ETABS®

Integrated
Three Dimensional
Static and Dynamic Analysis and Design
of
Building Systems

STEEL FRAME DESIGN MANUAL

Computers and Structures, Inc.
Berkeley, California, USA

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

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

Overview

ETABS features powerful and completely integrated modules for design of both steel and reinforced concrete structures. The program provides the user with options to create, modify, analyze and design structural models, all from within the same user interface. The program is capable of performing initial member sizing and optimization from within the same interface.

The program provides an interactive environment in which the user can study the stress conditions, make appropriate changes, such as revising member properties, and re-examine the results without the need to re-run the analysis. A single mouse click on an element brings up detailed design information. Members can be grouped together for design purposes. The output in both graphical and tabulated formats can be readily printed.

The program is structured to support a wide variety of the latest national and international building design codes for the automated design and check of concrete and steel frame members. The program currently supports the following steel design codes:

- U.S. AISC/ASD (1989),
- U.S. AISC/LRFD (1993),
The design is based upon a set of user-specified loading combinations. However, the program provides a set of default load combinations for each design code supported in ETABS. If the default load combinations are acceptable, no definition of additional load combination is required.

In the design optimization process the program picks the least weight section required for strength for each element to be designed, from a set of user specified sections. Different sets of available sections can be specified for different groups of elements. Also several elements can be grouped to be designed to have the same section.

In the check process the program produces demand/capacity ratios for axial load and biaxial moment interactions and shear. The demand/capacity ratios are based on element stress and allowable stress for allowable stress design, and on factored loads (actions) and factored capacities (resistances) for limit state design.

The checks are made for each user specified (or program defaulted) load combination and at several user controlled stations along the length of the element. Maximum demand/capacity ratios are then reported and/or used for design optimization.

All allowable stress values or design capacity values for axial, bending and shear actions are calculated by the program. Tedious calculations associated with evaluating effective length factors for columns in moment frame type structures are automated in the algorithms.

When using 1997 UBC-ASD or UBC-LRFD design codes, requirements for continuity plates at the beam to column connections are evaluated. The program performs a joint shear analysis to determine if doubler plates are required in any of the joint panel zones. Maximum beam shears required for the beam shear connection design are reported. Also maximum axial tension or compression values that are generated in the member are reported.

Special 1997 UBC-ASD and UBC-LRFD seismic design provisions are implemented in the current version of the program. The ratio of the beam flexural capacities with respect to the column reduced flexural capacities (reduced for axial force effect) associated with the weak beam-strong column aspect of any beam/column
intersection, are reported for special moment resisting frames. Capacity require-
ments associated with seismic framing systems that require ductility are checked.

The presentation of the output is clear and concise. The information is in a form that
allows the designer to take appropriate remedial measures if there is member over-
stress. Backup design information produced by the program is also provided for
convenient verification of the results.

English as well as SI and MKS metric units can be used to define the model geometry
and to specify design parameters.

Organization

This manual is organized in the following way:

Chapter II outlines various aspects of the steel design procedures of the ETABS
program. This chapter describes the common terminology of steel design as imple-
mented in ETABS.

Each of seven subsequent chapters gives a detailed description of a specific code of
practice as interpreted by and implemented in ETABS. Each chapter describes the
design loading combinations to be considered; allowable stress or capacity calcula-
tions for tension, compression, bending, and shear; calculations of demand/capac-
ity ratios; and other special considerations required by the code. In addition, Chap-
ter V and VI describe the determination of continuity plate area, doubler plate
thickness, beam connection shear, and brace connection force according to the
UBC ASD and LRFD codes, respectively.

- Chapter III gives a detailed description of the AISC-ASD code (AISC 1989) as
  implemented in ETABS.
- Chapter IV gives a detailed description of the AISC-LRFD code (AISC 1993)
  as implemented in ETABS.
- Chapter V gives a detailed description of the UBC-ASD steel building code
  (UBC 1997) as implemented in ETABS.
- Chapter VI gives a detailed description of the UBC-LRFD steel building code
  (UBC 1997) as implemented in ETABS.
- Chapter VII gives a detailed description of the Canadian code (CISC 1994) as
  implemented in ETABS.
- Chapter VIII gives a detailed description of the British code BS 5950 (BSI
  1990) as implemented in ETABS.
Chapter IX gives a detailed description of the Eurocode 3 (CEN 1992) as implemented in ETABS. Chapter X outlines various aspects of the tabular and graphical output from ETABS related to steel design.

Recommended Reading

It is recommended that the user read Chapter II “Design Algorithms” and one of seven subsequent chapters corresponding to the code of interest to the user. Finally the user should read “Design Output” in Chapter X for understanding and interpreting ETABS output related to steel design. If the user’s interest is in the UBC-ASD steel design code, it is recommended that the user should also read the chapter related to AISC-ASD. Similarly, if the user’s interest is in the UBC-LRFD steel design code, it is recommended that the user should also read the chapter related to AISC-LRFD.

A steel design tutorial is presented in the ETABS Quick Tutorial manual. It is recommended that first time users follow through the steps of this tutorial before reading this manual.
Chapter II

Design Algorithms

This chapter outlines various aspects of the steel check and design procedures that are used by the ETABS program. The steel design and check may be performed according to one of the following codes of practice.

Details of the algorithms associated with each of these codes as implemented and interpreted in ETABS are described in subsequent chapters. However, this chapter provides a background which is common to all the design codes. For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- References to the AISC-ASD89 code carry the prefix of “ASD”
- References to the AISC-LRFD93 code carry the prefix of “LRFD”
- References to the UBC-ASD97 code carry the prefix of “UBC”
- References to the UBC-LRFD97 code carry the prefix of “UBC”
- References to the Canadian code carry the prefix of “CISC”
- References to the British code carry the prefix of “BS”
- References to the Eurocode carry the prefix of “EC3”

It is assumed that the user has an engineering background in the general area of structural steel design and familiarity with at least one of the above mentioned design codes.

Design Load Combinations

The design load combinations are used for determining the various combinations of the load cases for which the structure needs to be designed/checked. The load combination factors to be used vary with the selected design code. The load combination factors are applied to the forces and moments obtained from the associated load cases and the results are then summed to obtain the factored design forces and moments for the load combination.

For multi-valued load combinations involving response spectrum, time history, moving loads and multi-valued combinations (of type enveloping, square-root of the sum of the squares or absolute) where any correspondence between interacting quantities is lost, the program automatically produces multiple sub combinations using maxima/minima permutations of interacting quantities. Separate combinations with negative factors for response spectrum cases are not required because the
program automatically takes the minima to be the negative of the maxima for response spectrum cases and the above described permutations generate the required sub combinations.

When a design combination involves only a single multi-valued case of time history or moving load, further options are available. The program has an option to request that time history combinations produce sub combinations for each time step of the time history. Also an option is available to request that moving load combinations produce sub combinations using maxima and minima of each design quantity but with corresponding values of interacting quantities.

For normal loading conditions involving static dead load, live load, wind load, and earthquake load, and/or dynamic response spectrum earthquake load, the program has built-in default loading combinations for each design code. These are based on the code recommendations and are documented for each code in the corresponding chapters.

For other loading conditions involving moving load, time history, pattern live loads, separate consideration of roof live load, snow load, etc., the user must define design loading combinations either in lieu of or in addition to the default design loading combinations.

The default load combinations assume all static load cases declared as dead load to be additive. Similarly, all cases declared as live load are assumed additive. However, each static load case declared as wind or earthquake, or response spectrum cases, is assumed to be non additive with each other and produces multiple lateral load combinations. Also wind and static earthquake cases produce separate loading combinations with the sense (positive or negative) reversed. If these conditions are not correct, the user must provide the appropriate design combinations.

The default load combinations are included in design if the user requests them to be included or if no other user defined combination is available for concrete design. If any default combination is included in design, then all default combinations will automatically be updated by the program any time the user changes to a different design code or if static or response spectrum load cases are modified.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

The user is cautioned that if moving load or time history results are not requested to be recovered in the analysis for some or all the frame members, then the effects of these loads will be assumed to be zero in any combination that includes them.
Design and Check Stations

For each load combination, each beam, column, or brace element is designed or checked at a number of locations along the length of the element. The locations are based on equally spaced segments along the clear length of the element. By default there will be at least 3 stations in a column or brace element and the stations in a beam will be at most 2 feet (0.5m if model is created in SI unit) apart. The number of segments in an element can be overwritten by the user before the analysis is made. The user can refine the design along the length of an element by requesting more segments. See the section “Frame Output Stations Assigned to Line Objects” in the *ETABS User’s Manual Volume 1* (CSI 1999) for details.

The axial-flexure interaction ratios as well as shear stress ratios are calculated for each station along the length of the member for each load combination. The actual member stress components and corresponding allowable stresses are calculated. Then, the stress ratios are evaluated according to the code. The controlling compression and/or tension stress ratio is then obtained, along with the corresponding identification of the station, load combination, and code-equation. A stress ratio greater than 1.0 indicates an overstress or exceeding a limit state.

When using 1997 UBC ASD or LRFD design codes, requirements for continuity plates at the beam to column connections are evaluated at the topmost station of each column. The program also performs a joint shear analysis at the same station to determine if doubler plates are required in any of the joint panel zones. Maximum beam shears required for the beam shear connection design at the two ends are reported. Also maximum axial tension or compression values that are generated at the two ends in the braces are reported. The ratio of the beam flexural capacities with respect to the column reduced flexural capacities (reduced for axial force effect) associated with the weak beam-strong column aspect of any beam/column intersection, are reported for special moment resisting frames.

**P-Δ Effects**

Except for AISC-ASD and UBC-ASD design codes, the ETABS design algorithms require that the analysis results include the P-Δ effects. The P-Δ effects are considered differently for “braced” or “nonway” and “unbraced” or “sway” components of moments in frames. For the braced moments in frames, the effect of P-Δ is limited to “individual member stability”. For unbraced components, “lateral drift effects” should be considered in addition to “individual member stability” effect. In ETABS, it is assumed that “braced” or “nonway” moments are contributed from...
the “dead” or “live” loads. Whereas, “unbraced” or “sway” moments are contributed from all other types of loads.

For the individual member stability effects, the moments are magnified with moment magnification factors as in the AISC-LRFD and UBC-LRFD codes or are considered directly in the design equations as in the Canadian, British, and European codes. No moment magnification is applied to the AISC-ASD and UBC-ASD codes.

For lateral drift effects of unbraced or sway frames, ETABS assumes that the amplification is already included in the results because P-Δ effects are considered for all but AISC-ASD and UBC-ASD codes.

The users of ETABS should be aware that the default analysis option in ETABS for P-Δ effect is turned OFF. The default number of iterations for P-Δ analysis is 1. **The user should turn the P-Δ analysis ON and set the maximum number of iterations for the analysis.** No P-Δ analysis is required for the AISC-ASD and UBC-ASD codes. For further reference, the user is referred to *ETABS User’s Manual Volume 2* (CSI 1999). The user is also cautioned that ETABS currently considers P-Δ effects due to axial loads in frame members only. Forces in other types of elements do not contribute to this effect. If significant forces are present in other types of elements, for example, large axial loads in shear walls modeled as shell elements, then the additional forces computed for P-Δ will be inaccurate.

**Element Unsupported Lengths**

To account for column slenderness effects, the column unsupported lengths are required. The two unsupported lengths are \( l_{33} \) and \( l_{22} \). See Figure II-1. These are the lengths between support points of the element in the corresponding directions. The length \( l_{33} \) corresponds to instability about the 3-3 axis (major axis), and \( l_{22} \) corresponds to instability about the 2-2 axis (minor axis). The length \( l_{22} \) is also used for lateral-torsional buckling caused by major direction bending (i.e., about the 3-3 axis). See Figure II-2 for correspondence between the ETABS axes and the axes in the design codes.

Normally, the unsupported element length is equal to the length of the element, i.e., the distance between END-I and END-J of the element. See Figure II-1. The program, however, allows users to assign several elements to be treated as a single member for design. This can be done differently for major and minor bending. Therefore, extraneous joints, as shown in Figure II-3, that affect the unsupported length of an element are automatically taken into consideration.
Figure II-1
*Major and Minor Axes of Bending*

Figure II-2
*Correspondence between ETABS Axes and Code Axes*
In determining the values for \( l_{22} \) and \( l_{33} \) of the elements, the program recognizes various aspects of the structure that have an effect on these lengths, such as member connectivity, diaphragm constraints and support points. The program automatically locates the element support points and evaluates the corresponding unsupported element length.

Therefore, the unsupported length of a column may actually be evaluated as being greater than the corresponding element length. If the beam frames into only one direction of the column, the beam is assumed to give lateral support only in that direction. The user has options to specify the unsupported lengths of the elements on an element-by-element basis.

