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Integrated Solution for  
Structural Analysis and Design

**Steel Frame Design Manual**

SP 16.13330.2011





**Steel Frame  
Design Manual  
SP 16.13330.2011  
For SAP2000®**

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# Chapter 1

## Introduction

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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.

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 “SP 16.13330.2011.” This option covers the “*Code of Rules SP 16.13330.2011 – Steel Structures Revised Edition SNIIP II-23-81\* Official Edition; Russian Federation, Ministry of Regional Development, Moscow*” (SP16 2011). The implementation covers loading and load combinations from “*Code of Practice SP 20.13330.2011 – Loads and Actions, Updated Version SNIIP 2.01.07-85\* Official Edition; Russian Federation, Ministry of Regional Development, Moscow*” (SP20 2011).

The design codes supported under “SP 16.13330.2011” 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 SP 16.13330.2011 steel frame design option. Chapter 2 addresses prerequisites related to modeling and analysis for a successful design in accordance with SP 16.13330.2011. Chapter 3 provides detailed descriptions of the specific requirements as implemented in SP 16.13330.2011.

## 1.2 Recommended Reading/Practice

It is strongly recommended that you read this manual and review any applicable “Watch & Learn” Series™ 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.

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## Chapter 2

# Modeling, Analysis and Design Prerequisites

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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.

- The SP 16.13330.2011 code is referenced with the prefix “**SP16.**”
- The SP 20.13330.2011 code is referenced with the prefix “**SP20.**”

### 2.1 Check and Design Capability

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

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

passes all of the special requirements, the section is considered to be adequate, else the section is considered to be failed. The D/C ratio limit is taken as 0.95 by default. However, this value can be overwritten in the Preferences (Chapter 3).

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

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

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

## 2.2 Analysis Sections vs. Design Sections

Analysis sections are those section properties used to analyze the model when the analysis is run. The design section is whatever section is used in the steel frame design. It is possible for the last used analysis section and the current design section to be different. For example, an analysis may be run using a W18X35 beam, and then in the design, it may be found that a W16X31 beam worked. In that case, the last used analysis section is the W18X35 and the current design section is the W16X31. Before the design process is complete, verify that the last used analysis section and the current design section are the same. Refer to the program Help for more information about completing this task.

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

- Assigning a frame section property assigns the section as both the analysis section and the design section.
- Running an analysis always sets the analysis section to be the same as the current design section.
- Assigning an Auto Select section list to a frame object initially sets the analysis and design section to be the section in the list with the median weight.
- Unlocking a model deletes the design results, but it does not delete or change the design section.
- Altering the Design Combinations in any way deletes the design results, but does not delete or change the design section.
- Altering any of the steel frame design preferences deletes the design results, but does not delete or change the design section.

## 2.3 Design and Check Stations

For each design combination, steel frame members (beams, columns, and braces) are designed (optimized) or checked at a number of locations (stations) along the length of the object. The stations are located at equally spaced segments along the clear length of the object. By default, at least three stations will be located in a column or brace member, and the stations in a beam will be spaced at most 0.5 meter apart (2 feet if the model has been created in US units). The user can overwrite the number of stations in an object before the analysis is made using the **Assign** menu. The user can refine the design along the length of a member by requesting more stations.

## 2.4 Demand/Capacity Ratios

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

- The factored forces are calculated for axial, flexural, and shear at each defined station for each design combination. The bending moments are calculated about the principal axes. For I-Shape, Box, Channel, T-Shape, Double-Angle, Pipe, Circular, and Rectangular sections, the principal axes coincide with the geometric axes. For Single-Angle sections, the design considers the principal properties. For General sections, it is assumed that all section properties are given in terms of the principal directions.

For Single-Angle sections, the shear forces are calculated for directions along the geometric axes. For all other sections, the program calculates the shear forces along the geometric and principal axes.

- The design strengths are calculated for compression, tension, bending and shear based on the equations provided later in this manual. For flexure, the design strengths are calculated based on the principal axes of bending. For the I-Shape, Box, Channel, Circular, Pipe, T-Shape, Double-Angle and Rectangular sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations related to flexural stresses are based on that.

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

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

## 2.5 Design Load Combinations

The design load combinations are the various combinations of the prescribed analysis cases for which the structure needs to be checked. The program creates a number of default design load combinations for steel frame design. Users can add their own design combinations as well as modify or delete the program default design load combinations. An unlimited number of design load combinations can be specified.

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

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

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

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

For multi-valued design combinations, such as those involving response spectrum, time history, moving loads and envelopes, where any correspondence between forces is lost, the program automatically produces sub-combinations using the maxima/minima values of the interacting forces. Separate combina-



tions with negative factors for response spectrum analysis cases are not required because the program automatically takes the minima to be the negative of the maxima response when preparing the sub-combinations described previously.

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

## 2.6 Second Order P-Delta Effects

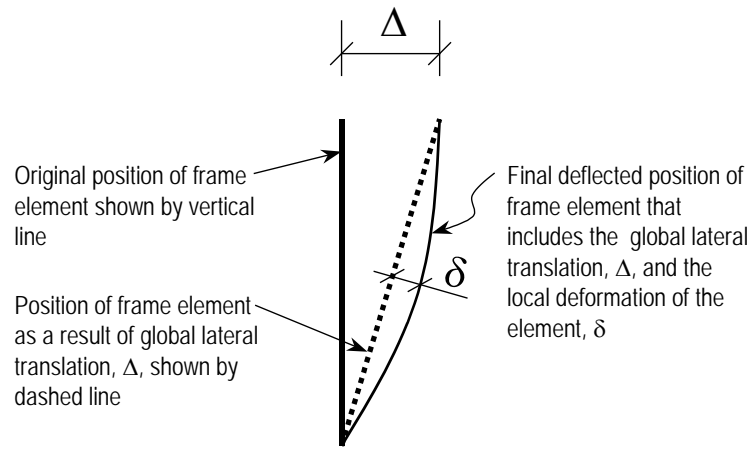
Modern design provisions are based on the principle that the member forces are calculated by a second-order elastic analysis, where the equilibrium is satisfied on the deformed geometry of the structure. The effects of the loads acting on the deformed geometry of the structure are known as the second-order or the P-Delta effects.

The P-Delta effects come from two sources: global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure 2-1, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by  $\Delta$ . The local deformation of the member is shown as  $\delta$ . The total second order P-Delta effects on this frame object are those caused by both  $\Delta$  and  $\delta$ .

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  $\Delta$  deformation ( $P-\Delta$  effect) shown in Figure 2-1, but it does not typically capture the effect of the  $\delta$  deformation ( $P-\delta$  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  $P-\Delta$  and  $P-\delta$  effects. Approximate second-order analysis procedures based on amplification of responses from first-order analysis for calculating the required flexural and axial strengths are common in current design codes and have the following general form:



**Figure 2-1 P-Δ and P-δ effects**

$$M_{CAP} = U_1 (M_{nt} + U_2 M_{lt})$$

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  $\delta$  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  $\Delta$  deformation in Figure 2-1)

$U_1$  = Unitless amplification factor multiplying  $M_{nt}$

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

A rigorous second order analysis or the amplification of first order analysis results to estimate the effect of second order effects is required (SP16 4.2.4, 4.2.5). The program has the capability of performing both. In the first case, the

required strengths are determined directly from the analysis results without any amplification factors (i.e.,  $U_1$  and  $U_2$  are equal to 1).

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

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  $P-\Delta$  effect, it is recommended to use more than one element per line object for the cases where both  $P-\Delta$  and  $P-\delta$  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  $P-\Delta$  and  $P-\delta$  effects are captured better.

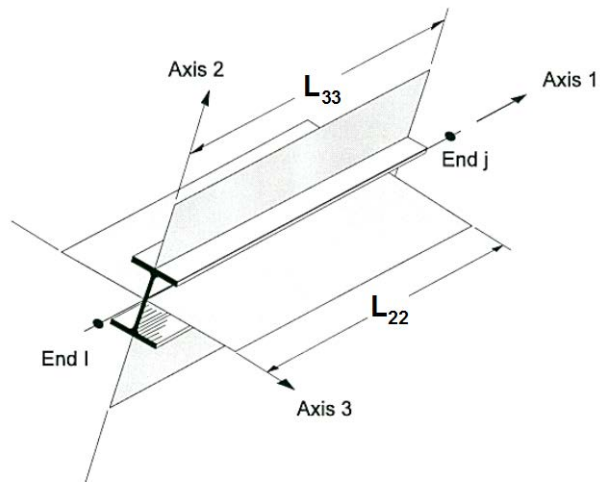
## 2.7 Member Unsupported Lengths

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

Two unsupported lengths,  $L_{33}$  and  $L_{22}$ , as shown in Figure 2-2 are to be considered for flexural buckling. These are the lengths between support points of the member in the corresponding directions. The length  $L_{33}$  corresponds to insta-

bility about the 3-3 axis (major axis), and  $L_{22}$  corresponds to instability about the 2-2 axis (minor axis). The length  $L_{LTB}$  (also termed  $L_z$ ), not shown in the figure, is also used for lateral-torsional buckling caused by major direction bending (i.e., about the 3-3 axis).

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



**Figure 2-2 Unsupported lengths  $L_{33}$  and  $L_{22}$**

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

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

## 2.8 Effects of Breaking a Member into Multiple Elements

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

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

1. **The unbraced length:** The unbraced length is really the unsupported length between braces. If no intermediate brace is present in the member, the unbraced length is typically calculated automatically by the program from the top of the flange of the beam framing the column at the bottom to the bottom of the flange of the beam framing the column at the top. The automatically calculated length factor typically becomes less than 1. If there are intermediate bracing points, the user should overwrite the unbraced length factor in the program. The user should choose the critical (larger) one. Even if the user breaks the element, the program typically picks up the unbraced length correctly, provided that there is no intermediate bracing point.
2.  **$\mu$ -factor:** Even if the user breaks the member into pieces, the program typically can pick up the  $\mu$ -factors correctly. However, sometimes it can not. The user should note the  $\mu$ -factors. All segments of the member should have the same  $\mu$ -factor and that factor should be calculated based on the entire member. If the calculated  $\mu$ -factor is not reasonable, the user can overwrite the  $\mu$ -factors for all the segments.
3.  **$\phi_b$ -factor:** The  $\phi_b$  factor should be based on the end moments of unbraced lengths of each segment and should not be based on the end moments of the member (SP16 App. G, Table G1). The program already calculates the  $\phi_b$  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  $\phi_b$  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  $\phi_b$  factor manually for the critical combination and overwrite its value for that segment.

4.  $\phi_e$  factor: The logic is similar to that for the  $\omega_l$  factor.

If the user models a column with a single element and makes sure that the  $L$ -factors and  $\mu$ -factors are correct, the effect of  $U_1$  and  $U_2$  will be picked up correctly. The factors  $\phi_b$  and  $\phi_e$  will be picked up correctly if there is no intermediate bracing point. The calculated  $\phi_b$  and  $\phi_e$  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  $\mu$ -factor are correct, the effect of  $\phi_b$  and  $\phi_e$  will be picked up correctly. The factors  $\phi_b$  and  $\phi_e$  will be picked up correctly if the member is broken at the bracing points. The calculated  $\phi_b$  and  $\phi_e$  factors will be conservative if the member is not broken at the bracing points.

## 2.9 Effective Length Factor ( $\mu$ )

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,  $\mu L$ , and is based on elastic/inelastic stability theory. The effective buckling length is used to calculate an axial compressive strength factor,  $\phi$ , through an empirical column curve that accounts for geometric imperfections, distributed yielding, and residual stresses present in the cross-section.

