when clicked displays member specific design details in a spreadsheet type format. The tabular display of design information has been described in the previous section. In that case, the information displayed is specific to the selected member only. The tabular output is shown for all stations and all design load combinations for the selected member. An example of design details table that displays after clicking the **Tabular Data** button is shown in Figure 5-9. This button is available in SAP2000, but not in ETABS.

Figure 5-9 Member specific design details in tabular format

5.4   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 "Indian IS 800:2007" code, 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 to access the Choose Tables for Printing form. An example of that form is shown in Figure 5-10. Use the options on the form to choose which table or set of tables to save in a file or print. This form is very similar to the Choose Tables for Display form, which has been described earlier in this chapter in the "Tabular Display of Design Information" section.
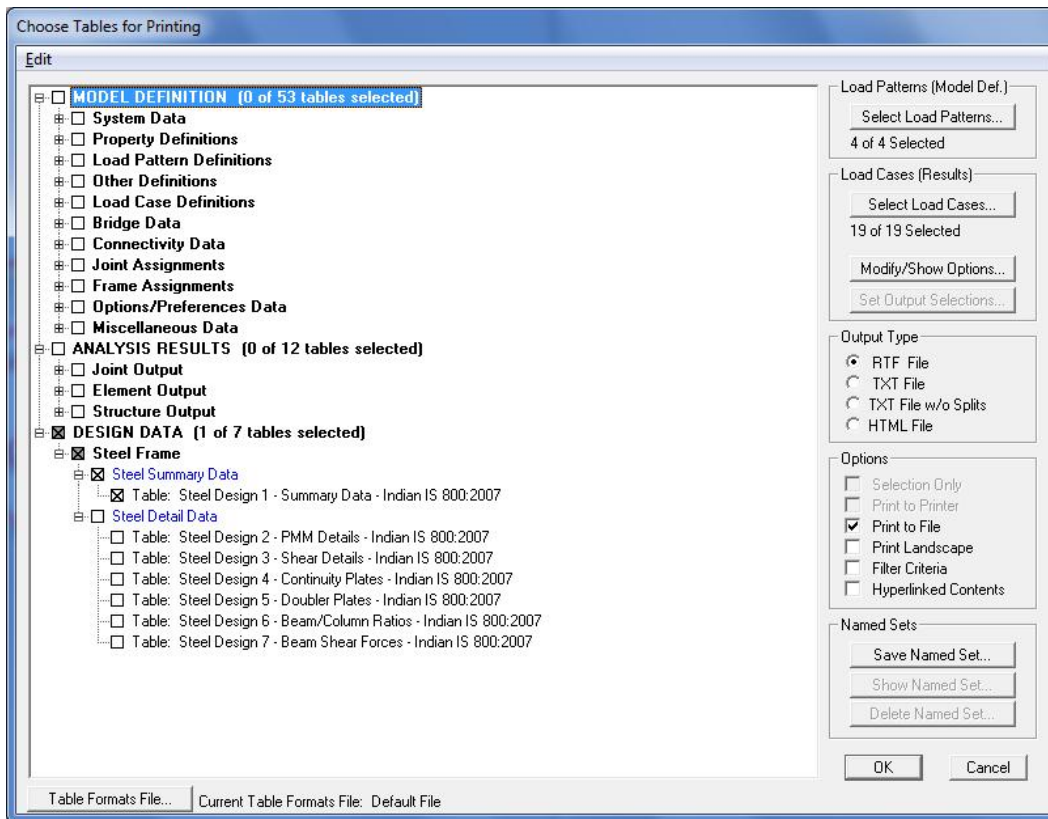
Figure 5-10 Choice of design tables for saving in a file or printing

The name 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 printing or saving to a file. If a branch of the tree is selected, all of the tables under that branch are selected. When all options for printing or saving the data to file have been specified, the settings can be saved as a Named Set using the **Save Named Set** button. The Named Set can be easily recalled in the future, reducing the need to reselect the options and ensuring that output is consistent from one printing effort to another.

If one or more structural members are selected before the Choose Tables for Printing form is accessed, the Selection Only check box will be checked, and the program will save or print the data for the selected members only; uncheck the check box to save or print the data for all appropriate members.

The output can be generated in a variety of formats, including rich text format (.rft), plain text with and without splits or page breaks (.txt), and hyperlink text markup language (.htm) by choosing the appropriate option on the right side of the form.

Printed output can be specified to have a landscaped orientation, in which the width of the resulting printout on a given page is longer than the height of the printout.

Filtering criteria can be applied if those criteria were specified using the **Format-Filter-Sort menu > Format Table** command when the selected data tables were displayed using the **Display menu > Show Tables** command. The output also can be specified to include a hyperlinked contents to facilitate accessing specific areas of the printout.

There are other buttons on the form that do not affect saving or printing design information. For example, the **Select Load Patterns** button affects only saving/printing model definition data. The **Select Load Cases** and **Modify/Show Options** buttons affect only saving/printing analysis results.

After clicking all the necessary tables for steel frame design and all necessary options, clicking the **OK** button will save/print the design tables.

Although the **File name > Print Tables** command path is dedicated for saving/printing design information, it is not the only path to do so. Design tables can be saved in different formats using the path **Display menu > Show Tables** command.

The preceding description applies to SAP2000. ETABS differs slightly.

5.5   Error and Warning Messages

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

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