![Figure II-3](image)

**Figure II-3**

*Unsupported Lengths are Affected by Intermediate Nodal Points*

**Effective Length Factor (\( K \))**

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

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

\[
S_{cx} = \sum \left( \frac{E_c I_c}{L_c} \right)_x \quad S_{bx} = \sum \left( \frac{E_b I_b}{L_b} \right)_x \\
S_{cy} = \sum \left( \frac{E_c I_c}{L_c} \right)_y \quad S_{by} = \sum \left( \frac{E_b I_b}{L_b} \right)_y
\]

where the $x$ and $y$ subscripts correspond to the global $X$ and $Y$ directions and the $c$ and $b$ subscripts refer to column and beam. The local 2-2 and 3-3 terms $EI_{22}/I_{22}$ and $EI_{33}/I_{33}$ 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:

\[
G^I_{22} = \frac{S^I_{c22}}{S^I_{b22}} \quad G^J_{22} = \frac{S^J_{c22}}{S^J_{b22}} \\
G^I_{33} = \frac{S^I_{c33}}{S^I_{b33}} \quad G^J_{33} = \frac{S^J_{c33}}{S^J_{b33}}
\]

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

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

from which $K = \pi / \alpha$. This relationship is the mathematical formulation for the evaluation of $K$ factors for moment-resisting frames assuming sidesway to be uninhibited. For other structures, such as braced frame structures, the $K$-factors for all members are usually unity and should be set so by the user. The following are some important aspects associated with the column $K$-factor algorithm:
An element that has a pin at the joint under consideration will not enter the stiffness summations calculated above. An element that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated $EI$ value. Also, beam elements that have no column member at the far end from the joint under consideration, such as cantilevers, will not enter the stiffness summation.

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

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

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

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

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

Design of Continuity Plates

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

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

To achieve a beam/column moment connection, continuity plates such as shown in Figure II-4 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.
Figure II-4

Plan Showing Continuity Plates for a Column of I-Section
For connection conditions described in the last two items above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam.

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

When using 1997 UBC ASD or LRFD design codes, the program investigates whether the continuity plates are required. Columns of I-sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e. parallel to the column major direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.

**Design of Doubler Plates**

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

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

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending upon the loading and the geometry of the steel beams that frame into the column, either along the column major direction, or at an angle so that the beams have components along the column major direction. See Figure II-5. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames only. No check is made for braced frames. Doubler plate requirements are evaluated when using UBC ASD and LRFD codes.
Figure II-5
*Elevation and Plan of Doubler Plates for a Column of I-Section*
Choice of Input Units

English as well as SI and MKS metric units can be used for input. But the codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. For example, AISC-ASD code is published in kip-inch-second units. By default, all equations and descriptions presented in the chapter “Check/Design for AISC-ASD89” correspond to kip-inch-second units. However, any system of units can be used to define and design the structure in ETABS.
This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the AISC-ASD89 design code (AISC 1989a). Various notations used in this chapter are described in Table III-1.

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

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this chapter. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates overstress. Similarly, a shear capacity ratio is also calculated separately.
\[ \begin{align*}
A &= \text{Cross-sectional area, in}^2 \\
A_e &= \text{Effective cross-sectional area for slender sections, in}^2 \\
A_f &= \text{Area of flange, in}^2 \\
A_g &= \text{Gross cross-sectional area, in}^2 \\
A_{s1}, A_{s2} &= \text{Major and minor shear areas, in}^2 \\
A_w &= \text{Web shear area, } d t_w, \text{ in}^2 \\
C_b &= \text{Bending Coefficient} \\
C_m &= \text{Moment Coefficient} \\
C_n &= \text{Warping constant, in}^6 \\
D &= \text{Outside diameter of pipes, in} \\
E &= \text{Modulus of elasticity, ksi} \\
F_a &= \text{Allowable axial stress, ksi} \\
F_b &= \text{Allowable bending stress, ksi} \\
F_{b1}, F_{b2} &= \text{Allowable major and minor bending stresses, ksi} \\
F_{cr} &= \text{Critical compressive stress, ksi} \\
F_{v133} &= \frac{12 \pi^2 E}{23(K_{s1}l_{s3}/r_{s3})^2} \\
F_{v22} &= \frac{12 \pi^2 E}{23(K_{s2}l_{s2}/r_{s2})^2} \\
F_v &= \text{Allowable shear stress, ksi} \\
F_y &= \text{Yield stress of material, ksi} \\
K &= \text{Effective length factor} \\
K_{s1}, K_{s2} &= \text{Effective length } K\text{-factors in the major and minor directions} \\
M_{s1}, M_{s2} &= \text{Major and minor bending moments in member, kip-in} \\
M_{ob} &= \text{Lateral-torsional moment for angle sections, kip-in} \\
P &= \text{Axial force in member, kips} \\
P_e &= \text{Euler buckling load, kips} \\
Q &= \text{Reduction factor for slender section, } = Q_s Q_a \\
Q_s &= \text{Reduction factor for stiffened slender elements} \\
Q_a &= \text{Reduction factor for unstiffened slender elements} \\
S &= \text{Section modulus, in}^3 \\
S_{s1}, S_{s2} &= \text{Major and minor section moduli, in}^3
\end{align*} \]

**Table III-1**

*AISC-ASD Notations*
Chapter III  Check/Design for AISC-ASD89

\[ S_{\text{eff,33}}, S_{\text{eff,22}} = \text{Effective major and minor section moduli for slender sections, in}^3 \]
\[ S_c = \text{Section modulus for compression in an angle section, in}^3 \]
\[ V_2, V_3 = \text{Shear forces in major and minor directions, kips} \]
\[ b = \text{Nominal dimension of plate in a section, in longer leg of angle sections, } \]
\[ b_f - 2t_w \text{ for welded and } b_f - 3t_w \text{ for rolled box sections, etc.} \]
\[ b_e = \text{Effective width of flange, in} \]
\[ b_f = \text{Flange width, in} \]
\[ d = \text{Overall depth of member, in} \]
\[ f_a = \text{Axial stress either in compression or in tension, ksi} \]
\[ f_b = \text{Normal stress in bending, ksi} \]
\[ f_{b,33}, f_{b,22} = \text{Normal stress in major and minor direction bending, ksi} \]
\[ f_v = \text{Shear stress, ksi} \]
\[ f_{v,23} = \text{Shear stress in major and minor direction bending, ksi} \]
\[ h = \text{Clear distance between flanges for I shaped sections (} d - 2t_f \text{), in} \]
\[ h_e = \text{Effective distance between flanges less fillets, in} \]
\[ k = \text{Distance from outer face of flange to web toe of fillet, in} \]
\[ k_c = \text{Parameter used for classification of sections,} \]
\[ \left\{ \begin{array}{ll}
4.05 & \text{if } h/t_w > 70, \\
\frac{h}{t_w} & \text{if } h/t_w \leq 70.
\end{array} \right. \]
\[ l_{33}, l_{22} = \text{Major and minor direction unbraced member lengths, in} \]
\[ l_e = \text{Critical length, in} \]
\[ r = \text{Radius of gyration, in} \]
\[ r_{33}, r_{22} = \text{Radii of gyration in the major and minor directions, in} \]
\[ r_e = \text{Minimum Radius of gyration for angles, in} \]
\[ t = \text{Thickness of a plate in I, box, channel, angle, and T sections, in} \]
\[ t_f = \text{Flange thickness, in} \]
\[ t_w = \text{Web thickness, in} \]
\[ \beta_w = \text{Special section property for angles, in} \]

Table III-1
AISC-ASD Notations (cont.)
English as well as SI and MKS metric units can be used for input. But the code is based on Kip-Inch-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Kip-Inch-Second** units unless otherwise noted.

### Design Loading Combinations

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

- \( DL \) (ASD A4.1)
- \( DL + LL \) (ASD A4.1)
- \( DL \pm WL \) (ASD A4.1)
- \( DL + LL \pm WL \) (ASD A4.1)
- \( DL \pm EL \) (ASD A4.1)
- \( DL + LL \pm EL \) (ASD A4.1)

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

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

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

### Classification of Sections

The allowable stresses for axial compression and flexure are dependent upon the classification of sections as either Compact, Noncompact, Slender, or Too Slender. ETABS classifies the individual members according to the limiting width/thickness ratios given in Table III-2 (ASD B5.1, F3.1, F5, G1, A-B5-2). The definition
AISC-ASD89: Axes Conventions

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

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

Figure III-1
AISC-ASD Definition of Geometric Properties
<table>
<thead>
<tr>
<th>Section Description</th>
<th>Ratio Checked</th>
<th>Compact Section</th>
<th>Noncompact Section</th>
<th>Slender Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>$b_i / 2t_i$ (rolled)</td>
<td>$\leq 65 / \sqrt{F_y}$</td>
<td>$\leq 95 / \sqrt{F_y}$</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$b_i / 2t_i$ (welded)</td>
<td>$\leq 65 / \sqrt{F_y}$</td>
<td>$\leq 95 / \sqrt{F_y} / k_c$</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$d / t_u$</td>
<td></td>
<td>No limit</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$h / t_u$</td>
<td>No limit</td>
<td>No limit</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td></td>
<td>If compression only, $\leq 253 / \sqrt{F_y}$, otherwise $\leq 760 / \sqrt{F_y}$</td>
<td>$\leq \frac{14000}{\sqrt{F_y} (F_y + 16.5)}$</td>
<td>$\leq 260$</td>
</tr>
<tr>
<td>BOX</td>
<td>$b / t_i$</td>
<td>$\leq 190 / \sqrt{F_y}$</td>
<td>$\leq 238 / \sqrt{F_y}$</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$d / t_u$</td>
<td>As for I-shapes</td>
<td>No limit</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$h / t_u$</td>
<td>No limit</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>$t_c \geq t_i / 2, d_u \leq 6b_i$</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$b / t_i$</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$d / t_u$</td>
<td>As for I-shapes</td>
<td>No limit</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>$h / t_u$</td>
<td>No limit</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>No limit</td>
<td>No limit</td>
<td>If welded $b_i / d_u \leq 0.25$, $t_i / t_u \leq 3.0$, if rolled $b_i / d_u \leq 0.5$, $t_i / t_u \leq 2.0$</td>
</tr>
</tbody>
</table>

**Table III-2**

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

24 Classification of Sections
of the section properties required in this table is given in Figure III-1 and Table III-1.

If the section dimensions satisfy the limits shown in the table, the section is classified as either Compact, Noncompact, or Slender. If the section satisfies the criteria for Compact sections, then the section is classified as Compact section. If the section does not satisfy the criteria for Compact sections but satisfies the criteria for

<table>
<thead>
<tr>
<th>Section Description</th>
<th>Ratio Checked</th>
<th>Compact Section</th>
<th>Noncompact Section</th>
<th>Slender Section</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>T-SHAPE</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( b_t / 2t_f )</td>
<td>( \leq 65 / \sqrt{F_y} )</td>
<td>( \leq 95 / \sqrt{F_y} )</td>
<td>No limit</td>
</tr>
<tr>
<td></td>
<td>( d / t_n )</td>
<td>Not applicable</td>
<td>( \leq 127 / \sqrt{F_y} )</td>
<td>No limit</td>
</tr>
</tbody>
</table>
|                     | Other         | No limit        | No limit           | If welded \( b_t / d_n \geq 0.5 \), \( t_l / t_n \geq 1.25 \)  
|                     |               |                 |                    | If rolled \( b_t / d_n \geq 0.5 \), \( t_l / t_n \geq 1.10 \) |
| **DOUBLE ANGLES**   | \( b / t \)   | Not applicable | \( \leq 76 / \sqrt{F_y} \) | No limit |
| **ANGLE**           | \( b / t \)   | Not applicable | \( \leq 76 / \sqrt{F_y} \) | No limit |
| **PIPE**            | \( D / t \)   | \( \leq 3,300 / F_y \) | \( \leq 3,300 / F_y \) | \( \leq 13,000 / F_y \)  
|                     |               |                 |                    | (Compression only) No limit for flexure |
| **ROUND BAR**       | —             |                 | Assumed Compact    |                 |
| **RECTANGLE**       | —             |                 | Assumed Noncompact |                 |
| **GENERAL**         | —             |                 | Assumed Noncompact |                 |

Table III-2  
Limiting Width-Thickness Ratios for Classification of Sections Based on AISC-ASD (Cont.)
Noncompact sections, the section is classified as Noncompact section. If the section does not satisfy the criteria for Compact and Noncompact sections but satisfies the criteria for Slender sections, the section is classified as Slender section. If the limits for Slender sections are not met, the section is classified as Too Slender. **Stress check of Too Slender sections is beyond the scope of ETABS.**

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

**Calculation of Stresses**

The stresses are calculated at each of the previously defined stations. The member stresses for non-slender sections that are calculated for each load combination are, in general, based on the gross cross-sectional properties:

\[
f_a = \frac{P}{A}
\]
\[
f_{b,33} = \frac{M_{33}}{S_{33}}
\]
\[
f_{b,22} = \frac{M_{22}}{S_{22}}
\]
\[
f_{v,2} = \frac{V_{2}}{A_{v,2}}
\]
\[
f_{v,3} = \frac{V_{3}}{A_{v,3}}
\]

If the section is slender with slender stiffened elements, like slender web in I, Channel, and Box sections or slender flanges in Box, effective section moduli based on reduced web and reduced flange dimensions are used in calculating stresses.

\[
f_a = \frac{P}{A} \quad \text{(ASD A-B5.2d)}
\]
\[
f_{b,33} = \frac{M_{33}}{S_{\text{eff},33}} \quad \text{(ASD A-B5.2d)}
\]
\[
f_{b,22} = \frac{M_{22}}{S_{\text{eff},22}} \quad \text{(ASD A-B5.2d)}
\]
\[
f_{v,2} = \frac{V_{2}}{A_{v,2}} \quad \text{(ASD A-B5.2d)}
\]
\[
f_{v,3} = \frac{V_{3}}{A_{v,3}} \quad \text{(ASD A-B5.2d)}
\]