The  $\mu$ -factor is used for calculating the Euler axial capacity assuming that all the beam-column joints are free to sway, i.e., lateral translation is allowed. The resulting axial capacity is used in calculating  $\phi$  factor. The  $\mu$ -factor is always greater than 1 if the frame is a sway frame. The program calculates the  $\mu$ -factor automatically based on sway condition. The program also allows the user to overwrite  $\mu$ -factors on a member-by-member basis. The same  $\mu$ -factor

is supposed to be used in calculation of  $\phi_b$  and  $\phi_e$  factors. If the frame is not really a sway frame, the user should overwrite the  $\mu$ -factors.

$\mu$  has two values: one for major direction,  $\mu_{major}$ , and the other for minor direction,  $\mu_{minor}$ .

There is another  $\mu$ -factor,  $\mu_{LTB}$  for lateral torsional buckling. By default,  $\mu_{LTB}$  is taken as equal to  $\mu_{minor}$ . However the user can overwrite this on a member-by-member basis.

The rest of this section is dedicated to the determination of  $\mu$ -factors.

The  $\mu$ -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  $\mu$ -factor calculation is relatively complex. For the purpose of calculating  $\mu$ -factors, the objects are identified as columns, beam and braces. All frame objects parallel to the Z-axis are classified as columns. All objects parallel to the X-Y plane are classified as beams. The remainders are considered to be braces.

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

$$\begin{aligned} S_{cx} &= \sum \left( \frac{E_c I_c}{L_c} \right)_x & S_{bx} &= \sum \left( \frac{E_b I_b}{L_b} \right)_x \\ S_{cy} &= \sum \left( \frac{E_c I_c}{L_c} \right)_y & S_{by} &= \sum \left( \frac{E_b I_b}{L_b} \right)_y \end{aligned}$$

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

and the END-J of the member are calculated about the 2-2 and 3-3 directions as follows:

$$G^I_{22} = \frac{S^I_{c22}}{S^I_{b22}} \qquad G^J_{22} = \frac{S^J_{c22}}{S^J_{b22}}$$

$$G^I_{33} = \frac{S^I_{c33}}{S^I_{b33}} \qquad G^J_{33} = \frac{S^J_{c33}}{S^J_{b33}}$$

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

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

from which  $\mu = \frac{\pi}{\alpha}$ . This relationship is the mathematical formulation for the evaluation of  $\mu$ -factors for moment-resisting frames assuming sidesway to be uninhibited. For other structures, such as braced frame structures, the  $\mu$ -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  $\mu$ -factor algorithm:

- An object that has a pin at the joint under consideration will not enter the stiffness summations calculated previously. An object that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated  $EI$  value. Also, beam members that have no column member at the far end from the joint under consideration, such as cantilevers, will not enter the stiffness summation.
- If there are no beams framing into a particular direction of a column member, the associated  $G$ -value will be infinity. If the  $G$ -value at any one end of a column for a particular direction is infinity, the  $\mu$ -factor corresponding to that direction is set equal to unity.



- If rotational releases exist at both ends of an object for a particular direction, the corresponding  $\mu$ -factor is set to unity.
- The automated  $\mu$ -factor calculation procedure occasionally can generate artificially high  $\mu$ -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  $\mu$ -factors of unity are to be used.
- All  $\mu$ -factor produced by the program can be overwritten by the user. These values should be reviewed and any unacceptable values should be replaced.
- The beams and braces are assigned  $\mu$ -factors of unity.

## 2.10 Supported Framing Types

The code recognizes the following types of framing systems.

<b>Framing Type</b>	<b>References</b>
Restrained or Braced Frame	SP16 4.2.4
Unrestrained or Unbraced Frame	SP16 4.2.4

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## 2.11 Frame Design Procedure Overwrites

The structural model may contain frame elements made of several structural materials: steel, concrete, aluminum, cold-formed steel and other materials. The program supports separate design procedures for each material type. By default the program determines the design procedure from the material of the frame member.

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

ETABS supports both regular steel frame design and composite beam design. The determination of design procedure is different. If the material is concrete, the design procedure is concrete. If the material is steel, the default design 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. Refer to the program Help for more information about changing the design procedure. A change in design will be successful only if the design procedure is valid for that member, i.e., the program will not allow the user to change the design procedure for a steel frame object to concrete frame design.

## 2.12 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.

## 2.13 Automated Iterative Design

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

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

- The Design Sections and the last-used Analysis Sections are the same.
- The number of iterations performed is equal to the number of iterations specified for the maximum number on the Preferences form.

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## Chapter 3 Design

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This chapter provides a detailed description of the algorithms used by the programs in the design/check of structures in accordance with “*Code of Rules SP 16.13330.2011 – Steel Structures Revised Edition SNiP II-23-81\* Official Edition; Russian Federation, Ministry of Regional Development, Moscow*” (SP16 2011). The implementation covers load combinations from “SP 20.13330.2011,” which is described in *Section 3.4 Design Loading Combinations* in this chapter. The loading based on “*Code of Practice SP 20.13330.2011 – Loads and Actions, Updated Version SNiP 2.01.07-85\* Official Edition; Russian Federation, Ministry of Regional Development, Moscow*” (SP20 2011).

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

- The SP 16.13330.2011 code is referenced with the prefix “**SP16.**”
- The SP 20.13330.2011 code is referenced with the prefix “**SP20.**”

### 3.1 Notations

The various notations used in this manual are described herein.

Please note that the code uses the  $x$  and  $y$  axes as the major and minor principal axes, respectively. Whereas the program uses the 3 and 2 axes as the major and minor principal axes (local axes), respectively. This documentation tried to write the equations in the same manner as done in the code itself. However in certain situations, the indices  $x$  and 3 are used synonymously. Similarly, the indices  $y$  and 2 are used synonymously. This is displayed in Table 3.1.

$A$	Gross cross-sectional area, mm <sup>2</sup>
$A_{bn}$	Net cross-sectional area of bolt, mm <sup>2</sup>
$A_d$	Diagonal cross-sectional area, mm <sup>2</sup>
$A_f$	Cross-sectional area of flange (belt), mm <sup>2</sup>
$A_n$	Net cross-sectional area, mm <sup>2</sup>
$A_w$	Wall cross-sectional area, mm <sup>2</sup>
$A_{wf}$	Cross-sectional area of weld fillet metal, mm <sup>2</sup>
$A_{wz}$	Cross-sectional area of weld metal zone, mm <sup>2</sup>
$E$	Modulus of elasticity, MPa
$F$	Force, N
$G$	Shear modulus, MPa
$I$	Moment of inertia of gross cross-section, mm <sup>4</sup>
$I_b$	Moment of inertia of branch cross-section, mm <sup>4</sup>
$I_m; I_d$	Moments of inertia of belt cross-section and diagonal web elements, mm <sup>4</sup>
$I_r$	Moment of inertia of rib and rail cross-section, mm <sup>4</sup>
$I_{rl}$	Moment of inertia of longitudinal rib cross-section, mm <sup>4</sup>
$I_t$	Moment of inertia under free torsion, mm <sup>4</sup>
$I_x; I_y$	Moment of inertia about $x - x$ and $y - y$ axis respectively, mm <sup>4</sup>
$I_{xn}; I_{yn}$	As above, net cross-section, mm <sup>4</sup>
$I_\omega$	Cross-sectional sectorial moment of inertia, mm <sup>4</sup>
$M$	Moment, bending moment, N-mm

$M_x; M_y$	Moments about x – x and y – y axes respectively, N-mm
$N$	Longitudinal force, N
$N_{ad}$	Additional stress, N-mm <sup>2</sup>
$N_{bm}$	Longitudinal force from moment in stanchion line, N
$Q$	Transverse force, shear force, N
$Q_{fic}$	Conventional transverse force for connectives, N
$Q_s$	Conventional transverse force per plate system arranged in one plane, N
$R_{ba}$	Design strength of tension of footing bolts, N-mm <sup>2</sup>
$R_{bh}$	Design strength of tension of high-strength bolts, N-mm <sup>2</sup>
$R_{bp}$	Design strength of bearing stress of pin joint, N-mm <sup>2</sup>
$R_{bs}$	Design strength of pin joint cross-section, N-mm <sup>2</sup>
$R_{bt}$	Design strength of tension of pin joint, N-mm <sup>2</sup>
$R_{bun}$	Characteristic strength of bolt steel that shall be taken equal to the breaking strength $\sigma_v$ in conformance with State standards and specifications for bolts, N-mm <sup>2</sup>
$R_{bu}$	Characteristic strength of tension of U-shaped bolts, N-mm <sup>2</sup>
$R_{byn}$	Characteristic strength of bolt steel that shall be taken equal to the liquid limit $\sigma_t$ in conformance with State standards and specifications for bolts, N-mm <sup>2</sup>
$R_{cd}$	Characteristic strength of diametric compression of rollers (during free contact in limited mobility constructions), N-mm <sup>2</sup>
$R_{dh}$	Characteristic strength of high-tensile wire, N-mm <sup>2</sup>
$R_{tp}$	Characteristic strength of local collapse in hinge-joints (spigot) during close contact, N-mm <sup>2</sup>
$R_p$	Characteristic strength of steel to butt end collapse (if matching is available), N-mm <sup>2</sup>
$R_s$	Characteristic strength of steel for shearing, N-mm <sup>2</sup>
$R_u$	Characteristic strength of steel for tension, collapse, bending of breaking strength, N-mm <sup>2</sup>

$R_{un}$	Characteristic strength of steel that shall be taken equal to minimal value $\sigma_v$ in conformance with the State standards and specifications on steel, N-mm <sup>2</sup>
$R_v$	Design fatigue resistance of steel, N-mm <sup>2</sup>
$R_{wcf}$	Design strength of corner joints to weld metal cross-section (conventional), N-mm <sup>2</sup>
$R_{wtu}$	Design tension, collapse, bending strength of butt weld across breaking strength, N-mm <sup>2</sup>
$R_{wum}$	Characteristic strength of weld metal across breaking strength, N-mm <sup>2</sup>
$R_{ws}$	Design shear strength of butt weld, N-mm <sup>2</sup>
$R_{wy}$	Design tensile, collapse and bending strength of butt welds at liquid limit, N-mm <sup>2</sup>
$R_{wz}$	Design cross-sectional (conventional) strength of corner joints at metal welding line, N-mm <sup>2</sup>
$R_y$	Design tensile, collapse, bending strength at liquid limit, N-mm <sup>2</sup>
$R_{yf}$	As above, for flange (belt), N-mm <sup>2</sup>
$R_{yw}$	As above, for flange, N-mm <sup>2</sup>
$R_{yn}$	Steel liquid limit that shall be taken equal to liquid limit value $\sigma_{ly}$ in conformance with State standards and specifications on steel, N-mm <sup>2</sup>
$S$	Static moment of gross cross-sectional shift along zero axis, N-mm
$W_x; W_y$	Gross resistance of cross-sectional moments along x-x and y-y axis respectively, N-mm
$W_c; W_t$	Moment cross-sectional resistance for collapsed and tension flange respectively, N-mm
$W_{xn}; W_{yn}$	Net resistance of cross-sectional moments along x – x and y – y axis respectively, N-mm
$b$	Width, mm
$b_{ef}$	Effective width, mm
$b_f$	Flange (belt) width, mm