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

For Single-angle sections, the shear stresses are calculated for directions along the geometric axes. For all other sections the shear stresses are calculated along the geometric and principle axes.
Calculation of Allowable Stresses

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

*If the user specifies nonzero allowable stresses for one or more elements in the ETABS “Allowable Stress Overwrites” form, these values will override the above mentioned calculated values for those elements. The specified allowable stresses should be based on the principal axes of bending.*

**Allowable Stress in Tension**

The allowable axial tensile stress value \( F_a \) is assumed to be \( 0.6 F_y \).

\[
F_a = 0.6 F_y \quad \text{(ASD D1, ASD SAM 2)}
\]

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

**Allowable Stress in Compression**

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

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

**Flexural Buckling**

The allowable axial compressive stress value, \( F_a \), depends on the slenderness ratio \( Kl/r \) based on gross section properties and a corresponding critical value, \( C_c \), where
\[
\frac{KL}{r} = \max \left\{ \frac{K_{33} l_{33}}{r_{33}}, \frac{K_{22} l_{22}}{r_{22}} \right\}, \quad \text{and}
\]

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

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

For Compact or Noncompact sections \( F_a \) is evaluated as follows:

\[
F_a = \begin{cases} 
\left\{ 1.0 - \frac{(KL/r)^2}{2C_c^2} \right\} F_y, & \text{if } \frac{KL}{r} \leq C_c, \quad \text{(ASD E2-1, SAM 4-1)} \\
\frac{5}{3} + \frac{3(KL/r)}{8 C_c} - \frac{(KL/r)^3}{8 C_c^3}, & \text{if } \frac{KL}{r} > C_c. \quad \text{(ASD E2-2, SAM 4-2)}
\end{cases}
\]

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

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

\[
F_a = \begin{cases} 
Q \left\{ 1.0 - \frac{(KL/r)^2}{2C_c^2} \right\} F_y, & \text{if } \frac{KL}{r} \leq C'_c, \quad \text{(ASD A-B5-11, SAM 4-1)} \\
\frac{12 \pi^2 E}{23(KL/r)^2}, & \text{if } \frac{KL}{r} > C'_c. \quad \text{(ASD A-B5-12, SAM 4-2)}
\end{cases}
\]

where,

\[
C'_c = \sqrt{\frac{2\pi^2 E}{Q F_y}}. \quad \text{(ASD A-B5.2c, ASD SAM 4)}
\]
For slender sections, if $Kl/r$ is greater than 200, then the calculated value of $F_a$ is taken not to exceed its value calculated by using the equation ASD A-B5-12 (ASD B7, E1).

For slender Pipe sections $F_a$ is evaluated as follows:

$$F_a = \frac{662}{D/t} + 0.40 F_y$$  \hspace{1cm} \text{(ASD A-B5-9)}

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

$$Q = Q_s Q_a$$, where \hspace{1cm} \text{(ASD A-B5.2.c, SAM 4)}

$Q_s$ = reduction factor for unstiffened slender elements, and \hspace{1cm} \text{(ASD A-B5.2.a)}

$Q_a$ = reduction factor for stiffened slender elements. \hspace{1cm} \text{(ASD A-B5.2.c)}

The $Q_s$ factors for slender sections are calculated as described in Table III-3 (ASD A-B5.2a, ASD SAM 4). The $Q_a$ factors for slender sections are calculated as the ratio of effective cross-sectional area and the gross cross-sectional area.

$$Q_a = \frac{A_e}{A_g}$$ \hspace{1cm} \text{(ASD A-B5-10)}

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

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

$b_e$ for unstiffened elements is taken equal to $b$, and $b_e$ for stiffened elements is taken equal to or less than $b$ as given in Table III-4 (ASD A-B5.2b). For webs in I, box, and Channel sections, $h_e$ is used as $b_e$ and $h$ is used as $b$ in the above equation.

**Flexural-Torsional Buckling**

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

$$F_a = Q \left\{ \frac{1.0 - \left(\frac{Kl/r}{c_e} \right)^2}{2 C_{c_e}^2} \right\} F_y$$ \hspace{1cm} \text{if} \left(\frac{Kl/r}{c_e} \right) \leq C_{c_e}' \hspace{1cm} \text{(E2-1, A-B5-11)}$$
### Table III-3

Reduction Factor for Unstiffened Slender Elements, $Q_s$

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Reduction Factor for Unstiffened Slender Elements ($Q_s$)</th>
<th>Equation Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>$Q_s = \begin{cases} 1 &amp; \text{if } b_i/2t_i \leq 95/\left(\sqrt{F_y/k_c}\right), \ 1.0 \times 2.693 - 0.00309 \left[b_i/2t_i\right] \sqrt{F_y/k_c} + 26.200 k_i \left[b_i/2t_i\right]^{2} F_{y} &amp; \text{if } 95/\left(\sqrt{F_y/k_c}\right) &lt; b_i/2t_i &lt; 195/\left(\sqrt{F_y/k_c}\right), \ 26.200 k_i \left[b_i/2t_i\right]^{2} F_{y} &amp; \text{if } b_i/2t_i \geq 195/\left(\sqrt{F_y/k_c}\right). \end{cases}$</td>
<td>ASD A-B5-3, ASD A-B5-4</td>
</tr>
<tr>
<td>BOX</td>
<td>$Q_s = 1$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>As for I-shapes with $b_i/2t_i$ replaced by $b_i/t_i$.</td>
<td>ASD A-B5-3, ASD A-B5-4</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>For flanges, as for flanges in I-shapes. For web see below.</td>
<td>ASD A-B5-3, ASD A-B5-4, ASD A-B5-5, ASD A-B5-6</td>
</tr>
<tr>
<td>DOUBLE-ANGLE</td>
<td>$Q_s = \begin{cases} 1.0, &amp; \text{if } d/t_i \leq 127/\left(\sqrt{F_y}\right), \ 1.340 - 0.00447 [b/t_i] \sqrt{F_y}, &amp; \text{if } 127/\left(\sqrt{F_y}\right) &lt; d/t_i &lt; 176/\left(\sqrt{F_y}\right), \ 15,500 / \left[b/t_i\right]^2 \sqrt{F_y}, &amp; \text{if } d/t_i \geq 176/\left(\sqrt{F_y}\right). \end{cases}$</td>
<td>ASD A-B5-1, ASD A-B5-2, SAM 4-3</td>
</tr>
<tr>
<td>ANGLE</td>
<td>$Q_s = \begin{cases} 1.0, &amp; \text{if } b/t_i \leq 76/\left(\sqrt{F_y}\right), \ 1.340 - 0.00447 [b/t_i] \sqrt{F_y}, &amp; \text{if } 76/\left(\sqrt{F_y}\right) &lt; b/t_i &lt; 155/\left(\sqrt{F_y}\right), \ 15,500 / \left[b/t_i\right]^2 \sqrt{F_y}, &amp; \text{if } b/t_i \geq 155/\left(\sqrt{F_y}\right). \end{cases}$</td>
<td>ASD A-B5-1, ASD A-B5-2, SAM 4-3</td>
</tr>
<tr>
<td>PIPE</td>
<td>$Q_s = 1$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>$Q_s = 1$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>$Q_s = 1$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>GENERAL</td>
<td>$Q_s = 1$</td>
<td>ASD A-B5.2c</td>
</tr>
</tbody>
</table>
### Chapter III  Check/Design for AISC-ASD89

#### Table III-4
**Effective Width for Stiffened Sections**

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Effective Width for Stiffened Sections</th>
<th>Equation Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>$h_i = \begin{cases} \frac{h}{253.3} \left[ 1 - \frac{44.3}{(h/t_c)\sqrt{f}} \right], &amp; \text{if } \frac{h}{t_c} \leq \frac{195.74}{\sqrt{f}}. \ \frac{h}{t_c} &gt; \frac{195.74}{\sqrt{f}}. &amp; \text{compression only, } f = \frac{P}{A_e} \end{cases}$</td>
<td>ASD A-B5-8</td>
</tr>
<tr>
<td>BOX</td>
<td>$b_i = \begin{cases} \frac{h}{253.3} \left[ 1 - \frac{44.3}{(h/t_c)\sqrt{f}} \right], &amp; \text{if } \frac{h}{t_c} \leq \frac{195.74}{\sqrt{f}}. \ \frac{h}{t_c} &gt; \frac{195.74}{\sqrt{f}}. &amp; \text{compression only, } f = \frac{P}{A_e} \end{cases}$</td>
<td>ASD A-B5-8</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$h_i = \begin{cases} \frac{h}{253.3} \left[ 1 - \frac{50.3}{(h/t_c)\sqrt{f}} \right], &amp; \text{if } \frac{h}{t_c} \leq \frac{195.74}{\sqrt{f}}. \ \frac{h}{t_c} &gt; \frac{195.74}{\sqrt{f}}. &amp; \text{compression, flexure, } f = 0.6f_y \end{cases}$</td>
<td>ASD A-B5-7</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$b_i = b$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>DOUBLE-ANGLE</td>
<td>$b_i = b$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>ANGLE</td>
<td>$b_i = b$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>PIPE</td>
<td>$Q_i = 1.$ (However, special expression for allowable axial stress is given.)</td>
<td>ASD A-B5-9</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>Not applicable</td>
<td>—</td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>$b_i = b$</td>
<td>ASD A-B5.2c</td>
</tr>
<tr>
<td>GENERAL</td>
<td>Not applicable</td>
<td>—</td>
</tr>
</tbody>
</table>

Note: A reduction factor of 3/4 is applied on $f$ for axial-compression-only cases and if the load combination includes any wind load or seismic load (ASD A-B5.2b).
\[
F_a = \frac{12 \pi^2 E}{23(K_l/r)^2} ,
\]
if \((K_l/r)_e > C'_e\). (E2-2, A-B5-12)

where,

\[
C'_e = \sqrt{\frac{2\pi^2 E}{Q F_y}} , \text{ and } \quad (ASD E2, A-B5.2c, SAM 4)
\]

\[
(K_l/r)_e = \sqrt{\frac{\pi^2 E}{F_e}} . \quad (ASD C-E2-2, SAM 4-4)
\]

ASD Commentary (ASD C-E3) refers to the 1986 version of the AISC-LRFD code for the calculation of \(F_e\). The 1993 version of the AISC-LRFD code is the same as the 1986 version in this respect. \(F_e\) is calculated in ETABS as follows:

- **For Rectangular, I, Box, and Pipe sections:**

\[
F_e = \left[ \frac{\pi^2 EC_w}{(K_z I_z)^2} + GJ \right] \frac{1}{I_{22} + I_{33}} \quad (LRFD A-E3-5)
\]

- **For T-sections and Double-angles:**

\[
F_e = \left( \frac{F_{e22} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e22}F_{e33}H}{(F_{e22} + F_{e33})^2}} \right] \quad (LRFD A-E3-6)
\]

- **For Channels:**

\[
F_e = \left( \frac{F_{e33} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e33}F_{e33}H}{(F_{e33} + F_{e33})^2}} \right] \quad (LRFD A-E3-6)
\]

- **For Single-angle sections with equal legs:**

\[
F_e = \left( \frac{F_{e33} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e33}F_{e33}H}{(F_{e33} + F_{e33})^2}} \right] \quad (ASD SAM C-C4-1)
\]

- **For Single-angle sections with unequal legs, \(F_e\) is calculated as the minimum real root of the following cubic equation (ASD SAM C-C4-2, LRFD A-E3-7):**
\[(F_e - F_{e33}) (F_e - F_{e22}) (F_e - F_{e2}) - F_e^2 (F_e - F_{e22}) \frac{x_0^2}{r_0^2} - F_e^2 (F_e - F_{e33}) \frac{y_0^2}{r_0^2} = 0,\]

where,

\[x_0, y_0\] are the coordinates of the shear center with respect to the centroid,
\[x_0 = 0\] for double-angle and T-shaped members (y-axis of symmetry),
\[r_0 = \sqrt{x_0^2 + y_0^2 + \frac{l_{22} + l_{33}}{A_g}}\] = polar radius of gyration about the shear center,

\[H = \frac{1}{2} \left( \frac{x_0^2 + y_0^2}{r_0^2} \right),\] (LRFD A-E3-9)

\[F_{e33} = \frac{\pi^2 E}{(K_{33} l_{33} / r_{33})^2},\] (LRFD A-E3-10)

\[F_{e22} = \frac{\pi^2 E}{(K_{22} l_{22} / r_{22})^2},\] (LRFD A-E3-11)

\[F_{et} = \left[ \frac{\pi^2 E C_w}{(K_z l_z)^2} + GJ \right] \frac{1}{A r_0^2},\] (LRFD A-E3-12)

\[K_{22}, K_{33}\] are effective length factors in minor and major directions,
\[K_z\] is the effective length factor for torsional buckling, and it is taken equal to \[K_{22}\] in ETABS,
\[l_{22}, l_{33}\] are effective lengths in the minor and major directions,
\[l_z\] is the effective length for torsional buckling, and it is taken equal to \[l_{22}\].

For angle sections, the principal moment of inertia and radii of gyration are used for computing \(F_e\) (ASD SAM 4). Also, the maximum value of \(Kl\), i.e., \[\max(K_{22} l_{22}, K_{33} l_{33})\], is used in place of \(K_{22} l_{22}\) or \(K_{33} l_{33}\) in calculating \(F_{e22}\) and \(F_{e33}\) in this case.
Allowable Stress in Bending

The allowable bending stress depends on the following criteria: the geometric shape of the cross-section, the axis of bending, the compactness of the section, and a length parameter.

I-sections

For I-sections the length parameter is taken as the laterally unbraced length, \( l_{22} \), which is compared to a critical length, \( l_c \). The critical length is defined as

$$ l_c = \min \left\{ \frac{76 b_f}{\sqrt{F_y}}, \frac{20,000 A_f}{d F_y} \right\} \text{, where} \quad (ASD F1-2) $$

\( A_f \) is the area of compression flange.

Major Axis of Bending

If \( l_{22} \) is less than \( l_c \), the major allowable bending stress for Compact and Noncompact sections is taken depending on whether the section is welded or rolled and whether \( f_y \) is greater than 65 ksi or not.

For Compact sections:

\[
F_{b33} = 0.66 F_y \quad \text{if} \quad f_y \leq 65 \text{ ksi} \quad (ASD F1-1)
\]

\[
F_{b33} = 0.60 F_y \quad \text{if} \quad f_y > 65 \text{ ksi} \quad (ASD F1-5)
\]

For Noncompact sections:

\[
F_{b33} = \left( 0.79 - 0.002 \frac{b_f}{2 t_f} \sqrt{F_y} \right) F_y \quad \text{if rolled and} \quad f_y \leq 65 \text{ ksi} \quad (ASD F1-3)
\]

\[
F_{b33} = \left( 0.79 - 0.002 \frac{b_f}{2 t_f} \sqrt{k_c} \right) F_y \quad \text{if welded and} \quad f_y \leq 65 \text{ ksi} \quad (ASDF1-4)
\]

\[
F_{b33} = 0.60 F_y \quad \text{if} \quad f_y > 65 \text{ ksi} \quad (ASD F1-5)
\]

If the unbraced length \( l_{22} \) is greater than \( l_c \), then for both Compact and Noncompact I-sections the allowable bending stress depends on the \( \frac{l_{22}}{r_y} \) ratio.
For \( \frac{l_{22}}{r_T} \leq \frac{102,000 C_b}{F_y} \), \[ F_{b33} = 0.60 F_y, \] (ASD F1-6)

for \( \frac{102,000 C_b}{F_y} < \frac{l_{22}}{r_T} \leq \frac{510,000 C_b}{F_y} \),

\[
F_{b33} = \frac{2}{3} \frac{F_y (l_{22} / r_T)^2}{1530,000 C_b} F_y \leq 0.60 F_y, \quad \text{and} \quad (ASD \text{ F1-6})
\]

for \( \frac{l_{22}}{r_T} > \frac{510,000 C_b}{F_y} \),

\[
F_{b33} = \frac{170,000 C_b}{(l_{22} / r_T)^2} \leq 0.60 F_y, \quad (ASD \text{ F1-7})
\]

and \( F_{b33} \) is taken not to be less than that given by the following formula:

\[
F_{b33} = \frac{12000 C_b}{l_{22} (d / A_f)} \leq 0.6 F_y \quad (ASD \text{ F1-8})
\]

where,

\( r_T \) is the radius of gyration of a section comprising the compression flange and 1/3 the compression web taken about an axis in the plane of the web,

\[
C_b = 1.75 + 1.05 \left( \frac{M_a}{M_b} \right) + 0.3 \left( \frac{M_a}{M_b} \right)^2 \leq 2.3, \quad \text{where} \quad (ASD \text{ F1.3})
\]

\( M_a \) and \( M_b \) are the end moments of any unbraced segment of the member and \( M_a \) is numerically less than \( M_b \); \( M_a / M_b \) being positive for double curvature bending and negative for single curvature bending. Also, if any moment within the segment is greater than \( M_b \), \( C_b \) is taken as 1.0. Also, \( C_b \) is taken as 1.0 for cantilevers and frames braced against joint translation (ASD F1.3). ETABS defaults \( C_b \) to 1.0 if the unbraced length, \( l_{22} \), of the member is redefined by the
user (i.e. it is not equal to the length of the member). The user can overwrite the value of $C_b$ for any member by specifying it.

The allowable bending stress for Slender sections bent about their major axis is determined in the same way as for a Noncompact section. Then the following additional considerations are taken into account.

If the web is slender, then the previously computed allowable bending stress is reduced as follows:

$$F_{b_{33}}' = R_{PG} R_e F_{b_{33}},$$

where

$$R_{PG} = 1.0 - 0.0005 \frac{A_w}{A_f} \left[ \frac{h}{t} - \frac{760}{\sqrt{F_{b_{33}}}} \right] \leq 1.0,$$

$$R_e = \frac{12 + (3\alpha - \alpha^3) A_w}{12 + 2 A_w A_f} \leq 1.0,$$  \hspace{1cm} \text{(hybrid girders)}$$

$$R_e = 1.0,$$  \hspace{1cm} \text{(non-hybrid girders)}$$

$A_w$ = Area of web, $in^2$,

$A_f$ = Area of compression flange, $in^2$.

$$\alpha = \frac{0.6 F_y}{F_{b_{33}}} \leq 1.0$$

$F_{b_{33}}$ = Allowable bending stress assuming the section is non-compact, and

$F_{b_{33}}'$ = Allowable bending stress after considering web slenderness.

In the above expressions, $R_e$ is taken as 1, because currently ETABS deals with only non-hybrid girders.

If the flange is slender, then the previously computed allowable bending stress is taken to be limited as follows.

$$F_{b_{33}}' \leq Q_s \left( 0.6 F_y \right),$$

where

$Q_s$ is defined earlier.
**Minor Axis of Bending**

The minor direction allowable bending stress $F_{b22}$ is taken as follows:

For Compact sections:

$$F_{b22} = 0.75 F_y$$ if $f_y \leq 65 \text{ ksi}$, \hspace{1cm} (ASD F2-1)

$$F_{b22} = 0.60 F_y$$ if $f_y > 65 \text{ ksi}$. \hspace{1cm} (ASD F2-2)

For Noncompact and Slender sections:

$$F_{b22} = \left(1.075 - 0.005 \frac{b_f}{2r_f} \sqrt{\frac{F_y}{F_y}}\right) F_y,$$ if $f_y \leq 65 \text{ ksi}$. \hspace{1cm} (ASD F2-3)

$$F_{b22} = 0.60 F_y$$ if $f_y > 65 \text{ ksi}$. \hspace{1cm} (ASD F2-2)

**Channel sections**

For Channel sections the length parameter is taken as the laterally unbraced length, $l_{22}$, which is compared to a critical length, $l_c$. The critical length is defined as

$$l_c = \min \left\{ \frac{76 b_f}{\sqrt{F_y}}, \frac{20,000 A_f}{d F_y} \right\}, \text{ where}$$ \hspace{1cm} (ASD F1-2)

$A_f$ is the area of compression flange,

**Major Axis of Bending**

If $l_{22}$ is less than $l_c$, the major allowable bending stress $F_{b33}$ for Compact and Noncompact sections is taken depending on whether the section is welded or rolled and whether $f_y$ is greater than 65 ksi or not.

For Compact sections:

$$F_{b33} = 0.66 F_y$$ if $f_y \leq 65 \text{ ksi}$. \hspace{1cm} (ASD F1-1)

$$F_{b33} = 0.60 F_y$$ if $f_y > 65 \text{ ksi}$. \hspace{1cm} (ASD F1-5)

For Noncompact sections:

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{r_f} \sqrt{F_y}\right) F_y,$$ if rolled and $f_y \leq 65 \text{ ksi}$. \hspace{1cm} (ASD F1-3)
ETABS Steel Design Manual

\[ F_{b33} = \left( 0.79 - 0.002 \frac{b_f}{t_f} \sqrt{\frac{F_y}{k_y}} \right) F_y, \text{ if welded and } f_y \leq 65 \text{ ksi.} \text{ (ASD F1-4)} \]

\[ F_{b33} = 0.60 F_y \text{ if } f_y > 65 \text{ ksi.} \text{ (ASD F1-5)} \]

If the unbraced length \( l_{22} \) is greater than \( l_c \), then for both Compact and Noncompact Channel sections the allowable bending stress is taken as follows:

\[ F_{b33} = \frac{12,000 C_b}{l_{22} \left( d / A_f \right)} \leq 0.6 F_y \text{ (ASD F1-8)} \]

The allowable bending stress for Slender sections bent about their major axis is determined in the same way as for a Noncompact section. Then the following additional considerations are taken into account.

If the web is slender, then the previously computed allowable bending stress is reduced as follows:

\[ F'_{b33} = R_e R_{PG} F_{b33} \text{ (ASD G2-1)} \]

If the flange is slender, the previously computed allowable bending stress is taken to be limited as follows:

\[ F'_{b33} \leq Q_s \left( 0.6 F_y \right) \text{ (ASD A-B5.2a, A-B5.2d)} \]

The definition for \( r_T, C_b, A_f, A_w, R_e, R_{PG}, Q_s, F_{b33}, \text{ and } F'_{b33} \) are given earlier.

**Minor Axis of Bending**

The minor direction allowable bending stress \( F_{b22} \) is taken as follows:

\[ F_{b22} = 0.60 F_y \text{ (ASD F2-2)} \]

**T-sections and Double angles**

For T sections and Double angles, the allowable bending stress for both major and minor axes bending is taken as,

\[ F_b = 0.60 F_y. \]
Box Sections and Rectangular Tubes

For all Box sections and Rectangular tubes, the length parameter is taken as the laterally unbraced length, $l_{22}$, measured compared to a critical length, $l_c$. The critical length is defined as

$$l_c = \max \left\{ \left( 1950 + 1200 \frac{M_a}{M_b} \right) \frac{b}{F_y}, \frac{1200 b}{F_y} \right\}$$ (ASD F3-2)

where $M_a$ and $M_b$ have the same definition as noted earlier in the formula for $C_B$. If $l_{22}$ is specified by the user, $l_c$ is taken as $\frac{1200 b}{F_y}$ in ETABS.

**Major Axis of Bending**

If $l_{22}$ is less than $l_c$, the allowable bending stress in the major direction of bending is taken as:

$$F_{b33} = 0.66 F_y$$ (for Compact sections) (ASD F3-1)

$$F_{b33} = 0.60 F_y$$ (for Noncompact sections) (ASD F3-3)

If $l_{22}$ exceeds $l_c$, the allowable bending stress in the major direction of bending for both Compact and Noncompact sections is taken as:

$$F_{b33} = 0.60 F_y$$ (ASD F3-3)

The major direction allowable bending stress for Slender sections is determined in the same way as for a Noncompact section. Then the following additional consideration is taken into account. If the web is slender, then the previously computed allowable bending stress is reduced as follows:

$$F_{b33}' = R_e R_{PG} F_{b33}$$ (ASD G2-1)

The definition for $R_e$, $R_{PG}$, $F_{b33}$, and $F_{b33}'$ are given earlier.

If the flange is slender, no additional consideration is needed in computing allowable bending stress. However, effective section dimensions are calculated and the section modulus is modified according to its slenderness.

**Minor Axis of Bending**

If $l_{22}$ is less than $l_c$, the allowable bending stress in the minor direction of bending is taken as:
If $l_{22}$ exceeds $l_c$, the allowable bending stress in the minor direction of bending is taken, irrespective of compactness, as:

$$F_{b22} = 0.60 F_y$$  \hspace{1cm} (ASD F3-3)

**Pipe Sections**

For Pipe sections, the allowable bending stress for both major and minor axes of bending is taken as

$$F_b = 0.66 F_y \quad \text{(for Compact sections), and}$$  \hspace{1cm} (ASD F3-1)

$$F_b = 0.60 F_y \quad \text{(for Noncompact and Slender sections).}$$  \hspace{1cm} (ASD F3-3)

**Round Bars**

The allowable stress for both the major and minor axis of bending of round bars is taken as,

$$F_b = 0.75 F_y .$$ \hspace{1cm} (ASD F2-1)

**Rectangular and Square Bars**

The allowable stress for both the major and minor axis of bending of solid square bars is taken as,

$$F_b = 0.75 F_y .$$ \hspace{1cm} (ASD F2-1)

For solid rectangular bars bent about their major axes, the allowable stress is given by

$$F_b = 0.60 F_y . \quad \text{And}$$

the allowable stress for minor axis bending of rectangular bars is taken as,

$$F_b = 0.75 F_y .$$ \hspace{1cm} (ASD F2-1)
Single-Angle Sections

The allowable flexural stresses for Single-angles are calculated based on their principal axes of bending (ASD SAM 5.3).

**Major Axis of Bending**

The allowable stress for major axis bending is the minimum considering the limit state of lateral-torsional buckling and local buckling (ASD SAM 5.1).