$b_r$	Width of projected arch band, overhang, mm
$c_x; c_y$	Factors of inelastic flexural strain along x – x, y – y axis respectively
$d$	Bolt hole diameter, mm
$d_b$	Full diameter of bolt shaft, mm
$e$	Eccentricity of normal force, mm
$h$	Height, mm
$h_{ef}$	Design height of wall, mm
$h_w$	Wall height, mm
$i$	Radius of cross-sectional inertia, mm
$i_{\min}$	Minimum radius of cross-sectional inertia, mm
$i_x; i_y$	Inertia radius of cross-section along x – x and y – y axis respectively, mm
$k_f$	Leg fillet weld, mm
$l$	Length, span, mm
$l_c$	Length of stand, rack, bar, mm
$l_d$	Length of diagonal rod, mm
$l_{ef}$	Effective length, mm
$l_m$	Flange (rack) span length, mm
$l_s$	Jointing length, mm
$l_w$	Weld length, mm
$l_x; l_y$	Effective length of elements in planes perpendicular to x – x and y – y axis respectfully, mm
$m$	Eccentricity ratio $m = eA/W_c$
$r$	Radius, mm
$t$	Thickness, mm
$t_f$	Flange (belt) thickness, mm
$t_w$	Wall thickness, mm



$\alpha_f$	Ratio of flange (belt) cross-section and wall $\alpha_f = A_f/A_w$
$\beta_f, \beta_z$	Factors of fillet weld across weld metal and metal line, mm
$\gamma_b$	Service factor of bolt joint
$\gamma_c$	Service factor
$\gamma_f$	Partial safety factor for loads
$\gamma_m$	Partial safety factor for material
$\gamma_n$	Importance safety factor
$\gamma_u$	Safety factor for breaking strength
$\gamma_s$	Safety factor for system stability
$\eta$	Factor of cross-section form impact
$\lambda$	Elasticity $\lambda = I_{ef}/i$ , N-mm <sup>2</sup>
$\bar{\lambda}$	Conventional elasticity $\bar{\lambda} = \lambda\sqrt{R_y/E}$ , N-mm <sup>2</sup>
$\lambda_{ef}$	Equated slenderness of end-to-end cross-section
$\bar{\lambda}_{ef}$	Conventional equated slenderness of end-to-end cross-section, $\bar{\lambda}_{ef} = \lambda\sqrt{R_y/E}$
$\lambda_f$	Conventional slenderness of belt-overhang, $\bar{\lambda}_f = (b_{ef}/t_f)\lambda\sqrt{R_y/E}$
$\lambda_{f,1}$	Conventional slenderness of flange plate, $\bar{\lambda}_{f,1} = (b_{ef,1}/t_f)\lambda\sqrt{R_y/E}$
$\lambda_w$	Conventional slenderness of wall, $\bar{\lambda}_w = (h_{ef}/t_w)\lambda\sqrt{R_y/E}$
$\lambda_{uf}$	Limit conventional slenderness of belt-overhang (flange plate)
$\lambda_{uw}$	Limit conventional slenderness of wall
$\lambda_x; \lambda_y$	Rated element slenderness in planes perpendicular to x – x and y – y axes respectively
$ \sigma $	Absolute value of normal stress, N-mm <sup>2</sup>
$\sigma_{loc}$	Local stress, N-mm <sup>2</sup>
$\sigma_x; \sigma_y$	Normal stress parallel to x – x and y – y axes respectively, N-mm <sup>2</sup>
$\tau$	Shear stress, N-mm <sup>2</sup>
$\varphi$	Stability coefficient at axial compression

$\phi_{x(y)}$	Stability coefficient at compression
$\phi_b$	Stability coefficient at bending
$\phi_e$	Compression coefficient with bending
$\phi_{exy}$	Stability coefficient at bending in two planes

## 3.2 Design Preferences

The steel frame design preferences are basic assignments that apply to all of the steel frame members. Table 3-1 lists steel frame design preferences for “SP 16.13330.2011.” Default values are provided for all preference items. Thus, it is not necessary to specify or change any of the preferences. However, at least review the default values to ensure they are acceptable. Some of the preference items also are available as member-specific Overwrite items. The overwrites are described in the next section. Overwritten values take precedence over the preferences. Refer to the program Help for information about changing Preferences.

**Table 3-1: Steel Frame Design Preferences**

Item	Possible Values	Default Value	Description
Design Code	Design codes available in the current version	AISC360-10/ IBC 2006	The selected design code. Subsequent design is based on this selected code.
Multi-Response Case Design	Envelopes, Step-by-Step, Last Step, Envelopes, All, Step-by-Step - All	Envelopes	Select to indicate how results for multivalued cases (Time history, Nonlinear static or Multi-step static) are considered in the design. - Envelope - considers enveloping values for Time History and Multi-step static and last step values for Nonlinear static. Step-by-Step - considers step by step values for Time History and Multi-step static and last step values for Nonlinear static. Last Step - considers last values for Time History, Multi-step static and Nonlinear static. Envelope - All - considers enveloping values for Time History, Multi-step static and Nonlinear static. Step-by-Step - All - considers step by step values for Time History, Multi-step static and Nonlinear static. Step-by-Step and Step-by-Step - All default to the corresponding Envelope if more than one multivalued case is present in the combo.
Framing Type	Moment Frame, Braced Frame	Moment Frame	This item is used for ductility and stability considerations in the design.

**Table 3-1: Steel Frame Design Preferences**

Item	Possible Values	Default Value	Description
Section Class	Class 1, Class 2, Class 3, Class 4	Class 1	The section class requested. This is either "Class 1", "Class 2", "Class 3", or "Class 4". It determines the interaction equations to be used. If not overwritten, it is taken from the preference of the code of the code, "SP 16.13330.2011."
GammaM	$\geq 0$	1.1	The partial safety factor for material (SP 16.13330.2011 6.1, Table 3).
GammaC	$\geq 0$	1.0	The service factor for structural members and connections (SP 16.13330.2011 4.3.2, Table 1).
GammaU	$\geq 0$	1.3	The safety factor for breaking strength of structural members designed in terms of their behavior with the use of design strength $R_u$ (SP 16.13330.2011 4.3.2).
GammaC1	$\geq 0$	1.0	The service factor for structural single angle members in tension (SP 16.13330.2011 7.1.2).
Consider Deflection?	Yes, No	Yes	Toggle to consider the deflection limit (Yes) or to not consider the deflection limit (No).
DL Limit, L/	$\geq 0$	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/	$\geq 0$	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/	$\geq 0$	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/	$\geq 0$	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/	$\geq 0$	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	$\leq 1.0$	0.75	The live load factor for automatic generation of load combinations involving pattern live loads and dead loads.
Demand/Capacity Ratio Limit	$\leq 1.0$	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 “SP 16.13330.2011.” Default values are provided for all Overwrite items. Thus, it is not necessary to specify or change any of the Overwrites. However, at least review the default values to ensure they are acceptable. When changes are made to Overwrite items, the program applies the changes only to the elements to which they are specifically assigned. Overwritten values take precedence over the Preferences. Refer to the program Help for information about changing Overwrites.

**Table 3-2 Steel Frame Design Overwrites**

Item	Possible Values	Default Value	Description
Current Design Section	Any defined steel section	Analysis section	The design section for the selected frame object. When this Overwrite is applied, any previous auto select section assigned to the frame object is removed.
Fame Type	Moment Frame, Braced Frame	From Preferences	This item is used for ductility and stability considerations in the design.
Section Class	Class 1, Class 2, Class 3, Class 4	Program determined	Section class to be used. This is either "Class 1", "Class 2", "Class 3", or "Class 4". It determines the interaction equations to be used.
GammaC	$\geq 0$	Program determined	The service factor for structural members and connections (SP 16.13330.2011 4.3.2, Table 1).
GammaU	$\geq 0$	Program determined	The safety factor for breaking strength of structural members designed in terms of their behavior with the use of design strength $R_u$ (SP 16.13330.2011 4.3.2).
GammaC1	$\geq 0$	Program determined	The service factor for structural single angle members in tension (SP 16.13330.2011 7.1.2).
Column Buckling Curve (x-x)	a, b, or c	Program determined	Column buckling curve to be used. This is either "a", "b", or "c". It determines the imperfection factors for buckling curve. It is used to find the alpha and beta factors, and to eventually delta and phi factors, for compression capacity per SP 16.13330.2011 7.1.3, Table 7. If not overwritten, it is taken from the Table 7 of the code, "SP 16.13330.2011."
Column Buckling Curve (y-y)	a, b, or c	Program determined	Column buckling curve to be used. This is either "a", "b", or "c". It determines the imperfection factors for buckling curve. It is used to find the alpha and beta factors, and to eventually delta and phi factors, for compression capacity per SP 16.13330.2011 7.1.3, Table 7. If not overwritten, it is taken from the Table 7 of the code, "SP 16.13330.2011."

**Table 3-2 Steel Frame Design Overwrites**

Item	Possible Values	Default Value	Description
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), as both.
DL Limit, L/	$\geq 0$	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/	$\geq 0$	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/	$\geq 0$	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/	$\geq 0$	Program Determined	Deflection limit for total load. Inputting 240 means that the limit is L/240. Inputting zero means no check will be made of this item.
Total-Camber Limit, L/	$\geq 0$	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	$\geq 0$	Program Determined	Deflection limit for dead load. Inputting zero means no check will be made of this item.
Super DL+LL Limit, abs	$\geq 0$	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	$\geq 0$	Program Determined	Deflection limit for superimposed live load. Inputting zero means no check will be made of this item.
Total Limit, abs	$\geq 0$	Program Determined	Deflection limit for total load. Inputting zero means no check will be made of this item.
Total-Camber Limit, abs	$\geq 0$	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	$\geq 0$	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	$\geq 0$	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.

**Table 3-2 Steel Frame Design Overwrites**

Item	Possible Values	Default Value	Description
Live Load Reduction Factor	$\geq 0$	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)	$\geq 0$	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)	$\geq 0$	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)	$\geq 0$	Program Determined	Unbraced length factor for lateral-torsional buckling for the frame object; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Effective Length Factor Braced ( $\mu_{braced}$ Major)	$\geq 0$	Program Determined	Effective length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the $B_1$ factor.
Effective Length Factor Braced ( $\mu_{braced}$ Minor)	$\geq 0$	Program Determined	Effective length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the $U_1$ factor.
Effective Length Factor Unbraced ( $\mu_{unbraced}$ Major)	$\geq 0$	Program Determined	Effective length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the $B_1$ factor.

**Table 3-2 Steel Frame Design Overwrites**

Item	Possible Values	Default Value	Description
Effective Length Factor Unbraced ( $\mu_{unbraced}$ Minor)	$\geq 0$	Program Determined	Effective length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the $U_1$ factor.
Effective Length Factor (LTB)	$\geq 0$	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.
Design Yield Strength, $R_y$	$\geq 0$	Program Determined	Material yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Design Fracture Strength, $R_u$	$\geq 0$	Program Determined	Material yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Design Shear Strength, $R_u$	$\geq 0$	Program Determined	Material yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Demand/Capacity Ratio Limit	$\geq 0$	Program Determined	The stress ratio limit to be used for acceptability. Stress ratios that are less than or equal to this value are considered acceptable. Program determined value means it is taken from the steel preferences. Specifying 0 means the value is program determined.

### 3.4 Design Loading Combinations

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

For the SP 16.13330.2011 code, the following default design combinations are generated by the program (SP20 6.2, 6.4, 7.3, Table 7.1, 7.4, 8.2.2, Table 8.2, 10.12, 11.1.12):

1.1 DL	(SP20 6.2(a))
1.1 DL + (1.3*1.0) LL + (1.4*0.9) SL	(SP20 6.2(a))
1.1 DL + (1.3*0.9) LL + (1.4*1.0) SL	(SP20 6.2(a))
0.9 DL ± (1.4*1.0) WL	(SP20 6.2(a))
1.1 DL + (1.3*1.0) LL ± (1.4*0.9) WL	(SP20 6.2(a))
1.1 DL + (1.4*0.9) SL ± (1.4*1.0) WL	(SP20 6.2(a))
1.1 DL + (1.3*0.9) LL + (1.4*1.0) SL ± (1.4*0.7) WL	(SP20 6.2(a))
0.9 DL ± (1.0*1.0) EL	(SP20 6.2(b))
1.1 DL + (1.3*0.8) LL + (1.4*0.8) SL ± (1.3*1.0) EL	(SP20 6.2(b))

The combinations described herein are the default loading combinations only. They can be deleted or edited as required by the design code or engineer-of-record. The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

### 3.5 Classification of Sections for Local Buckling

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

Sections are classified as Compact, Noncompact, or Slender sections in accordance with the code (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10). For a section to qualify as Compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios  $\lambda_p$  from Tables 9 and 10 of the code. If the width-thickness ratio of one or more compression elements exceeds  $\lambda_p$ , but does not exceed  $\lambda_r$  from Tables 9 and 10, the section is Noncompact. If the width-thickness ratio of any element exceeds  $\lambda_r$  but does not exceed  $\lambda_s$ , the



section is Slender. If the width-thickness ratio of any element exceed  $\lambda_s$ , the section is considered Too Slender. The expressions of  $\lambda_p$ ,  $\lambda_r$ , and  $\lambda_s$ , as implemented in the program, are reported in Table 3-1 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10). The limit demarcating Slender and Too Slender has been identified as  $\lambda_s$  in this document.

For compression, sections are classified as nonslender element or slender element sections as reported in Table 3-2 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10). For a nonslender element section, the width-to-thickness ratios of its compression elements shall not exceed  $\lambda_r$  from Table 3-2. If the width-to-thickness ratio of any compression element exceeds  $\lambda_r$ , the section is a slender element section.

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

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

- (a) For flanges of I-shaped members and tees, the width  $b$  is one-half the full-flange width,  $b_f$ .
- (b) For legs of angles and flanges of channels and zees, the width  $b$  is the full nominal dimension.
- (c) For plates, the width  $b$  is the distance from the free edge to the first row of fasteners or line of welds.
- (d) For stems of tees,  $d$  is taken as the full nominal depth of the section.

Refer to Table 3-1 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10) for the graphic representation of unstiffened element dimensions.

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

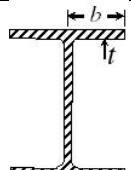
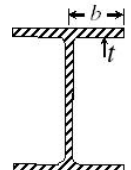
- (a) For webs of rolled or formed sections,  $h$  is the clear distance between flanges less the fillet or corner radius at each flange;  $h_c$  is twice the distance from

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

- (b) For webs of built-up sections,  $h$  is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and  $h_c$  is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used;  $h_p$  is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
- (c) For flange or diaphragm plates in built-up sections, the width  $b$  is the distance between adjacent lines of fasteners or lines of welds.
- (d) For flanges of rectangular hollow structural sections (HSS), the width  $b$  is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS,  $h$  is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known,  $b$  and  $h$  shall be taken as the corresponding outside dimension minus three times the thickness.

Refer to Table 3-1 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10) for the graphic representation of stiffened element dimensions.

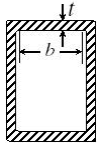
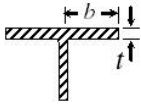
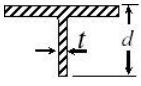
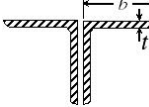
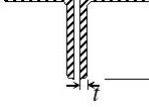
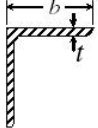
**Table 3-3 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections Subjected to Axial Compression**

Section Type	Description of Element	Example	Eqn.	Width-Thickness Ratio, $(\lambda)$	Limiting Width-Thickness Ratios for Compression Element
					NonSlender Limit $(\bar{\lambda}_{inv}, \bar{\lambda}_{uf})$
Doubly Symmetric I-Shape	Axial only compression in flanges of rolled I-Shapes		37	$b_f/2t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.36 + 0.10 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.36 + 0.10 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.36 + 0.10 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Axial only compression in flanges of built-up I-Shapes		37	$b_f/2t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.36 + 0.10 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.36 + 0.10 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.36 + 0.10 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$

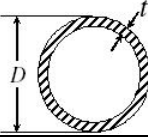
**Table 3-3 Limiting Width-Thickness Ratios of Compression Elements for Classification Sections Subjected to Axial Compression**

Section Type	Description of Element	Example	Eqn.	Width-Thickness Ratio, $(\lambda)$	Limiting Width-Thickness Ratios for Compression Element
					NonSlender Limit $(\bar{\lambda}_{uw}, \bar{\lambda}_{uf})$
	Web in axial only compression		23, 34	$h/t_w$	$\bar{\lambda}_{uw} = \begin{cases} 1.30 + 0.15 \bar{\lambda}^2, & \text{if } \bar{\lambda} \leq 2, \\ 1.20 + 0.35 \bar{\lambda} \leq 2.3, & \text{if } \bar{\lambda} > 2. \end{cases}$
Singly Symmetric I-Shapes	Axial only compression in flanges of rolled I-Shapes		37	$b_f/2t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.36 + 0.10 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.36 + 0.10 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.36 + 0.10 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Axial only compression in flanges of built-up I-Shapes		37	$b_f/2t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.36 + 0.10 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.36 + 0.10 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.36 + 0.10 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
Channel	Axial only compression in flanges		38	$b_f/t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.43 + 0.08 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.43 + 0.08 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.43 + 0.08 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Web in axial only compression		27, 28	$h/t_w$	$\bar{\lambda}_{uw} = \begin{cases} 1.0, & \text{if } \bar{\lambda} \leq 0.8, \\ 0.85 + 0.19 \bar{\lambda} \leq 1.6, & \text{if } \bar{\lambda} > 0.8. \end{cases}$
Double Channel	Axial only compression in flanges		38	$b_f/t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.43 + 0.08 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.43 + 0.08 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.43 + 0.08 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Web in axial only compression		27, 28	$h/t_w$	$\bar{\lambda}_{uw} = \begin{cases} 1.0, & \text{if } \bar{\lambda} \leq 0.8, \\ 0.85 + 0.19 \bar{\lambda} \leq 1.6, & \text{if } \bar{\lambda} > 0.8. \end{cases}$

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

Section Type	Description of Element	Example	Eqn.	Width-Thickness Ratio, $(\lambda)$	Limiting Width-Thickness Ratios for Compression Element
					NonSlender Limit $(\bar{\lambda}_{nw}, \bar{\lambda}_{uf})$
Box	Axial compression		25, 26	$b/t$	$\bar{\lambda}_{uf} = \begin{cases} 1.2, & \text{if } \bar{\lambda} \leq 1, \\ 1.0 + 0.20 \bar{\lambda} \leq 1.6, & \text{if } \bar{\lambda} > 1. \end{cases}$
				$h/t_w$	$\bar{\lambda}_{nw} = \begin{cases} 1.2, & \text{if } \bar{\lambda} \leq 1, \\ 1.0 + 0.20 \bar{\lambda} \leq 1.6, & \text{if } \bar{\lambda} > 1. \end{cases}$
T-Shape	Axial compression in flanges		37	$b_f/2t_f$	$\bar{\lambda}_{uf} = \begin{cases} (0.36 + 0.10 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.36 + 0.10 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.36 + 0.10 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Compression in stems		29	$d/t_w$	$\bar{\lambda}_{nw} = \begin{cases} (0.85 + 0.19 \times 0.8) \left( 1 + 0.25 \sqrt{2 - \frac{b_f}{h_{ef}}} \right), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.85 + 0.19 \bar{\lambda}) \left( 1 + 0.25 \sqrt{2 - \frac{b_f}{h_{ef}}} \right), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.85 + 0.19 \times 4) \left( 1 + 0.25 \sqrt{2 - \frac{b_f}{h_{ef}}} \right), & \text{if } \bar{\lambda} > 4. \end{cases}$
Double Angle	Any type of compression in leg		39	$b/t$	$\bar{\lambda}_{uf} = \begin{cases} (0.40 + 0.19 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.40 + 0.19 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.40 + 0.19 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
	Any type of compression in leg		39	$b/t$	$\bar{\lambda}_{uf} = \begin{cases} (0.40 + 0.19 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.40 + 0.19 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.40 + 0.19 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$
Angle	Axial only compression in any leg		39	$b/t$	$\bar{\lambda}_{uf} = \begin{cases} (0.40 + 0.19 (0.8)), & \text{if } \bar{\lambda} \leq 0.8, \\ (0.40 + 0.19 \bar{\lambda}), & \text{if } 0.8 \leq \bar{\lambda} \leq 4, \\ (0.40 + 0.19 (4.0)), & \text{if } \bar{\lambda} > 4. \end{cases}$

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

Section Type	Description of Element	Example	Eqn.	Width-Thickness Ratio, $(\lambda)$	Limiting Width-Thickness Ratios for Compression Element
					NonSlender Limit $(\bar{\lambda}_{inv}, \bar{\lambda}_{uf})$
Pipe	Axial only compression		A I S C	$D/t$	$0.11 E/R_y$
Round Bar	-----	-----	---	-----	Assumed NonSlender
Rectan-gular	-----	-----	---	-----	Assumed NonSlender
General	-----	-----	---	-----	Assumed NonSlender
SD Section	-----	-----	---	-----	Assumed NonSlender

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.

For the determination of the nominal strengths for axial compression and flexure, the sections are classified as Class 1 (Plastic), Class 2 (Compact), Class 3 (Noncompact), or Class 4 (Slender). The program classifies the individual sections in accordance with Table 3-3 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10). As specified in that table, a section is classified as Class 1, Class 2, Class 3 or Class 4 as applicable.

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

- For flanges of I-shaped members and tees, the width  $b_{el}$  is one-half the full nominal dimension,  $b_f$ .
- For legs of angles and flanges of channels and zeas, the width  $b_{el}$  is the full nominal dimension.
- For plates, the width  $b_{el}$  is the distance from the free edge to the first row of fasteners or line of welds.
- For webs of hot rolled sections,  $h$  is the clear distance between flanges.

Refer to Table 3-3 (SP16 7.1.3, 7.3.3, 7.3.11, Table 9, Table 10) for the graphic representation of element dimensions.

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

- (a) For webs of rolled or formed sections,  $h$  is the clear distance between flanges less the fillet or corner radius at each flange;  $h_c$  is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
- (b) For webs of built-up sections,  $h$  is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used.
- (c) For flange or diaphragm plates in built-up sections, the width  $b$  is the distance between adjacent lines of fasteners or lines of welds.
- (d) For flanges of rectangular hollow structural sections (HSS), the width  $b$  is the nominal outside dimension less four times the wall thickness. For webs of rectangular HSS,  $h$  is the nominal outside dimension less four times the wall thickness. The thickness,  $t$ , shall be taken as the design wall thickness.