The allowable major bending stress for Single-angles for the limit state of lateral-torsional buckling is given as follows (ASD SAM 5.1.3):

\[
F_{b,\text{major}} = \begin{cases} 
0.55 - 0.10 \frac{F_{ob}}{F_y} F_{ob}, & \text{if } F_{ob} \leq F_y \\
0.95 - 0.50 \left( \frac{F_y}{F_{ob}} \right) F_y, & \text{if } F_{ob} > F_y 
\end{cases} \quad (\text{ASD SAM 5-3a})
\]

where, \( F_{ob} \) is the elastic lateral-torsional buckling stress as calculated below.

The elastic lateral-torsional buckling stress, \( F_{ob} \), for equal-leg angles is taken as

\[
F_{ob} = C_b \frac{28.250}{l/t},
\]

and for unequal-leg angles \( F_{ob} \) is calculated as

\[
F_{ob} = 143.100 \frac{C_b}{S_{\text{major}}} \frac{I_{\text{min}}}{l^2} \left[ \sqrt{\beta_w^2 + 0.052(\frac{l t}{r_{\text{min}}})^2} + \beta_w \right], \quad (\text{ASD SAM 5-6})
\]

where,

\[
t = \min\left(t_w, t_f\right),
\]

\[
l = \max\left(l_{\text{22}}, l_{\text{33}}\right),
\]

\( I_{\text{min}} = \) minor principal moment of inertia,

\( I_{\text{max}} = \) major principal moment of inertia,

\( S_{\text{major}} = \) major section modulus for compression at the tip of one leg,

\( r_{\text{min}} = \) radius of gyration for minor principal axis.
in the above expressions $C_b$ is calculated in the same way as is done for I sections with the exception that the upper limit of $C_b$ is taken here as 1.5 instead of 2.3.

$$C_b = 1.75 + 1.05 \left( \frac{M_a}{M_b} \right) + 0.3 \left( \frac{M_a}{M_b} \right)^2 \leq 1.5 \quad \text{(ASD F1.3, SAM 5.2.2)}$$

The allowable major bending stress for Single-angles for the limit state of local buckling is given as follows (ASD SAM 5.1.1):

$$F_{b, major} = 0.66 \frac{b}{t} F_y, \quad \text{if} \quad \frac{b}{t} \leq \frac{65}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1a)}$$

$$F_{b, major} = 0.60 F_y, \quad \text{if} \quad \frac{65}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{76}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1b)}$$

$$F_{b, major} = Q \left( 0.60 F_y \right), \quad \text{if} \quad \frac{b}{t} > \frac{76}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1c)}$$

where,

$t$ = thickness of the leg under consideration,

$b$ = length of the leg under consideration, and

$Q$ = slenderness reduction factor for local buckling. \quad \text{(ASD A-B5-2, SAM 4)}

In calculating the allowable bending stress for Single-angles for the limit state of local buckling, the allowable stresses are calculated considering the fact that either of
the two tips can be under compression. The minimum allowable stress is considered.

**Minor Axis of Bending**

The allowable minor bending stress for Single-angles is given as follows (ASD SAM 5.1.1, 5.3.1b, 5.3.2b):

\[
F_{b,\text{minor}} = 0.66 F_y \quad \text{if} \quad \frac{b}{t} \leq \frac{65}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1a)}
\]

\[
F_{b,\text{minor}} = 0.60 F_y \quad \text{if} \quad \frac{65}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{76}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1b)}
\]

\[
F_{b,\text{minor}} = Q \left(0.60 F_y\right), \quad \text{if} \quad \frac{b}{t} > \frac{76}{\sqrt{F_y}}, \quad \text{(ASD SAM 5-1c)}
\]

In calculating the allowable bending stress for Single-angles it is assumed that the sign of the moment is such that both the tips are under compression. The minimum allowable stress is considered.

**General Sections**

For General sections the allowable bending stress for both major and minor axes bending is taken as,

\[
F_b = 0.60 F_y.
\]

**Allowable Stress in Shear**

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

**Major Axis of Bending**

The allowable shear stress for all sections except I, Box and Channel sections is taken in ETABS as:

\[
F_v = 0.40 F_y \quad \text{(ASD F4-1, SAM 3-1)}
\]
The allowable shear stress for major direction shears in I-shapes, boxes and channels is evaluated as follows:

\[ F_v = 0.40 \frac{F_y}{t_w}, \quad \text{if} \quad \frac{h}{t_w} \leq \frac{380}{\sqrt{F_y}}, \quad \text{and} \quad (ASD \ F4-1) \]

\[ F_v = \frac{C_v}{2.89} F_y \leq 0.40 F_y, \quad \text{if} \quad \frac{380}{\sqrt{F_y}} < \frac{h}{t_w} \leq 260. \quad (ASD \ F4-2) \]

where,

\[ C_v = \begin{cases} 
45,000 \frac{k_v}{F_y} & \text{if} \quad \frac{h}{t_w} \geq 56,250 \frac{k_v}{F_y}, \\
190 \frac{k_v}{\sqrt{F_y}} & \text{if} \quad \frac{h}{t_w} < 56,250 \frac{k_v}{F_y}, \\
t_w \sqrt{F_y} & \text{if} \quad \frac{h}{t_w} \leq 56,250 \frac{k_v}{F_y}.
\end{cases} \quad (ASD \ F4) \]

\[ k_v = \begin{cases} 
4.00 + \frac{5.34}{(a/h)^2} & \text{if} \quad \frac{a}{h} \leq 1, \\
5.34 + \frac{4.00}{(a/h)^2} & \text{if} \quad \frac{a}{h} > 1.
\end{cases} \quad (ASD \ F4) \]

\[ t_w = \text{Thickness of the web,} \]

\[ a = \text{Clear distance between transverse stiffeners, in. Currently it is taken conservatively as the length, } l_{22}, \text{ of the member in ETABS,} \]

\[ h = \text{Clear distance between flanges at the section, in.} \]

**Minor Axis of Bending**

The allowable shear stress for minor direction shears is taken as:

\[ F_v = 0.40 F_y \quad (ASD \ F4-1, \ SAM \ 3-1) \]

**Calculation of Stress Ratios**

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

During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.

**Axial and Bending Stresses**

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

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

  $$
  \frac{f_a}{F_a} + \frac{C_{m33} f_{33}}{F_{33}} + \frac{C_{m22} f_{22}}{F_{22}}, \quad \text{and} \quad (ASD \ H1-1, SAM 6.1)
  $$

  $$
  \frac{f_a}{Q(0.60 F_y)} + \frac{f_{33}}{F_{33}} + \frac{f_{22}}{F_{22}}, \quad \text{where} \quad (ASD \ H1-2, SAM 6.1)
  $$

  $f_a$, $f_{33}$, $f_{22}$, $F_a$, $F_{33}$, and $F_{22}$ are defined earlier in this chapter, $C_{m33}$ and $C_{m22}$ are coefficients representing distribution of moment along the member length.

$$
C_m = \begin{cases} 
1.00, & \text{if length is overwritten,} \\
1.00, & \text{if tension member,} \\
0.85, & \text{if sway frame,} \\
0.6 - 0.4 \frac{M_a}{M_b}, & \text{if nonsway, no transverse loading,} \\
0.85, & \text{if nonsway, trans. load, end restrained,} \\
1.00, & \text{if nonsway, trans. load, end unrestrained.}
\end{cases} \quad (ASD \ H1)
$$

For sway frame $C_m = 0.85$, for nonsway frame without transverse load $C_m = 0.6 - 0.4 M_a / M_b$, for nonsway frame with transverse load and end restrained compression member $C_m = 0.85$, and for nonsway frame with transverse load and end unrestrained compression member $C_m = 1.00$ (ASD H1), where $M_a / M_b$ is the ratio of the smaller to the larger moment at the ends of the
member, \( M_a / M_b \) being positive for double curvature bending and negative for single curvature bending. When \( M_b \) is zero, \( C_m \) is taken as 1.0. The program defaults \( C_m \) to 1.0 if the unbraced length factor, \( l \), of the member is redefined by either the user or the program, i.e., if the unbraced length is not equal to the length of the member. The user can overwrite the value of \( C_m \) for any member. \( C_m \) assumes two values, \( C_{m22} \) and \( C_{m33} \), associated with the major and minor directions.

\( F_e' \) is given by

\[
F_e' = \frac{12\pi^2 E}{23(KI / r)^2}.
\]  

ASD H1)

A factor of \( 4/3 \) is applied on \( F_e' \) and \( 0.6F_e \) if the load combination includes any wind load or seismic load (ASD H1, ASD A5.2).

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

\[
\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}
\]  

ASD H1-3, SAM 6.1)

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

\[
\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{and}
\]

\[
\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}
\]

ASD H2-1, SAM 6.2)

where

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

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

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

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

Shear Stresses

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

\[
\frac{f_{v2}}{F_v} \quad \text{and} \quad \frac{f_{v3}}{F_v}
\]

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

When designing for combinations involving earthquake and wind loads, allowable shear stresses are increased by a factor of 4/3 of the regular allowable value (ASD A5.2).
Check/Design for AISC-LRFD93

This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the AISC-LRFD93 design code (AISC 1993). Various notations used in this chapter are described in Table IV-1.

For referring to pertinent sections and equations of the original LRFD code, a unique prefix “LRFD” is assigned. However, all references to the “Specifications for Load and Resistance Factored Design of Single-Angle Members” (AISC 1994) carry the prefix of “LRFD SAM”.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this chapter. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ratio is also calculated separately.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Cross-sectional area, $in^2$</td>
</tr>
<tr>
<td>$A_e$</td>
<td>Effective cross-sectional area for slender sections, $in^2$</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross cross-sectional area, $in^2$</td>
</tr>
<tr>
<td>$A_{s1}, A_{s2}$</td>
<td>Major and minor shear areas, $in^2$</td>
</tr>
<tr>
<td>$A_w$</td>
<td>Shear area, equal $dt$ per web, $in^2$</td>
</tr>
<tr>
<td>$B_1$</td>
<td>Moment magnification factor for moments not causing sidesway</td>
</tr>
<tr>
<td>$B_2$</td>
<td>Moment magnification factor for moments causing sidesway</td>
</tr>
<tr>
<td>$C_b$</td>
<td>Bending coefficient</td>
</tr>
<tr>
<td>$C_m$</td>
<td>Moment coefficient</td>
</tr>
<tr>
<td>$C_w$</td>
<td>Warping constant, $in^6$</td>
</tr>
<tr>
<td>$D$</td>
<td>Outside diameter of pipes, $in$</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity, ksi</td>
</tr>
<tr>
<td>$F_{cr}$</td>
<td>Critical compressive stress, ksi</td>
</tr>
<tr>
<td>$F_r$</td>
<td>Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi</td>
</tr>
<tr>
<td>$F_y$</td>
<td>Yield stress of material, ksi</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus, ksi</td>
</tr>
<tr>
<td>$I_{22}$</td>
<td>Minor moment of inertia, $in^4$</td>
</tr>
<tr>
<td>$I_{33}$</td>
<td>Major moment of inertia, $in^4$</td>
</tr>
<tr>
<td>$J$</td>
<td>Torsional constant for the section, $in^4$</td>
</tr>
<tr>
<td>$K$</td>
<td>Effective length factor</td>
</tr>
<tr>
<td>$K_{33}, K_{22}$</td>
<td>Effective length K-factors in the major and minor directions</td>
</tr>
<tr>
<td>$L_n$</td>
<td>Laterally unbraced length of member, $in$</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Limiting laterally unbraced length for full plastic capacity, $in$</td>
</tr>
<tr>
<td>$L_r$</td>
<td>Limiting laterally unbraced length for inelastic lateral-torsional buckling, $in$</td>
</tr>
<tr>
<td>$M_{cr}$</td>
<td>Elastic buckling moment, kip-in</td>
</tr>
<tr>
<td>$M_o$</td>
<td>Factored moments causing sidesway, kip-in</td>
</tr>
<tr>
<td>$M_w$</td>
<td>Factored moments not causing sidesway, kip-in</td>
</tr>
<tr>
<td>$M_{33}, M_{22}$</td>
<td>Nominal bending strength in major and minor directions, kip-in</td>
</tr>
<tr>
<td>$M_{ob}$</td>
<td>Elastic lateral-torsional buckling moment for angle sections, kip-in</td>
</tr>
<tr>
<td>$M_{13}, M_{12}$</td>
<td>Major and minor limiting buckling moments, kip-in</td>
</tr>
<tr>
<td>$M_o$</td>
<td>Factored moment in member, kip-in</td>
</tr>
<tr>
<td>$M_{33}, M_{22}$</td>
<td>Factored major and minor moments in member, kip-in</td>
</tr>
<tr>
<td>$P_e$</td>
<td>Euler buckling load, kips</td>
</tr>
<tr>
<td>$P_n$</td>
<td>Nominal axial load strength, kip</td>
</tr>
<tr>
<td>$P_u$</td>
<td>Factored axial force in member, kips</td>
</tr>
<tr>
<td>$P_y$</td>
<td>$A_y F_y$, kips</td>
</tr>
<tr>
<td>$Q$</td>
<td>Reduction factor for slender section, $= Q_o Q_s$</td>
</tr>
</tbody>
</table>