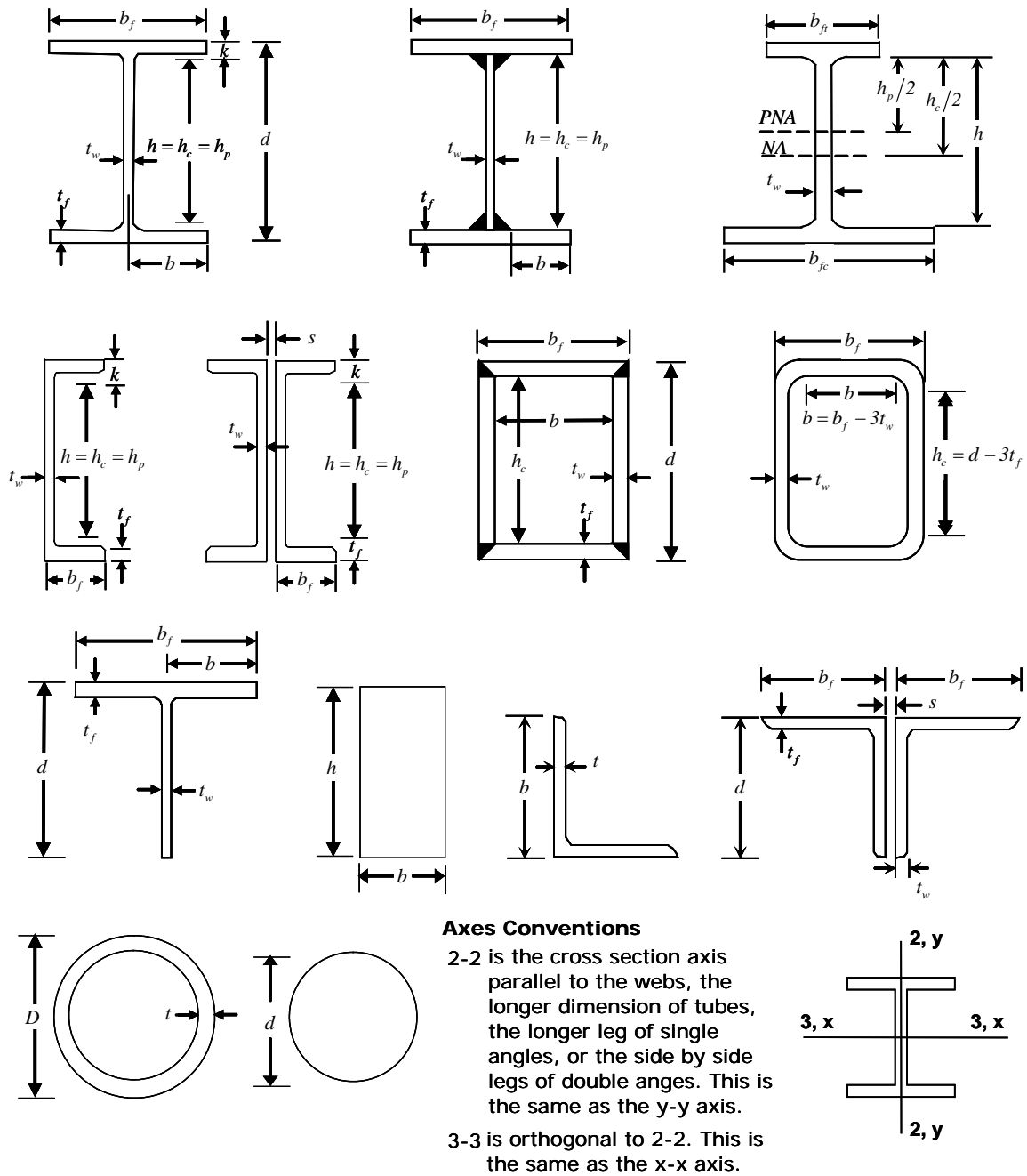


Figure 3-1 SP 16.13330.2011 Definition of Geometric Properties

In classifying web slenderness of I-Shapes, Box, Channel, Double Channel, and all other sections, it is assumed that there are no intermediate stiffeners. Double Angles and Channels are conservatively assumed to be separated. When slenderness parameters are larger than the limits for Class 4 sections, the section is classified as Too Slender. **Stress check of Too Slender sections is beyond the scope of this program.**

### 3.6 Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are  $N$ ,  $M_x$  (or  $M_{33}$ ),  $M_y$  (or  $M_{22}$ ),  $Q_x$  (or  $Q_{22}$ ),  $Q_y$  (or  $Q_{33}$ ), and  $T$  corresponding to factored values of the axial compression, the major and minor moments and shears, and torsion, respectively. These factored loads are calculated at each of the previously defined stations.

### 3.7 Calculation of Design Strengths

For *all* sections, the design strengths for tension, compression, flexure, fracture, and shear are calculated based on SP16 6.1 as following.

$$R_y = \frac{R_{yn}}{\gamma_m} \quad (\text{SP16 6.1, Table 2})$$

$$R_u = \frac{R_{un}}{\gamma_m} \quad (\text{SP16 6.1, Table 2})$$

$$R_s = 0.58 \frac{R_{yn}}{\gamma_m} \quad (\text{SP16 6.1, Table 2})$$

where,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_u$  is the design fracture strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_s$  is the design strength for shear (SP16 6.1, Table 2),



$R_{yn}$  is the characteristic yield strength (SP16 6.1, Table C.5, C.6),

$R_{um}$  is the characteristic fracture strength (SP16 6.1, Table C.5, C.6),

$\gamma_m$  is the partial safety factor for material strength (SP16 6.1, Table 3).

The partial safety factor for material strength,  $\gamma_m$ , can be chosen for different materials satisfying the state standards and specifications for the rolled structural shapes, sheets, and pipes from the SP16 Table 3 (SP16 6.1, Table 3). However, the program allows the user to choose the value of  $\gamma_m$  in the preferences form. Its default value is 1.025 which is typical for rolled structural shapes (SP16 Table C.5).

The characteristic yield and fracture strengths,  $R_{yn}$  and  $R_{um}$ , respectively, can be chosen for different materials and different thicknesses satisfying the state standards and specifications for the rolled structural shapes, sheets, and pipes from the SP16 Tables C.5 and C.6. However, the program allows the user to choose the value of  $R_{yn}$  and  $R_{um}$  directly and expects to be input in the materials definition form.

The design yield and fracture strengths,  $R_y$  and  $R_u$ , respectively, are calculated by dividing characteristic strength values by reliability coefficients of the material using the equation given above. Unlike in the SP16 Tables C5 and C6, these values are NOT rounded up to 5 N/mm<sup>2</sup> in the program.

### 3.8 Checking Members for Combination Forces

This section addresses the checking of members subject to individual forces or a combination of two or more of the individual forces. The member state of failure is represented by the demand/capacity (D/C) ratios.

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.

The design flexural resistance for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I-Shape, Box, Channel, Double Channel, Circular, Pipe, T-Shape, and Double Angle sections, the principal axes coincide with their geometric axes. For the Single Angle sections, the principal axes are determined and all computations except shear are based on that.

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

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

### 3.8.1 Members Subjected to Axial Compression with or without Flexure

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

#### 3.8.1.1 Members Subjected to Axial Compression Only

If the axial load is compressive, the D/C ratio for sections with standard strength ( $R_{yn} \leq 440 \text{ N/mm}^2$ ) is given by:

$$\frac{N}{A_n R_y \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.1})$$

If the axial load is compressive, the D/C ratio for sections with higher characteristic strength ( $R_{yn} > 440 \text{ N/mm}^2$ ) is given by:

$$\frac{N}{A_n (R_u / \gamma_u) \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.1})$$

If the axial load is compressive, the D/C ratio for all sections is given by:

$$\frac{N}{\phi A R_y \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.3})$$

where,

$N$  is the factored axial compressive loading, including  $P$ - $\Delta$  effects,

$A$  is the gross area of cross section,

$A_n$  is the net area of cross section, an overwrite controls the ratio of net to gross area of the cross section,  $A_n/A$ ,

$R_{ym}$  is the characteristic yield strength (SP16 6.1, Table C.5, C.6, C.7),

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_u$  is the design fracture strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1, 16.1, Table 45),

$\gamma_u$  is the safety factor for structure members designed in terms of their behavior with the use of design strengths  $R_u$  (SP16 4.3.2),

$\phi$  is the stability coefficient at axial compression determined using equations as given in SP16 7.1.3.

For all types of cross-sections, the stability coefficient at axial compression,  $\phi$ , is determined as follows.

$$\phi = \begin{cases} 0.5 \left[ \frac{\delta - \sqrt{\delta^2 - 39.48 \bar{\lambda}^{-2}}}{\bar{\lambda}^2} \right] \leq \frac{7.6}{\bar{\lambda}}, & \text{if } \bar{\lambda} > 0.4, \\ 1, & \text{if } \bar{\lambda} \leq 0.4. \end{cases}, \quad (\text{SP16 7.1.3})$$

where,

$$\delta = 9.87(1 - \alpha + \beta\bar{\lambda}) + \bar{\lambda}^2, \quad (\text{SP16 7.1.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

$$\alpha = \begin{cases} 0.03, & \text{for Pipe and box shapes,} \\ 0.04, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.04, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections, and} \end{cases}$$

(SP16 7.1.3, Table 7)

$$\beta = \begin{cases} 0.06, & \text{for Pipe and box shapes,} \\ 0.09, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.14, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections.} \end{cases}$$

(SP16 7.1.3, Table 7)

### 3.8.1.2 Members Subjected to Flexure Only

For Class 1 members, if the member is loaded with moment in one of the principal plane only, the D/C ratio is given by:

$$\frac{M_x}{W_{xn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

$$\frac{M_y}{W_{yn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

For Class 1 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_x y}{I_{xn} R_y \gamma_c} \pm \frac{M_y x}{I_{yn} R_y \gamma_c} \pm \frac{B \omega}{I_{\omega} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

For Class 2 and Class 3 members, if the member is loaded with moment in one of the principal plane only, the D/C ratio is given by:

$$\frac{M_x}{c_x \beta W_{xn, \min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and Class 3} \quad (\text{SP16 8.2.3})$$

$$\frac{M_y}{c_y W_{yn, \min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and Class 3} \quad (\text{SP16 8.2.3})$$

For Class 2 and Class 3 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_x}{c_x \beta W_{xn, \min} R_y \gamma_c} + \frac{M_y}{c_y W_{yn, \min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and 3} \quad (\text{SP16 8.2.3})$$

For Class 1 members, if the member is loaded with moment in the major principal plane only, the D/C ratio is given by:

$$\frac{M_x}{\phi_b W_{cx} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.4.1})$$

For Class 1 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_x}{\phi_b W_{cx} R_y \gamma_c} + \frac{M_y}{W_{cy} R_y \gamma_c} + \frac{B}{W_{\omega} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.4.1})$$

where,

$M_x$  is the factored moment about the major principal axis,  $x$ ,

$M_y$  is the factored moment about the minor principal axis,  $y$ ,

$B$  is the bi-moment. It is assumed to be zero throughout the program,

$W_{cx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the compression flange,

$W_{cy}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the compression side,

$W_{xn,min}$  is the minimum section modulus of cross section about the principal axis,  $x$ ,

$W_{yn,min}$  is the minimum section modulus of cross section about the principal axis,  $y$ ,

$I_{xn}$  is the moment of inertia of cross section about the principal axis,  $x$ ,

$I_{yn}$  is the moment of inertia of cross section about the principal axis,  $y$ ,

$I_{\omega}$  is the cross-sectional sectorial moment of inertia. It is not really used in the program as the bi-moment,  $B$ , is assumed to be zero,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$c_x$  and  $c_y$  are the coefficients for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1),

$\beta$  is the coefficient that takes the effect of average shear stress on the moment capacity for Class 2 and Class 3 sections,

$\phi_b$  is the stability factor in bending to be calculated based on SP16 Annex G.

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

The coefficients  $c_x$  and  $c_y$  for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections for doubly-symmetric I-Shape, singly-symmetric I-Shape, Channel, double-Channel, Box, Pipe, Tee, Angle, Double-Angle, and Solid Rectangular shapes are calculated based on the SP16 Table F.1 given in Annex F. The values of  $c_x$  and  $c_y$  are interpolated for  $A_f/A_w$  when needed. For all other shapes including Solid Circular sections and SD-Sections,  $c_x$  and  $c_y$  are calculated assuming the shape is similar to the solid rectangular section as for the item 5 in the table.

The coefficient,  $\beta$ , that takes the effect of average shear stress on the moment capacity for Class 2 and Class 3 sections is calculated as follows:

$$\beta = \begin{cases} 1, & \text{if } 0 \leq \frac{\tau_x}{R_s} \leq 0.5, \\ 1 - \left[ \frac{0.2}{\alpha_f + 0.25} \right] \left( \frac{\tau_x}{R_s} \right)^4, & \text{if } 0.5 \leq \frac{\tau_x}{R_s} \leq 0.9, \end{cases}, \quad (\text{SP16 8.5.18})$$

where,

$$\tau_x = \frac{Q_x}{A_w}, \quad (\text{SP16 8.2.3})$$

$$\alpha_f = \frac{A_f}{A_w}, \quad (\text{SP16 8.2.3})$$

$R_s$  is the design shear strength (SP16 6.1, Table 2),

$A_f$  is the area of a single flange. For singly-symmetric I-shape it is the area of the smaller flange (SP16 8.2.3), and

$A_w$  is the area of area of the webs. For box sections it is the area of the two webs together (SP16 8.2.3).

The stability factor in bending,  $\phi_b$ , is calculated based on SP16 Annex G. The coefficient  $\phi_b$  is determined depending on the arrangement of supports, unbraced compressed zone, and type of load applications. It is assumed that the load is applied in the plane of minor axis strength--in the plane of maximum strength, and that the supporting section fixed against lateral displacement and rotation. For members with doubly-symmetric I-sections the coefficient  $\phi_b$  is calculated as follows. For all other cases see the section SP16 Annex G.

$$\phi_b = \begin{cases} \phi_1, & \text{if } \phi_1 \leq 0.85, \\ 0.68 + 0.21\phi_1, & \text{if } \phi_1 > 0.85, \end{cases} \quad (\text{SP16 } 8.4.1, \text{ G.1})$$

where,

$$\phi_1 = \psi \frac{I_y}{I_x} \left( \frac{h}{l_{ef}} \right)^2 \frac{E}{R_y}, \quad (\text{SP16 G.1})$$

$h$  is full height of the rolled cross-section (SP16 G.1),

$l_{ef}$  is the estimated length of the member determined in accordance with the requirements of SP16 8.4.2. It is the distance between two bracing points in the compression zone (SP16 8.2.3, G.1),

$\psi$  is the coefficient determined using the section SP16 G.3 (SP16 G.1). The value of coefficient  $\psi$  in the above formula is calculated by formulas of SP16 Tables G.1 and G.2 depending on the number of bracing points of the compressed zone, type of load space, and its applications, as well as on the coefficient  $\alpha$ , which is given below,

$$\alpha = 1.54 \frac{I_t}{I_y} \left( \frac{l_{ef}}{h} \right)^2, \quad (\text{SP16 G.4})$$



### 3.8.1.3 Members Subjected to Flexure and Compression

If the axial load is compressive, the section characteristic yield strength is less than 440 N/mm<sup>2</sup> ( $R_{ym} \leq 440$  N/mm<sup>2</sup>), the average shear stress for major and minor principal directions are small ( $\tau_x/R_s \leq 0.5$  and  $\tau_y/R_s \leq 0.5$ ), and significant axial force ( $N/A_n > 0.1$ ), the D/C ratio is given by:

$$\left( \frac{N}{A_n R_y \gamma_c} \right)^n + \frac{M_x}{c_x W_{xn, \min} R_y \gamma_c} + \frac{M_y}{c_y W_{yn, \min} R_y \gamma_c} + \frac{B}{W_{\omega n, \min} R_y \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.1})$$

If the axial load is compressive but the above conditions are not met, the D/C ratio is given by:

$$\frac{N}{A_n R_y \gamma_c} \pm \frac{M_x y}{I_{xn} R_y \gamma_c} \pm \frac{M_y x}{I_{yn} R_y \gamma_c} \pm \frac{B \omega}{I_{\omega n} R_y \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.1})$$

If the axial load is compressive, the section characteristic yield strength is more than 440 N/mm<sup>2</sup> ( $R_{ym} > 440$  N/mm<sup>2</sup>), the D/C ratio considering the tension flange yielding for singly symmetric sections including singly-symmetric I-shape, Channel, Tee, and Double Angle sections is given by the following:

$$\frac{N}{A_n (R_u / \gamma_u) \gamma_c} - \frac{M_x}{\delta W_{tx} (R_u / \gamma_u) \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.3})$$

where,

$M_x$  is the factored moment about the major principal axis,  $x$ ,

$M_y$  is the factored moment about the minor principal axis,  $y$ ,

$B$  is the bi-moment. It is assumed to be zero throughout the program,

$W_{cx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the compression flange,

$W_{cy}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the compression side,

$W_{tx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the tension flange,

$W_{ty}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the tension side,

$W_{xn,\min}$  is the minimum section modulus of cross section about the principal axis,  $x$ ,

$W_{yn,\min}$  is the minimum section modulus of cross section about the principal axis,  $y$ ,

$I_{xn}$  is the moment of inertia of cross section about the principal axis,  $x$ ,

$I_{yn}$  is the moment of inertia of cross section about the principal axis,  $y$ ,

$I_{\omega}$  is the cross-sectional sectorial moment of inertia. It is not really used in the program as the bi-moment,  $B$ , is assumed to be zero,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_u$  is the design fracture strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$\gamma_u$  is the safety factor for structure members designed in terms of their behavior with the use of design strengths  $R_u$  (SP16 4.3.2),

$c_x$  and  $c_y$  are the coefficients for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1) and it is described in more details in the previous section,

$n$  is the coefficient that is used as a power for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1),

$\delta$  is the amplification coefficient that is used to amplify the moment,

$$\delta = 1 - 0.1 \left( \frac{N}{AR_y / \bar{\lambda}^2} \right), \quad (\text{SP16 9.1.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

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

#### 3.8.1.4 Member Stability Subjected to Flexure and Compression

The design of beam column stability loaded in one of principal moment planes is carried out both in plane (plane mode of stability loss) and out of plane (flexural-torsional instability) with the following interaction equations.

If the axial load is compressive, the D/C ratio considering the integral beam-column stability in moment plane coinciding with symmetry plane is given by the following:

$$\frac{N}{\phi_{ex} AR_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.2})$$

$$\frac{N}{\phi_{ey} AR_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.2})$$

$$\frac{N}{\phi_x AR_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.8})$$

If the axial load is compressive, the D/C ratio considering the integral beam-column stability in out of moment plane is given by the following:

$$\frac{N}{c\phi_y AR_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.4})$$

If the axial load is compressive and the member is bent under bi-axial moments in two principal planes, the D/C ratio considering the integral beam-column stability is given by the following:

$$\frac{N}{\phi_{exy} AR_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.9})$$

If the axial load is compressive and the member is bent under uniaxial moment or bi-axial moments in two principal planes, the D/C ratio considering the beam-column stability is given by the following:

$$\frac{N}{\phi_{ey} AR_y \gamma_c} + \frac{M_x}{c_x \delta_x W_{xn, \min} R_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.10})$$

$$\frac{N}{\phi_{ex} AR_y \gamma_c} + \frac{M_y}{c_y \delta_x W_{yn, \min} R_y \gamma_c} \leq 1.0 \quad (\text{SP16 9.2.10})$$

where,

$N$  is the factored axial compressive loading, including  $P-\Delta$  effects,

$A$  is the gross area of cross section,

$A_n$  is the net area of cross section, an overwrite controls the ratio of net to gross area of the cross section,  $A/A_n$ ,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$\phi$  is the stability coefficient at axial compression determined using equations as given in SP16 7.1.3. It can have two values,  $\phi_x$  and  $\phi_y$ , for buckling about two principal axes,

$\phi_{ex}$  is the stability coefficient for axial compression with uniaxial buckling about the major principal axis,  $x$ , determined using SP16 9.2.1 and Table E.3,

$\phi_{ey}$  is the stability coefficient for axial compression with uniaxial buckling about the minor principal axis,  $y$ , determined using SP16 9.2.1 and Table E.3,

$\phi_x$  is the stability coefficient at axial compression for buckling about the major principal axis,  $x$ , determined using equations as given in SP16 7.1.3,

$\phi_y$  is the stability coefficient at axial compression for buckling about the minor principal axis,  $y$ , determined using equations as given in SP16 7.1.3,

$\phi_{exy}$  is the stability coefficient for axial compression with biaxial bending determined using equations as given in SP16 7.1.3,

$c$  is the stability related coefficient used for lateral-torsional buckling stability check and determined using equations as given in SP16 9.2.5,

$\delta_x$  and  $\delta_y$  are the moment amplification factors for axial compression determined using equations as given in SP16 9.2.10 as follows:

$$\delta_x = \begin{cases} 1, & \text{if } \bar{\lambda}_x \leq 1, \\ 1 - 0.1 \bar{\lambda}_x^2 \frac{N}{AR_y}, & \text{if } \bar{\lambda}_x > 1, \end{cases} \quad (\text{SP16 9.2.5})$$

$$\delta_y = \begin{cases} 1, & \text{if } \bar{\lambda}_y \leq 1, \\ 1 - 0.1 \bar{\lambda}_y^2 \frac{N}{AR_y}, & \text{if } \bar{\lambda}_y > 1, \end{cases} \quad (\text{SP16 9.2.5})$$

The stability coefficients for axial compression with uniaxial buckling,  $\phi_{ex}$  and  $\phi_{ey}$ , about the major and minor principal axis,  $x$  and  $y$ , respectively, are determined from the Table SP16 E.3 based on  $\bar{\lambda}$  and  $m_{ef}$  for the appropriate direction (SP16 9.2.2, Table E.3).

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

$$m_{ef} = \eta m, \quad (\text{SP16 9.2.2})$$

$$m = \frac{(M/W_c)}{(N/A)} = \frac{eA}{W_c}, \quad (\text{SP16 9.2.2})$$

$$e = \frac{M}{N}, \quad (\text{SP16 9.2.2})$$

$W_c$  is the section modulus of cross section about the appropriate axis computed for the compression flange,

$\eta$  is the influence coefficient for cross-sectional shape as given in SP16 Table E.2,

$N$  is the factored axial compressive loading in the member taken for the same load combination as that for  $M$ . It can differ slightly from station to station, and

$M$  is the factored equivalent moment about the appropriate axis of bending axis. It is determined as follows.

When the member is loaded in-span, the factored equivalent moment,  $M$ , about the appropriate axis of bending is calculated based on SP16 Table 20 as follows:

$$M = \begin{cases} M_2 > 0.5M_{\max}, & \text{if } \bar{\lambda} < 4, \text{ and } 0 \leq m_{\max} \leq 3, \\ M_2 + \frac{m_{\max} - 3}{17}(M_{\max} - M_2), & \text{if } \bar{\lambda} < 4, \text{ and } 3 < m_{\max} \leq 20. \\ M_1 & \text{if } \bar{\lambda} > 4, \text{ and } 0 \leq m_{\max} \leq 3, \\ M_1 + \frac{m_{\max} - 3}{17}(M_{\max} - M_1), & \text{if } \bar{\lambda} > 4, \text{ and } 3 < m_{\max} \leq 20, \end{cases}$$

(SP167.1.3)

where,

$M_{\max}$  is the maximum bending moment within the member length,

$M_1$  is the maximum bending moment within the middle third of the member length, but not less than  $0.5M_{\max}$ ,

$M_2 = M_{\max} - 0.25\bar{\lambda}(M_{\max} - M_1) > 0.5M_{\max}$ , and

$$m_{\max} = \frac{(M_{\max}/W_c)}{(N/A)} = \frac{e_{\max}A}{W_c}.$$

When the member is not loaded in-span, the factored equivalent moment,  $M$ , is not needed to calculate. In this case the value of  $m_{ef}$  about the appropriate axis of bending is calculated from the SP16 Table E.5 using the parameters  $m_{ef,1}$ ,  $\delta$ , and  $\bar{\lambda}$  where the parameters are defined as follows:

$$m_{ef,1} = \eta \frac{M_1}{N} \frac{A}{W_c}, \quad \text{(SP16 Table E.5)}$$

$$\delta = \frac{M_2}{M_1}, \quad \text{(SP16 Table E.5)}$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad \text{(SP16 7.1.3)}$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

$M_1$  is the maximum of the two end moment in the appropriate axis of bending (SP16 Table E.5),

$M_1$  is the minimum of the two end moment in the appropriate axis of bending (SP16 Table E.5).