**Table IV-1**

*AISC-LRFD Notations*
$Q_a$ = Reduction factor for stiffened slender elements
$Q_s$ = Reduction factor for unstiffened slender elements
$S$ = Section modulus, in³
$S_{eff,33}, S_{eff,22}$ = Effective major and minor section moduli for slender sections, in³
$S_c$ = Section modulus for compression in an angle section, in³
$V_{n2}, V_{n3}$ = Nominal major and minor shear strengths, kips
$V_{u2}, V_{u3}$ = Factored major and minor shear loads, kips
$Z$ = Plastic modulus, in³
$Z_{eff,33}, Z_{eff,22}$ = Major and minor plastic moduli, in³
$b$ = Nominal dimension of plate in a section, in longer leg of angle sections, $b_f - 2t_{ef}$ for welded and $b_f - 3t_{w}$ for rolled box sections, etc.
$b_e$ = Effective width of flange, in
$b_f$ = Flange width, in
$d$ = Overall depth of member, in
$d_e$ = Effective depth of web, in
$h_c$ = Clear distance between flanges less fillets, in assumed $d - 2k$ for rolled sections, and $d - 2t_f$ for welded sections
$k$ = Distance from outer face of flange to web toe of fillet, in
$k_c$ = Parameter used for section classification, $4 \sqrt{\frac{h_c}{t_w}}, 0.35 \leq k_c \leq 0.763$
$l_{33}, l_{22}$ = Major and minor direction unbraced member lengths, in
$r$ = Radius of gyration, in
$r_{33}, r_{22}$ = Radii of gyration in the major and minor directions, in
$t$ = Thickness, in
$t_f$ = Flange thickness, in
$t_w$ = Thickness of web, in
$\beta_w$ = Special section property for angles, in
$\lambda$ = Slenderness parameter
$\lambda_c, \lambda_e$ = Column slenderness parameters
$\lambda_{cp}$ = Limiting slenderness parameter for compact element
$\lambda_c = $ Limiting slenderness parameter for non-compact element
$\lambda_s = $ Limiting slenderness parameter for seismic element
$\lambda_{slender} = $ Limiting slenderness parameter for slender element
$\phi_b$ = Resistance factor for bending, 0.9
$\phi_c$ = Resistance factor for compression, 0.85
$\phi_t$ = Resistance factor for tension, 0.9
$\phi_v$ = Resistance factor for shear, 0.9
English as well as SI and MKS metric units can be used for input. But the code is based on Kip-Inch-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to Kip-Inch-Second units unless otherwise noted.

Design Loading Combinations

The design load combinations are the various combinations of the load cases for which the structure needs to be checked. For the AISC-LRFD93 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, then the following load combinations may have to be defined (LRFD A4.1):

1. 1.4 DL
2. 1.2 DL + 1.6 LL
3. 0.9 DL ± 1.3 WL
4. 1.2 DL ± 1.3 WL
5. 1.2 DL + 0.5 LL ± 1.3 WL
6. 0.9 DL ± 1.0 EL
7. 1.2 DL ± 1.0 EL
8. 1.2 DL + 0.5 LL ± 1.0 EL

These are also the default design load combinations in ETABS whenever the AISC-LRFD93 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

When using the AISC-LRFD93 code, ETABS design assumes that a P-Δ analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-Δ analysis be done at the factored load level of 1.2 DL plus 0.5 LL (White and Hajjar 1991).

Classification of Sections

The nominal strengths for axial compression and flexure are dependent on the classification of the section as Compact, Noncompact, Slender or Too Slender. ETABS
AISC-LRFD93: Axes Conventions

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

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

Figure IV-1
AISC-LRFD Definition of Geometric Properties
<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Check ( \lambda )</th>
<th>COMPACT ( (\lambda_p) )</th>
<th>NONCOMPACT ( \lambda_r )</th>
<th>SLENDER ( \lambda_{\text{slender}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE ( b_y/2t_y ) (rolled)</td>
<td>( \leq \frac{65}{\sqrt{F_y}} )</td>
<td>( \leq \frac{141}{\sqrt{F_y} - 100} )</td>
<td>No limit</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \leq \frac{65}{\sqrt{F_y}} )</td>
<td>( \leq \frac{162}{\sqrt{F_y - 16.5}} )</td>
<td>No limit</td>
<td></td>
</tr>
<tr>
<td>( h_y/t_y )</td>
<td>For ( P_y/\phi_y P_y \leq 0.125 )</td>
<td>( \leq \frac{640}{\sqrt{F_y}} \left( 1 - \frac{2.75 P_y}{\phi_y P_y} \right) )</td>
<td>For ( P_y/\phi_y P_y &gt; 0.125 )</td>
<td>( \leq \frac{970}{\sqrt{F_y}} \left( 1 - 0.74 \frac{P_y}{\phi_y P_y} \right) )</td>
</tr>
<tr>
<td>BOX ( b/t ) ( h_y/t_y )</td>
<td>( \leq \frac{190}{\sqrt{F_y}} )</td>
<td>No limit</td>
<td>( \leq \frac{238}{\sqrt{F_y}} )</td>
<td>No limit</td>
</tr>
<tr>
<td>CHANNEL ( b_y/t_y ) ( h_y/t_y )</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>T-SHAPE ( b_y/2t_y ) ( d/t_y )</td>
<td>As for I-Shapes</td>
<td>As for I-Shapes</td>
<td>No limit</td>
<td>No limit</td>
</tr>
<tr>
<td>ANGLE ( b/t )</td>
<td>Not applicable</td>
<td>( \leq \frac{76}{\sqrt{F_y}} )</td>
<td>No limit</td>
<td></td>
</tr>
<tr>
<td>DOUBLE-ANGLE ( b/t ) (Separated)</td>
<td>Not applicable</td>
<td>( \leq \frac{76}{\sqrt{F_y}} )</td>
<td>No limit</td>
<td></td>
</tr>
<tr>
<td>PIPE ( D/t )</td>
<td>( \leq \frac{2070}{F_y} )</td>
<td>( \leq \frac{8970}{F_y} )</td>
<td>( \leq \frac{13000}{F_y} ) (Compression only)</td>
<td>No limit for flexure</td>
</tr>
</tbody>
</table>

| ROUND BAR | — | Assumed Compact |
| RECTANGULAR | — | Assumed Noncompact |
| GENERAL | — | Assumed Noncompact |

**Table IV-2**  
 Limiting Width-Thickness Ratios for  
 Classification of Sections in Flexure based on AISC-LRFD
Table IV-3
Limiting Width-Thickness Ratios for
Classification of Sections (Special Cases) based on AISC-LRFD

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Width-Thickness Ratio $\lambda$</th>
<th>NONCOMPACT (Uniform Compression) $(M_{22} \approx M_{33} \approx 0)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE (rolled)</td>
<td>$b_i / 2t_i$</td>
<td>$\leq 95 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>I-SHAPE (welded)</td>
<td>$b_i / 2t_i$</td>
<td>$\leq 95 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>I-SHAPE</td>
<td>$h_i / t_i$</td>
<td>$\leq 253 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>BOX</td>
<td>$b / t_i$</td>
<td>$\leq 238 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>CHANNELE</td>
<td>$b_i / t_i$</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$h_i / t_i$</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$b_i / 2t_i$</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$d / t_i$</td>
<td>$\leq 127 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>ANGLE</td>
<td>$b / t$</td>
<td>$\leq 76 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>DOUBLE-ANGLE</td>
<td>$b / t$</td>
<td>$\leq 76 / \sqrt{F_y}$</td>
</tr>
<tr>
<td>PIPE</td>
<td>$D / t$</td>
<td>$\leq 3300 / F_y$</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td>Assumed Compact</td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>—</td>
<td>Assumed Noncompact</td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td>Assumed Noncompact</td>
</tr>
</tbody>
</table>
classifies individual members according to the limiting width/thickness ratios given in Table IV-2 and Table IV-3 (LRFD B5.1, A-G1, Table A-F1.1). The definition of the section properties required in these tables is given in Figure IV-1 and Table IV-1. Moreover, special considerations are required regarding the limits of width-thickness ratios for Compact sections in Seismic zones and Noncompact sections with compressive force as given in Table IV-3. If the limits for Slender sections are not met, the section is classified as Too Slender. **Stress check of Too Slender sections is beyond the scope of ETABS.**

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

**Calculation of Factored Forces**

The factored member loads that are calculated for each load combination are $P_u$, $M_{u31}$, $M_{u22}$, $V_{u2}$ and $V_{u3}$ corresponding to factored values of the axial load, the major moment, the minor moment, the major direction shear force and the minor direction shear force, respectively. These factored loads are calculated at each of the previously defined stations.

For loading combinations that cause compression in the member, the factored moment $M_u$ ($M_{u31}$ and $M_{u22}$ in the corresponding directions) is magnified to consider second order effects. The magnified moment in a particular direction is given by:

$$M = B_1 M_n + B_2 M_l$$

(LRFD C1-1, SAM 6)

where

- $B_1 = \text{Moment magnification factor for non-sidesway moments,}$
- $B_2 = \text{Moment magnification factor for sidesway moments,}$
- $M_n = \text{Factored moments not causing sidesway,}$
- $M_l = \text{Factored moments causing sidesway.}$

The moment magnification factors are associated with corresponding directions. The moment magnification factor $B_1$ for moments not causing sidesway is given by

$$B_1 = \frac{C_m}{(1 - P_u / P_e)} \geq 1.0$$

(LRFD C1-2, SAM 6-2)

where $P_e$ is the Euler buckling load ($P_e = \frac{A_s F_y}{\lambda^2}$, with $\lambda = \frac{Kl}{r\pi} \sqrt{\frac{F_y}{E}}$), and
$C_{m33}$ and $C_{m22}$ are coefficients representing distribution of moment along the member length.

$$C_m = \begin{cases} 
1.00, & \text{if length is overwritten,} \\
1.00, & \text{if tension member,} \\
1.00, & \text{if end unrestrained,} \\
0.6 - 0.4 \frac{M_a}{M_b}, & \text{if no transverse loading,} \\
0.85, & \text{if trans. load, end restrained,} \\
1.00, & \text{if trans. load, end unrestrained,}
\end{cases}$$

(LRFD C1-3)

$M_a / M_b$ is the ratio of the smaller to the larger moment at the ends of the member, $M_a / M_b$ being positive for double curvature bending and negative for single curvature bending. For tension members $C_m$ is assumed as 1.0. For compression members with transverse load on the member, $C_m$ is assumed as 1.0 for members with any unrestrained end and as 0.85 for members with two unrestrained ends. When $M_b$ is zero, $C_m$ is taken as 1.0. The program defaults $C_m$ to 1.0 if the unbraced length factor, $l$, of the member is redefined by either the user or the program, i.e., if the unbraced length is not equal to the length of the member. The user can overwrite the value of $C_m$ for any member. $C_m$ assumes two values, $C_{m22}$ and $C_{m33}$, associated with the major and minor directions.

The magnification factor $B_1$, must be a positive number. Therefore $P_u$ must be less than $P_e$. If $P_u$ is found to be greater than or equal to $P_e$, a failure condition is declared.

ETABS design assumes the analysis includes P-$\Delta$ effects, therefore $B_2$ is taken as unity for bending in both directions. It is suggested that the P-$\Delta$ analysis be done at the factored load level of 1.2 DL plus 0.5 LL (LRFD C2.2). See also White and Hajjar (1991).

For single angles, where the principal axes of bending are not coincident with the geometric axes (2-2 and 3-3), the program conservatively uses the maximum of $K_{33} l_{33}$ and $K_{22} l_{22}$ for determining the major and minor direction Euler buckling capacity.

If the program assumptions are not satisfactory for a particular structural model or member, the user has a choice of explicitly specifying the values of $B_1$ and $B_2$ for any member.
Calculation of Nominal Strengths

The nominal strengths in compression, tension, bending, and shear are computed for Compact, Noncompact, and Slender sections according to the following subsections. The nominal flexural strengths for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I, Box, Channel, Circular, Pipe, T, and Double-angle sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations except shear are based on that.

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

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

$$
\begin{align*}
\varphi_t &= \text{Resistance factor for tension, 0.9} \quad \text{(LRFD D1, H1, SAM 2, 6)} \\
\varphi_c &= \text{Resistance factor for compression, 0.85} \quad \text{(LRFD E2, E3, H1)} \\
\varphi_c' &= \text{Resistance factor for compression in angles, 0.90} \quad \text{(LRFD SAM 4, 6)} \\
\varphi_b &= \text{Resistance factor for bending, 0.9} \quad \text{(LRFD F1, H1, A-F1, A-G2, SAM 5)} \\
\varphi_s &= \text{Resistance factor for shear, 0.9} \quad \text{(LRFD F2, A-F2, A-G3, SAM 3)}
\end{align*}
$$

If the user specifies nonzero factored strengths for one or more elements in the “Capacity Overwrites” form, these values will override the above mentioned calculated values for those elements. The specified factored strengths should be based on the principal axes of bending.

Compression Capacity

The nominal compression strength is the minimum value obtained from flexural buckling, torsional buckling and flexural-torsional buckling. The strengths are determined according to the following subsections.

For members in compression, if $Kl/r$ is greater than 200, a message to that effect is printed (LRFD B7, SAM 4). For single angles, the minimum radius of gyration, $r_{zz}$, is used instead of $r_{z2}$ and $r_{z3}$ in computing $Kl/r$.