The influence coefficient,  $\eta$ , for cross-sectional shape is calculated for different sections from the SP16 Table E.2 using the parameters  $A_f/A_w$ ,  $\bar{\lambda}$ , and  $m$  where the parameters are defined as follows:

$$m = m_{\max} = \frac{(M_{\max}/W_c)}{(N/A)} = \frac{e_{\max} A}{W_c} \quad (\text{SP16 9.2.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \text{ and} \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}. \quad (\text{SP16 7.1.3})$$

In the above expression, the terms  $\bar{\lambda}$ ,  $\lambda$ ,  $l_{ef}$ ,  $r$ ,  $K$ ,  $m_{ef}$ ,  $m$ ,  $e$ ,  $M$ ,  $W_c$ , and  $\eta$  are to be used with appropriate direction.

The Table SP16 E.3 provides the value of  $\phi_e$  for the range of  $0.1 \leq m_{ef} \leq 20$ . If  $m_{ef} \leq 0.1$  then the moment is insignificant. In this case this limit state is not needed to be checked as other interaction equation involving predominantly axial force will cover this case. If  $m_{ef} > 20$  then the axial force is insignificant. In this case this limit state is not needed to be checked as other interaction equation involving predominantly bending moment will cover this case (SP16 9.2.2). However the interaction equation is checked anyway with the assumption that  $\phi_e = \phi$  where  $\phi_e$  is calculated per SP16 7.1.3.



The stability coefficients for axial compression,  $\phi_y$ , for buckling about the minor principal axis,  $y$ , is determined from the SP16 7.1.3 based on  $\bar{\lambda} = \bar{\lambda}_y = \bar{\lambda}_{22}$  (SP16 9.2.4, 7.1.3) as follows:

$$\phi = \begin{cases} 0.5 \left[ \frac{\delta - \sqrt{\delta^2 - 39.48\bar{\lambda}^2}}{\bar{\lambda}^2} \right] \leq \frac{7.6}{\bar{\lambda}^2}, & \text{if } \bar{\lambda} > 0.4, \\ 1, & \text{if } \bar{\lambda} \leq 0.4. \end{cases}, \quad (\text{SP16 7.1.3})$$

where,

$$\delta = 9.87(1 - \alpha + \beta\bar{\lambda}) + \bar{\lambda}^2, \quad (\text{SP16 7.1.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

$$\alpha = \begin{cases} 0.03, & \text{for Pipe and box shapes,} \\ 0.04, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.04, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections, and} \end{cases} \quad (\text{SP16 7.1.3, Table 7})$$

$$\beta = \begin{cases} 0.06, & \text{for Pipe and box shapes,} \\ 0.09, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.14, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections.} \end{cases} \quad (\text{SP16 7.1.3, Table 7})$$

The stability coefficients for axial compression,  $\phi_x$ , for buckling about the major principal axis,  $x$ , is determined from the SP16 7.1.3 based on  $\bar{\lambda} = \bar{\lambda}_x = \bar{\lambda}_{33}$  (SP16 9.2.8, 7.1.3) using the same procedure as stated above.

The stability coefficient for axial compression with biaxial bending,  $\phi_{exy}$ , is determined from the SP16 9.2.9 as follows:

$$\phi_{exy} = \phi_{ey} \left( 0.6\sqrt[3]{c} + 0.4\sqrt[4]{c} \right), \quad (\text{SP16 7.1.3})$$

where,

$\phi_{ey}$  is the stability coefficient for axial compression with uniaxial buckling about the minor principal axis,  $y$ , determined using SP16 9.2.1 and Table E.3 and its calculation procedure has been described earlier in this section,

$c$  is the stability related coefficient used for lateral-torsional buckling stability check and determined using equations as given in SP16 9.2.5.

The stability related coefficient,  $c$ , used for lateral-torsional buckling stability check is determined using equations as given in SP16 9.2.5 as follows:

$$c = \begin{cases} \frac{\beta}{1 + \alpha m_x} \leq 1, & \text{if } m_x \leq 5, \\ c_5(2 - 0.2m_x) + c_{10}(0.2m_x - 1), & \text{if } 5 < m_x \leq 10, \\ \frac{1}{1 + (\phi_y/\phi_b)m_x}, & \text{if } m_x > 10, \end{cases} \quad (\text{SP16 9.2.5})$$

$$c \leq c_{\max} \quad (\text{SP16 9.2.5})$$

where,

$$c_5 = \frac{\beta}{1 + \alpha(5.0)} \leq 1, \quad (\text{SP16 9.2.5})$$

$$c_{10} = \frac{1}{1 + (\phi_y/\phi_b)(5.0)}, \quad (\text{SP16 9.2.5})$$

$$m = \frac{(M_x/W_c)}{(N/A)} = \frac{eA}{W_c}, \quad (\text{SP16 9.2.2, 9.2.5})$$

$$e = \frac{M_x}{N}, \quad (\text{SP16 9.2.2})$$

$\phi_y$  is the stability coefficient at axial compression for buckling about the minor principal axis,  $y$ , determined using equations as given in SP16 7.1.3 and its calculation procedure has been described previously,

$\phi_b$  is the stability factor in bending to be calculated based on SP16 Annex G and its calculation procedure has been described previously,

$M_x$  is the factored equivalent moment about the major principal axis of bending. When the member is loaded in-span, the factored equivalent moment,  $M$ , about the appropriate axis of bending is calculated based on SP16 Table 20 as follows: For members with both ends fixed against lateral movement,  $M_x$  is the maximum moment within a middle third but not less than half in the overall maximum moment. For members with one fixed and one free ends,  $M_x$  is taken the fixed end moment but not less than the moment at a section at a distance of one third length from the fixed end.

$$M_x = \begin{cases} \max(M_{\text{max of middle 1/3 span}}, M_{\text{max of full span}}), & \text{if both ends are supported,} \\ \max(M_{\text{max fixed end}}, M_{\text{third point near support}}) & \text{if only one end is supported,} \end{cases} \quad (\text{SP167.1.3})$$

$\phi_{ey}$  is the stability coefficient for axial compression with uniaxial buckling about the minor principal axis,  $y$ , determined using SP16 9.2.1 and Table E.3 and its calculation procedure has been described earlier in this section,

$c_{\max}$  is the stability related coefficient used for lateral-torsional buckling stability check and determined using equations as given in SP16 E.1.

For all shapes except doubly-symmetric I-shape,  $c_{\max}$ , is taken as

$$c_{\max} = \frac{2}{1 + \delta B + \sqrt{(1 - \delta B)^2 + \frac{16}{\mu} \left( \alpha - \frac{e_x}{h} \right)^2}}, \quad (\text{SP16 E})$$

For all doubly-symmetric I-shapes,  $c_{\max}$ , is taken as

$$c_{\max} = \frac{1 + \frac{I_x}{I_y} + \frac{\alpha}{9.87}}{1 + 4 \left[ \left( \frac{i_x^2 + i_y^2}{h^2} \right) + \frac{e_x}{h} \right]}, \quad (\text{SP16 E})$$

where,

$$\delta = \frac{4\rho}{\mu}, \quad (\text{SP16 E})$$

$$B = 1 + 2 \left( \frac{\beta}{\rho} \right) \left( \frac{e_x}{h} \right), \quad (\text{SP16 E})$$

$$\mu = 8\omega + 0.156 \frac{I_t}{Ah^2} \lambda_y^2, \quad (\text{SP16 E})$$

$\alpha$ ,  $\beta$ , and  $\omega$  are the parameters in the expression  $c_{\max}$ . They are determined using equations as given in Table SP16 E.6.

$\delta_x$  and  $\delta_y$  are the moment amplification factors for axial compression determined using equations as given in SP16 9.2.10.

### 3.8.2 Members Subjected to Axial Tension with or without Flexure

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

#### 3.8.2.1 Members Subjected to Axial Tension Only

If the axial load is tensile, the D/C ratio for sections with standard strength ( $R_{ym} \leq 440 \text{ N/mm}^2$ ) is given by:

$$\frac{N}{A_n R_y \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.1})$$

If the axial load is tensile, the D/C ratio for sections with higher characteristic strength ( $R_{ym} > 440 \text{ N/mm}^2$ ) is given by:

$$\frac{N}{A_n (R_u / \gamma_u) \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.1})$$

If the axial load is tensile, the D/C ratio for all sections is given by:

$$\frac{N}{\phi A R_y \gamma_c} \leq 1.0 \quad (\text{SP16 7.1.3})$$

where,

$N$  is the factored axial tensile loading, including  $P$ - $\Delta$  effects,

$A$  is the gross area of cross section,

$A_n$  is the net area of cross section, an overwrite controls the ratio of net to gross area of the cross section,  $A/A_n$ ,

$R_{ym}$  is the characteristic yield strength (SP16 6.1, Table C.5, C.6, C.7),

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_u$  is the design fracture strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$\gamma_u$  is the safety factor for structure members designed in terms of their behavior with the use of design strengths  $R_u$  (SP16 4.3.2),

$\phi$  is the stability coefficient at axial compression determined using equations as given in SP16 7.1.3.

For all types of cross-sections, the stability coefficient at axial compression,  $\phi$ , is determined as follows.

$$\phi = \begin{cases} 0.5 \left[ \frac{\delta - \sqrt{\delta^2 - 39.48\bar{\lambda}^2}}{\bar{\lambda}^2} \right] \leq \frac{7.6}{\bar{\lambda}^2}, & \text{if } \bar{\lambda} > 0.4, \\ 1, & \text{if } \bar{\lambda} \leq 0.4. \end{cases} \quad (\text{SP16 7.1.3})$$

where,

$$\delta = 9.87(1 - \alpha + \beta\bar{\lambda}) + \bar{\lambda}^2, \quad (\text{SP16 7.1.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

$$\alpha = \begin{cases} 0.03, & \text{for Pipe and box shapes,} \\ 0.04, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.04, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections, and} \end{cases} \quad (\text{SP16 7.1.3, Table 7})$$

$$\beta = \begin{cases} 0.06, & \text{for Pipe and box shapes,} \\ 0.09, & \text{for I-Shape, Double Channel, welded-box,} \\ 0.14, & \text{for Angle, Channel, Tee, Double-Angle,} \\ & \text{and all other sections.} \end{cases} \quad (\text{SP16})$$

7.1.3, Table 7)

### 3.8.2.2 Members Subjected to Flexure Only

For Class 1 members, if the member is loaded with moment in one of the principal plane only, the D/C ratio is given by:

$$\frac{M_x}{W_{xn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

$$\frac{M_y}{W_{yn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

For Class 1 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_{xy}}{I_{xn} R_y \gamma_c} \pm \frac{M_{yx}}{I_{yn} R_y \gamma_c} \pm \frac{B\omega}{I_{\omega} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.2.1})$$

For Class 2 and Class 3 members, if the member is loaded with moment in one of the principal plane only, the D/C ratio is given by:

$$\frac{M_x}{c_x \beta W_{xn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and Class 3} \quad (\text{SP16 8.2.3})$$

$$\frac{M_y}{c_y W_{yn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and Class 3} \quad (\text{SP16 8.2.3})$$

For Class 2 and Class 3 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_x}{c_x \beta W_{xn,\min} R_y \gamma_c} + \frac{M_y}{c_y W_{yn,\min} R_y \gamma_c} \leq 1.0, \quad \text{for Class 2 and 3} \quad (\text{SP16 8.2.3})$$

For Class 1 members, if the member is loaded with moment in the major principal plane only, the D/C ratio is given by:

$$\frac{M_x}{\phi_b W_{cx} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.4.1})$$

For Class 1 members, if the member is loaded with bi-axial moment about both the principal planes, the D/C ratio is given by:

$$\frac{M_x}{\phi_b W_{cx} R_y \gamma_c} + \frac{M_y}{W_{cy} R_y \gamma_c} + \frac{B}{W_{\omega} R_y \gamma_c} \leq 1.0, \quad \text{for Class 1} \quad (\text{SP16 8.4.1})$$

where,

$M_x$  is the factored moment about the major principal axis,  $x$ ,

$M_y$  is the factored moment about the minor principal axis,  $y$ ,

$B$  is the bi-moment. It is assumed to be zero throughout the program,

$W_{cx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the compression flange,

$W_{cy}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the compression side,

$W_{xn,\min}$  is the minimum section modulus of cross section about the principal axis,  $x$ ,

$W_{yn,\min}$  is the minimum section modulus of cross section about the principal axis,  $y$ ,

$I_{xn}$  is the moment of inertia of cross section about the principal axis,  $x$ ,

$I_{yn}$  is the moment of inertia of cross section about the principal axis,  $y$ ,

$I_{\omega}$  is the cross-sectional sectorial moment of inertia. It is not really used in the program as the bi-moment,  $B$ , is assumed to be zero,



$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$c_x$  and  $c_y$  are the coefficients for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1),

$\beta$  is the coefficient that takes the effect of average shear stress on the moment capacity for Class 2 and Class 3 sections,

$\phi_b$  is the stability factor in bending to be calculated based on SP16 Annex G.

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

The coefficients  $c_x$  and  $c_y$  for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections for doubly-symmetric I-Shape, singly-symmetric I-Shape, Channel, double-Channel, Box, Pipe, Tee, Angle, Double-Angle, and Solid Rectangular shapes are calculated based on the SP16 Table F.1 given in Annex F. The values of  $c_x$  and  $c_y$  are interpolated for  $A_f/A_w$  when needed. For all other shapes including Solid Circular sections and SD-Sections,  $c_x$  and  $c_y$  are calculated assuming the shape is similar to the solid rectangular section as for the item 5 in the table.

The coefficient,  $\beta$ , that takes the effect of average shear stress on the moment capacity for Class 2 and Class 3 sections is calculated as follows:

$$\beta = \begin{cases} 1, & \text{if } 0 \leq \frac{\tau_x}{R_s} \leq 0.5, \\ 1 - \left[ \frac{0.2}{\alpha_f + 0.25} \right] \left( \frac{\tau_x}{R_s} \right)^4, & \text{if } 0.5 \leq \frac{\tau_x}{R_s} \leq 0.9, \end{cases}, \quad (\text{SP16 8.5.18})$$

where,

$$\tau_x = \frac{Q_x}{A_w}, \quad (\text{SP16 8.2.3})$$

$$\alpha_f = \frac{A_f}{A_w}, \quad (\text{SP16 8.2.3})$$

$R_s$  is the design shear strength (SP16 6.1, Table 2),

$A_f$  is the area of a single flange. For singly-symmetric I-shape it is the area of the smaller flange (SP16 8.2.3), and

$A_w$  is the area of area of the webs. For box sections it is the area of the two webs together (SP16 8.2.3).

The stability factor in bending,  $\phi_b$ , is calculated based on SP16 Annex G. The coefficient  $\phi_b$  is determined depending on the arrangement of supports, unbraced compressed zone, and type of load applications. It is assumed that the load is applied in the plane of minor axis strength---in the plane of maximum strength, and that the supporting section fixed against lateral displacement and rotation. For members with doubly-symmetric I-sections the coefficient  $\phi_b$  is calculated as follows. For all other cases see the section SP16 Annex G.

$$\phi_b = \begin{cases} \phi_1, & \text{if } \phi_1 \leq 0.85, \\ 0.68 + 0.21\phi_1, & \text{if } \phi_1 > 0.85, \end{cases} \quad (\text{SP16 8.4.1, G.1})$$

where,

$$\phi_1 = \psi \frac{I_y}{I_x} \left( \frac{h}{l_{ef}} \right)^2 \frac{E}{R_y}, \quad (\text{SP16 G.1})$$

$h$  is full height of the rolled cross-section (SP16 G.1),

$l_{ef}$  is the estimated length of the member determined in accordance with the requirements of SP16 8.4.2. It is the distance between two bracing points in the compression zone (SP16 8.2.3, G.1),

$\psi$  is the coefficient determined using the section SP16 G.3 (SP16 G.1). The value of coefficient  $\psi$  in the above formula is calculated by formulas of SP16 Tables G.1 and G.2 depending on the number of bracing points of the compressed zone, type of load space, and its applications, as well as on the coefficient  $\alpha$ , which is given below,

$$\alpha = 1.54 \frac{I_t}{I_y} \left( \frac{l_{ef}}{h} \right)^2, \quad (\text{SP16 G.4})$$

### 3.8.2.3 Members Subjected to Flexure and Tension

If the axial load is tensile, the section characteristic yield strength is less than 440 N/mm<sup>2</sup> ( $R_{yn} \leq 440$  N/mm<sup>2</sup>), the average shear stress for major and minor principal directions are small ( $\tau_x/R_s \leq 0.5$  and  $\tau_y/R_s \leq 0.5$ ), and significant axial force ( $N/A_n > 0.1$ ), the D/C ratio is given by:

$$\left( \frac{N}{A_n R_y \gamma_c} \right)^n + \frac{M_x}{c_x W_{x,\min} R_y \gamma_c} + \frac{M_y}{c_y W_{y,\min} R_y \gamma_c} + \frac{B}{W_{on,\min} R_y \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.1})$$

If the axial load is tensile but the above conditions are not met, the D/C ratio is given by:

$$\frac{N}{A_n R_y \gamma_c} \pm \frac{M_x y}{I_{xn} R_y \gamma_c} \pm \frac{M_y x}{I_{yn} R_y \gamma_c} \pm \frac{B \omega}{I_{on} R_y \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.1})$$

If the axial load is tensile, the section characteristic yield strength is more than 440 N/mm<sup>2</sup> ( $R_{yn} > 440$  N/mm<sup>2</sup>), the D/C ratio considering the tension flange yielding for singly symmetric sections including singly-symmetric I-shape, Channel, Tee, and Double Angle sections is given by the following:

$$\frac{N}{A_n (R_u/\gamma_u) \gamma_c} - \frac{M_x}{\delta W_{tx} (R_u/\gamma_u) \gamma_c} \leq 1.0, \quad (\text{SP16 9.1.3})$$

where,

$M_x$  is the factored moment about the major principal axis,  $x$ ,

$M_y$  is the factored moment about the minor principal axis,  $y$ ,

$B$  is the bi-moment. It is assumed to be zero throughout the program,

$W_{cx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the compression flange,

$W_{cy}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the compression side,

$W_{tx}$  is the section modulus of cross section about the principal axis,  $x$ , computed for the tension flange,

$W_{ty}$  is the section modulus of cross section about the principal axis,  $y$ , computed for the tension side,

$W_{xn,min}$  is the minimum section modulus of cross section about the principal axis,  $x$ ,

$W_{yn,min}$  is the minimum section modulus of cross section about the principal axis,  $y$ ,

$I_{xn}$  is the moment of inertia of cross section about the principal axis,  $x$ ,

$I_{yn}$  is the moment of inertia of cross section about the principal axis,  $y$ ,

$I_{\omega}$  is the cross-sectional sectorial moment of inertia. It is not really used in the program as the bi-moment,  $B$ , is assumed to be zero,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_u$  is the design fracture strength for tension, compression, and flexure (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1),

$\gamma_u$  is the safety factor for structure members designed in terms of their behavior with the use of design strengths  $R_u$  (SP16 4.3.2),

$c_x$  and  $c_y$  are the coefficients for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1) and it is described in more details in the previous section,

$n$  is the coefficient that is used as a power for the calculation of structural elements taking into account the development of plastic deformations for Class 2 and Class 3 sections (SP16 Annex F, Table F.1),

$\delta$  is the amplification coefficient that is used to amplify the moment,

$$\delta = 1 - 0.1 \bar{\lambda}^{-2} \left( \frac{N}{AR_y} \right), \quad (\text{SP16 9.1.3})$$

$$\bar{\lambda} = \lambda \sqrt{\frac{R_y}{E}}, \quad (\text{SP16 7.1.3})$$

$$\lambda = \frac{Kl_{ef}}{r}, \quad (\text{SP16 7.1.3})$$

### 3.8.3 Shear Stresses

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

$$\frac{Q_x}{A_w R_s \gamma_c} \leq 1.0, \quad (\text{SP16 8.2.3})$$

$$\frac{Q_y}{A_{fl} R_s \gamma_c} \leq 1.0, \quad (\text{SP16 8.2.3})$$

where,

$Q_x$  is the factored shear force causing bending of the about the major principal axis,  $x$ ,

$Q_y$  is the factored shear force causing bending of the about the minor principal axis,  $y$ ,

$A_w$  is the total area of all the webs,

$A_{fl}$  is the total area of all the flanges,

$R_y$  is the design yield strength for tension, compression, and flexure (SP16 6.1, Table 2),

$R_s$  is the design strength of steel for shearing (SP16 6.1, Table 2),

$\gamma_c$  is the service factor for structural members and connections (SP16 4.3.2, Table 1).

The total area of all the webs,  $A_w$ , and of all the flanges,  $A_{fl}$ , are taken as follows:

$$A_w = \begin{cases} dt_w & \text{for I-Shape sections,} \\ dt_w & \text{for Channel sections,} \\ dt_w & \text{for T-Shape sections,} \\ 2dt_w & \text{for Double Channel sections,} \\ 2dt_w & \text{for Box sections,} \\ dt_w & \text{for Angle sections,} \\ 2dt_w & \text{for Double Angle sections,} \\ (2/3)A & \text{for Rectangular solid sections,} \\ (3/4)A & \text{for Circular solid sections,} \\ (1/2)A & \text{for Pipe sections.} \end{cases} \quad (\text{SP16 8.2.3})$$

$$A_{fl} = \begin{cases} 2b_f t_f & \text{for I-Shape sections,} \\ 2b_f t_f & \text{for Channel sections,} \\ b_f t_f & \text{for T-Shape sections,} \\ 4b_f t_f & \text{for Double Channel sections,} \\ 2b_f t_f & \text{for Box sections,} \\ b_f t_f & \text{for Angle sections,} \\ 2b_f t_f & \text{for Double Angle sections,} \\ (2/3)A & \text{for Rectangular solid sections,} \\ (3/4)A & \text{for Circular solid sections,} \\ (1/2)A & \text{for Pipe sections.} \end{cases} \quad (\text{SP16 8.2.3})$$

For all other shapes  $A_w$  and  $A_{fl}$  are based on elastic equivalent shear area.

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 D/C ratios for shear, it is assumed that there is no internal stiffeners used to enhance shear strength of a section (SP16 8.2.3).

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