Flexural Buckling

The nominal axial compressive strength, $P_n$, depends on the slenderness ratio, $Kl/r$, and its critical value, $\lambda_c$, where
\[\frac{Kl}{r} = \max\left\{\frac{K_{33} l_{33}}{r_{33}}, \frac{K_{22} l_{22}}{r_{22}}\right\}, \text{ and}\]
\[\lambda_c = \frac{Kl}{r\pi \sqrt{\frac{F_y}{E}}}.\]  

(LRFD E2-4, SAM 4)

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

\(P_n\) for Compact or Noncompact sections is evaluated for flexural buckling as follows:

\[P_n = A_f F_{cr}, \text{ where}\]
\[F_{cr} = \left(0.658\frac{\lambda_c^2}{\lambda_c^2 + 1}\right) F_y, \text{ for } \lambda_c \leq 1.5, \text{ and}\]
\[F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y, \text{ for } \lambda_c > 1.5.\]  

(LRFD E2-2)

\[P_n\] for Slender sections is evaluated for flexural buckling as follows:

\[P_n = A_f F_{cr}, \text{ where}\]
\[F_{cr} = Q \left(0.658\frac{\lambda_c^2}{\lambda_c^2 + 1}\right) F_y, \text{ for } \lambda_c \sqrt{Q} \leq 1.5, \text{ and}\]
\[F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y, \text{ for } \lambda_c \sqrt{Q} > 1.5.\]  

(LRFD A-B5-16, SAM 4-2)

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

\[Q = Q_s Q_a, \text{ where}\]
\[Q_s = \text{reduction factor for unstiffened slender elements, and }\]
\[Q_a = \text{reduction factor for stiffened slender elements.}\]  

(LRFD A-B5.3a, LRFD A-B5.3c)

The \(Q_s\) factors for slender sections are calculated as described in Table IV-4 (LRFD A-B5.3a). The \(Q_s\) factors for slender sections are calculated as the ratio of effective cross-sectional area and the gross cross-sectional area (LRFD A-B5.3c).
<table>
<thead>
<tr>
<th>Section Type</th>
<th>Reduction Factor for Unstiffened Slender Elements ((Q_s))</th>
<th>Equation Reference</th>
</tr>
</thead>
</table>
| I-SHAPE      | \[
Q_s = \begin{cases} 
1.0, & \text{if } \frac{b_t}{2t_f} \leq \frac{95}{\sqrt{F_y}}, \\
1.415 - 0.00437[\frac{b_t}{2t_f}]\sqrt{\frac{F_y}{k}}, & \text{if } \frac{95}{\sqrt{F_y}} < \frac{b_t}{2t_f} < \frac{176}{\sqrt{F_y}}, \\
20,000/\left\{\left[\frac{b_t}{2t_f}\right]^2 F_y\right\}, & \text{if } \frac{b_t}{2t_f} \geq \frac{176}{\sqrt{F_y}}.
\end{cases}
\] (rolled) | LRFD A-B5-5, LRFD A-B5-6 |
|              | \[
Q_s = \begin{cases} 
1.0, & \text{if } \frac{b_t}{2t_f} \leq \frac{109}{\sqrt{F_y/k}}, \\
1.415 - 0.0038[\frac{b_t}{2t_f}]\sqrt{\frac{F_y}{k}}, & \text{if } \frac{109}{\sqrt{F_y/k}} < \frac{b_t}{2t_f} < \frac{200}{\sqrt{F_y/k}}, \\
26,200k/\left\{\left[\frac{b_t}{2t_f}\right]^2 F_y\right\}, & \text{if } \frac{b_t}{2t_f} \geq \frac{200}{\sqrt{F_y/k}}.
\end{cases}
\] (welded) | LRFD A-B5-7, LRFD A-B5-8 |
| BOX          | \(Q_s = 1\)                                               | LRFD A-B5.3d      |
| CHANNEL      | For flanges, as for flanges in I-shapes. For web see below. | LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8 |
| T-SHAPE      | \[
Q_s = \begin{cases} 
1.0, & \text{if } \frac{d}{t} \leq \frac{127}{\sqrt{F_y}}, \\
1.908 - 0.0015[\frac{d}{t}]\sqrt{\frac{F_y}{k}}, & \text{if } \frac{127}{\sqrt{F_y}} < \frac{d}{t} < \frac{176}{\sqrt{F_y}}, \\
20,000/\left\{\left[\frac{d}{t}\right]^2 F_y\right\}, & \text{if } \frac{d}{t} \geq \frac{176}{\sqrt{F_y}}.
\end{cases}
\] | LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8, LRFD A-B5-9, LRFD A-B5-10 |
| DOUBLE-ANGLE (Separated) | \[
Q_s = \begin{cases} 
1.0, & \text{if } \frac{b}{t} \leq \frac{67}{\sqrt{F_y}}, \\
1.340 - 0.0047[\frac{b}{t}]\sqrt{\frac{F_y}{k}}, & \text{if } \frac{67}{\sqrt{F_y}} < \frac{b}{t} < \frac{155}{\sqrt{F_y}}, \\
15,500/\left\{\left[\frac{b}{t}\right]^2 F_y\right\}, & \text{if } \frac{b}{t} \geq \frac{155}{\sqrt{F_y}}.
\end{cases}
\] | LRFD A-B5-3, LRFD A-B5-4 |
| ANGLE        | \[
Q_s = \begin{cases} 
1.0, & \text{if } \frac{b}{t} \leq \frac{446}{\sqrt{F_y/E}}, \\
1.34 - 0.76[\frac{b}{t}]\sqrt{\frac{F_y/E}{k}}, & \text{if } \frac{446}{\sqrt{F_y/E}} < \frac{b}{t} < \frac{910}{\sqrt{F_y/E}}, \\
0.534/\left\{\left[\frac{b}{t}\right]^2 [F_y/E]\right\}, & \text{if } \frac{b}{t} \geq \frac{910}{\sqrt{F_y/E}}.
\end{cases}
\] | LRFD SAM4-3 |
| PIPE         | \(Q_s = 1\)                                               | LRFD A-B5.3d      |
| ROUND BAR    | \(Q_s = 1\)                                               | LRFD A-B5.3d      |
| RECTANGULAR  | \(Q_s = 1\)                                               | LRFD A-B5.3d      |
| GENERAL      | \(Q_s = 1\)                                               | LRFD A-B5.3d      |

Table IV-4

Reduction Factor for Unstiffened Slender Elements, \(Q_s\)

60 Calculation of Nominal Strengths
<table>
<thead>
<tr>
<th>Section Type</th>
<th>Effective Width for Stiffened Sections</th>
<th>Equation Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>[ h_r = \frac{h}{326 \sqrt{f}} \left[ 1 - \frac{57.2}{(h/t_a)\sqrt{f}} \right], ] if ( \frac{h}{t_a} \leq \frac{253}{\sqrt{f}} ) (compression only, ( f = \frac{P}{A_t} ))</td>
<td>LRFD A-B5-12</td>
</tr>
<tr>
<td>BOX</td>
<td>[ h_r = \frac{b}{326 \sqrt{f}} \left[ 1 - \frac{57.2}{(b/t_a)\sqrt{f}} \right], ] if ( \frac{b}{t_a} \leq \frac{238}{\sqrt{f}} ) (compression only, ( f = \frac{P}{A_t} )) [ b_r = \frac{b}{326 \sqrt{f}} \left[ 1 - \frac{64.9}{(b/t_a)\sqrt{f}} \right], ] if ( \frac{b}{t_a} \geq \frac{238}{\sqrt{f}} ) (compr. or flexure, ( f = F_y ))</td>
<td>LRFD A-B5-11</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>[ h_r = \frac{h}{326 \sqrt{f}} \left[ 1 - \frac{57.2}{(h/t_a)\sqrt{f}} \right], ] if ( \frac{h}{t_a} \leq \frac{253}{\sqrt{f}} ) (compression only, ( f = \frac{P}{A_t} ))</td>
<td>LRFD A-B5-12</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>( b_r = b )</td>
<td>LRFD A-B5.3b</td>
</tr>
<tr>
<td>DOUBLE-ANGLE (Separated)</td>
<td>( b_r = b )</td>
<td>LRFD A-B5.3b</td>
</tr>
<tr>
<td>ANGLE</td>
<td>( b_r = b )</td>
<td>LRFD A-B5.3b</td>
</tr>
<tr>
<td>PIPE</td>
<td>[ Q_s = \begin{cases} 1, &amp; \text{if } \frac{D}{t} \leq \frac{3,300}{F_s} \text{ (compression only)} \ 1,100 \left( \frac{D}{t} \right) F_s + \frac{2}{3}, &amp; \text{if } \frac{D}{t} &gt; \frac{3,300}{F_s} \end{cases} ]</td>
<td>LRFD A-B5-13</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>Not applicable</td>
<td>—</td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>( b_r = b )</td>
<td>LRFD A-B5.3b</td>
</tr>
<tr>
<td>GENERAL</td>
<td>Not applicable</td>
<td>—</td>
</tr>
</tbody>
</table>

**Table IV-5**

*Effective Width for Stiffened Sections*
The effective cross-sectional area is computed based on effective width as follows:

\[ A_x = A_y - \sum (b - b_e) t \]

\( b_e \) for unstiffened elements is taken equal to \( b \), and \( b_e \) for stiffened elements is taken equal to or less than \( b \) as given in Table IV-5 (LRFD A-B5.3b). For webs in I, box, and Channel sections, \( h_e \) is used as \( b_e \) and \( h \) is used as \( b \) in the above equation.

**Flexural-Torsional Buckling**

\( P_n \) for flexural-torsional buckling of Double-angle and T-shaped compression members whose elements have width-thickness ratios less than \( \lambda_e \) is given by

\[
P_n = A_y F_{crf} \text{, where (LRFD E3-1)}
\]

\[
F_{crf} = \left( \frac{F_{cr2} + F_{cr}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{cr2}F_{cr}}{(F_{cr} + F_{cr2})^2}} \right], \text{ where (LRFD E3-1)}
\]

\[
F_{cr} = \frac{GJ}{Ar_0^2},
\]

\[
H = 1 - \left( \frac{x_0^2 + y_0^2}{r_0^2} \right)
\]

\( r_0 = \) Polar radius of gyration about the shear center,

\( x_0, y_0 \) are the coordinates of the shear center with respect to the centroid,

\( x_0 = 0 \) for double-angle and T-shaped members (y-axis of symmetry),

\( F_{cr2} \) is determined according to the equation LRFD E2-1 for flexural buckling about the minor axis of symmetry for \( \lambda_e = \frac{Kl}{\pi r_{22}^2} \sqrt{\frac{F}{E}} \).

**Torsional and Flexural-Torsional Buckling**

The strength of a compression member, \( P_n \), determined by the limit states of torsional and flexural-torsional buckling is determined as follows:

\[
P_n = A_y F_{cr} \text{, where (LRFD A-E3-1)}
\]
\[ F_{cr} = Q \left( 0.658^{\frac{3\sqrt{Q}}{F_y}} \right) F_y, \quad \text{for } \lambda_e \sqrt{Q} \leq 1.5, \quad \text{and} \quad (LRFD\ A-E3-2) \]

\[ F_{cr} = \left[ \frac{0.877}{\lambda_e^2} \right] F_y, \quad \text{for } \lambda_e \sqrt{Q} > 1.5. \quad (LRFD\ A-E3-3) \]

In the above equations, the slenderness parameter \( \lambda_e \) is calculated as

\[ \lambda_e = \sqrt{\frac{F_y}{F_e}}, \quad (LRFD\ A-E3-4) \]

where \( F_e \) is calculated as follows:

- For Rectangular, I, Box, and Pipe sections:
  \[ F_e = \left[ \frac{\pi^2 EC_w}{(K^* I_z)^2 + GJ} \right] \frac{1}{I_{22} + I_{33}} \quad (LRFD\ A-E3-5) \]

- For T-sections and Double-angles:
  \[ F_e = \left( \frac{F_{e22} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e22}F_{e33}H}{(F_{e22} + F_{e33})^2}} \right] \quad (LRFD\ A-E3-6) \]

- For Channels:
  \[ F_e = \left( \frac{F_{e33} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e33}F_{e33}H}{(F_{e33} + F_{e33})^2}} \right] \quad (LRFD\ A-E3-6) \]

- For Single-angles sections with equal legs:
  \[ F_e = \left( \frac{F_{e33} + F_{e33}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{e33}F_{e33}H}{(F_{e33} + F_{e33})^2}} \right] \quad (LRFD\ A-E3-6) \]

- For Single-angle sections with unequal legs, \( F_e \) is calculated as the minimum real root of the following cubic equation (LRFD A-E3-7):

\[
(F_e - F_{e33}) (F_e - F_{e22}) (F_e - F_{e22}) - F_e^2 (F_e - F_{e22}) \left( x_0^2 - \frac{F_e^2 (F_e - F_{e33})}{r_0^2} \right) = 0,
\]

where,
$x_0, y_0$ are the coordinates of the shear center with respect to the centroid, $x_0 = 0$ for double-angle and T-shaped members ($y$-axis of symmetry),

$$r_0 = \sqrt{x_0^2 + y_0^2 + \frac{l_{22} + l_{33}}{A_g}} = \text{polar radius of gyration about the shear center},$$

$$H = 1 - \left(\frac{x_0^2 + y_0^2}{r_0^2}\right), \quad \text{(LRFD A-E3-9)}$$

$$F_{e33} = \frac{\pi^2 E}{\left(K_{33} l_{33}/r_{33}\right)^2}, \quad \text{(LRFD A-E3-10)}$$

$$F_{e22} = \frac{\pi^2 E}{\left(K_{22} l_{22}/r_{22}\right)^2}, \quad \text{(LRFD A-E3-11)}$$

$$F_{ec} = \left[\frac{\pi^2 E C_w}{\left(K_e l_e\right)^2} + GJ\right] \frac{1}{A r_0^2}, \quad \text{(LRFD A-E3-12)}$$

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

$K_e$ is the effective length factor for torsional buckling, and it is taken equal to $K_{22}$ in ETABS,

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

$l_e$ is the effective length for torsional buckling, and it is taken equal to $l_{22}$.

For angle sections, the principal moment of inertia and radii of gyration are used for computing $F_e$. Also, the maximum value of $Kl$, i.e., $\max(K_{22} l_{22}, K_{33} l_{33})$, is used in place of $K_{22} l_{22}$ or $K_{33} l_{33}$ in calculating $F_{e22}$ and $F_{e33}$ in this case.

**Tension Capacity**

The nominal axial tensile strength value $P_n$ is based on the gross cross-sectional area and the yield stress.

$$P_n = A_g F_y \quad \text{(LRFD D1-1)}$$

**It should be noted that no net section checks are made.** For members in tension, if $l/r$ is greater than 300, a message to that effect is printed (LRFD B7, SAM 2). For
single angles, the minimum radius of gyration, \( r_{z} \), is used instead of \( r_{22} \) and \( r_{33} \) in computing \( Kt/r \).

**Nominal Strength in Bending**

The nominal bending strength depends on the following criteria: the geometric shape of the cross-section, the axis of bending, the compactness of the section, and a slenderness parameter for lateral-torsional buckling. The nominal strengths for all shapes of sections are calculated based on their principal axes of bending. For the Rectangular, I, Box, Channel, Circular, Pipe, T, and Double-angle sections, the principal axes coincide with their geometric axes. For the Single Angle sections, the principal axes are determined and all computations related to flexural strengths are based on that. The nominal bending strength is the minimum value obtained according to the limit states of yielding, lateral-torsional buckling, flange local buckling, and web local buckling, as follows:

**Yielding**

The flexural design strength of beams, determined by the limit state of yielding is:

\[
M_p = Z F_y \leq 1.5 S F_y \quad \text{(LRFD F1-1)}
\]

**Lateral-Torsional Buckling**

**Doubly Symmetric Shapes and Channels**

For I, Channel, Box, and Rectangular shaped members bent about the major axis, the moment capacity is given by the following equation (LRFD F1):

\[
M_{n33} = \begin{cases} 
M_{p33}, & \text{if } \quad L_b \leq L_p, \\
C_b \left[ M_{p33} - (M_{p33} - M_{r33}) \left( \frac{L_b - L_p}{L_p - L_p} \right) \right] \leq M_{p33}, & \text{if } \quad L_p < L_b \leq L_r, \\
M_{cr33} \leq M_{p33}, & \text{if } \quad L_b > L_r. 
\end{cases} 
\quad \text{(LRFD F1-1, F1-2, F1-12)}
\]

where,

\[
M_{n33} = \text{Nominal major bending strength,} \\
M_{p33} = \text{Major plastic moment, } Z_{33} F_y \leq 1.5 S_{33} F_y \quad \text{(LRFD F1.1)}
\]
\[ M_{r33} = \text{Major limiting buckling moment,} \]
\[ (F_y - F_r)S_{33} \text{ for I-shapes and channels,} \quad \text{(LRFD F1-7)} \]
\[ \text{and } F_y S_{eff,33} \text{ for rectangular bars and boxes,} \quad \text{(LRFD F1-11)} \]
\[ M_{cr33} = \text{Critical elastic moment,} \]
\[ \frac{C_b \pi}{L_b} \sqrt{EI_{22} GJ + \left( \frac{\pi E}{L_b} \right)^2 I_{22} C_w} \]
\[ \text{for I-shapes and channels, and} \quad \text{(LRFD F1-13)} \]
\[ \frac{57000 C_b \sqrt{J A}}{L_b / r_{22}} \text{ for boxes and rectangular bars,} \quad \text{(LRFD F1-14)} \]

\[ L_b = \text{Laterally unbraced length, } l_{22}, \]
\[ L_p = \text{Limiting laterally unbraced length for full plastic capacity,} \]
\[ \frac{300 r_{22}}{\sqrt{M_{p33} \sqrt{J A}}} \text{ for I-shapes and channels, and} \quad \text{(LRFD F1-4)} \]
\[ \frac{3750 r_{22}}{M_{p33} \sqrt{J A}} \text{ for boxes and rectangular bars,} \quad \text{(LRFD F1-5)} \]
\[ L_r = \text{Limiting laterally unbraced length for} \]
\[ \text{inelastic lateral-torsional buckling,} \]
\[ \frac{r_{22} X_1}{F_y - F_r} \left\{ 1 + \left[ \frac{1}{2} + X_2 (F_y - F_r)^2 \right] \right\}^{\frac{1}{2}} \]
\[ \text{for I-shapes and channels, and} \quad \text{(LRFD F1-6)} \]
\[ \frac{57000 r_{22} \sqrt{J A}}{M_{r33}} \text{ for boxes and rectangular bars,} \quad \text{(LRFD F1-10)} \]

\[ X_1 = \frac{\pi}{S_{33}} \sqrt{EGJA} \quad \text{(LRFD F1-8)} \]
\[ X_2 = \frac{4 C_w}{S_{33}} \left( \frac{S_{33}}{G J} \right)^2 \quad \text{(LRFD F1-9)} \]
\[ C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3 M_A + 4 M_B + 3 M_C} \quad \text{and} \quad \text{(LRFD F1-3)} \]

\( M_{max}, M_A, M_B, \) and \( M_C \) are absolute values of maximum moment, 1/4 point, center of span and 3/4 point major moments respectively, in the member. \( C_b \) should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. The user should overwrite \( C_b \) for cantilevers. The program also defaults \( C_b \) to 1.0 if the minor unbraced length, \( l_{22}, \) of the member is re-
defined by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of $C_b$ for any member.

For I, Channel, Box, and Rectangular shaped members bent about the minor axis, the moment capacity is given by the following equation:

$$M_{n_{22}} = M_{p_{22}} = Z_{22} F_y \leq 1.5 S_{22} F_y \quad \text{(LRFD F1)}$$

For pipes and circular bars bent about any axis,

$$M_n = M_p = Z F_y \leq 1.5 S F_y . \quad \text{(LRFD F1)}$$

**T-sections and Double Angles**

For T-shapes and Double-angles the nominal major bending strength is given as,

$$M_{n_{33}} = \pi \sqrt{EI_{22} G J} \left[ B + \sqrt{1 + B^2} \right], \text{ where}$$

$$M_{n_{33}} \leq 1.5 F_y S_{33} , \text{ for positive moment, stem in tension} \quad \text{(LRFD F1.2c)}$$

$$M_{n_{33}} \leq F_y S_{33} , \text{ for negative moment, stem in compression} \quad \text{(LRFD F1.2c)}$$

$$B = \pm 2.3 \frac{d}{L_b} \sqrt{I_{22} / J} . \quad \text{(LRFD F1-16)}$$

The positive sign for $B$ applies for tension in the stem of T-sections or the outstanding legs of double angles (positive moments) and the negative sign applies for compression in stem or legs (negative moments).

For T-shapes and double angles the nominal minor bending strength is assumed as,

$$M_{n_{22}} = S_{22} F_y .$$

**Single Angles**

The nominal strengths for Single-angles are calculated based on their principal axes of bending. The nominal major bending strength for Single-angles for the limit state of lateral-torsional buckling is given as follows (LRFD SAM 5.1.3):
where,

\( M_{y, major} \) = yield moment about the major principal axis of bending, considering the possibility of yielding at the heel and both of the leg tips,

\( M_{ob} \) = elastic lateral-torsional buckling moment as calculated below.

The elastic lateral-torsional buckling moment, \( M_{ob} \), for equal-leg angles is taken as

\[
M_{ob} = C_b \frac{0.46 E b^2 t^2}{l},
\]

(LRFD SAM 5-5)

and for unequal-leg angles the \( M_{ob} \) is calculated as

\[
M_{ob} = 4.9 E C_b \frac{I_{mn}}{l^2} \left[ \sqrt{\frac{Z^2}{2}} + 0.052(\frac{l t}{r_{mn}})^2 + \beta_w \right],
\]

(LRFD SAM 5-6)

where,

\( t = \min(t_w, t_f) \),

\( l = \max(l_{22}, l_{33}) \),

\( I_{mn} \) = minor principal axis moment of inertia,

\( I_{max} \) = major principal axis moment of inertia,

\( r_{mn} \) = radius of gyration for minor principal axis,

\( \beta_w = \frac{1}{I_{max}} \int z(w^2 + z^2) dA - 2z_0 \),

(LRFD SAM 5.3.2)

\( z \) = coordinate along the major principal axis,

\( w \) = coordinate along the minor principal axis, and

\( z_0 \) = coordinate of the shear center along the major principal axis with respect to the centroid.

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$\beta_w$ is a special section property for angles. It is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles (LRFD SAM 5.3.2). However, for conservative design in ETABS, it is always taken as negative for unequal-leg angles.

**General Sections**

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

$$M_n = S F_y.$$ 

**Flange Local Buckling**

The flexural design strength, $M_n$, of Noncompact and Slender beams for the limit state of Flange Local Buckling is calculated as follows (LRFD A-F1):

For major direction bending,

$$M_n = \begin{cases} 
M_{p33}, & \text{if } \lambda \leq \lambda_p, \\
M_{p33} - (M_{p33} - M_{p33}) \left( \frac{\lambda - \lambda_p}{\lambda_y - \lambda_p} \right), & \text{if } \lambda_p < \lambda < \lambda_r, \quad (\text{A-F1-3}) \\
M_{cr33} \leq M_{p33}, & \text{if } \lambda > \lambda_r.
\end{cases}$$

and for minor direction bending,

$$M_n = \begin{cases} 
M_{p22}, & \text{if } \lambda \leq \lambda_p, \\
M_{p22} - (M_{p22} - M_{p22}) \left( \frac{\lambda - \lambda_p}{\lambda_y - \lambda_p} \right), & \text{if } \lambda_p < \lambda < \lambda_r, \quad (\text{A-F1-3}) \\
M_{cr22} \leq M_{p22}, & \text{if } \lambda > \lambda_r.
\end{cases}$$

where,

- $M_{n33}$ = Nominal major bending strength,
- $M_{n22}$ = Nominal minor bending strength,
- $M_{p33}$ = Major plastic moment, $Z_{33} F_y \leq 1.5 S_{33} F_y$,
- $M_{p22}$ = Minor plastic moment, $Z_{22} F_y \leq 1.5 S_{22} F_y$. 

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*Calculation of Nominal Strengths* 69
\[ M_{r33} = \text{Major limiting buckling moment,} \]
\[ M_{r22} = \text{Minor limiting buckling moment,} \]
\[ M_{cr33} = \text{Major buckling moment,} \]
\[ M_{cr22} = \text{Minor buckling moment,} \]
\[ \lambda = \text{Controlling slenderness parameter,} \]
\[ \lambda_p = \text{Largest value of } \lambda \text{ for which } M_s = M_p, \text{ and} \]
\[ \lambda_r = \text{Largest value of } \lambda \text{ for which buckling is inelastic.} \]

The parameters \( \lambda, \lambda_p, \lambda_r, M_{r33}, M_{r22}, M_{cr33}, \text{ and } M_{cr22} \) for flange local buckling for different types of shapes are given below:

### I Shapes, Channels

\[ \lambda = \frac{b_f}{2t_f}, \quad \text{(for I sections)} \quad \text{(LRFD B5.1, Table A-F1.1)} \]

\[ \lambda = \frac{b_f}{t_f}, \quad \text{(for Channel sections)} \quad \text{(LRFD B5.1, Table A-F1.1)} \]

\[ \lambda_p = \frac{65}{\sqrt{F_y}}, \quad \text{LRFD B5.1, Table A-F1.1)} \]

\[ \lambda_r = \left\{ \begin{array}{ll}
\frac{141}{\sqrt{F_y - F_r}} & \text{For rolled shape,} \\
\frac{162}{\sqrt{(F_y - F_r)/k_c}} & \text{For welded shape,}
\end{array} \right. \quad \text{LRFD Table A-F1.1) } \]

\[ M_{r33} = (F_y - F_r) S_{33}, \quad \text{(LRFD Table A-F1.1) } \]

\[ M_{r22} = F_y S_{22}, \quad \text{(LRFD Table A-F1.1) } \]

\[ M_{cr33} = \left\{ \begin{array}{ll}
\frac{20,000}{\lambda^2} S_{33}, & \text{For rolled shape,} \\
\frac{26,200k_c}{\lambda^2} S_{33}, & \text{For welded shape,}
\end{array} \right. \quad \text{LRFD Table A-F1.1) } \]

\[ M_{cr22} = \left\{ \begin{array}{ll}
\frac{20,000}{\lambda^2} S_{22}, & \text{For rolled shape,} \\
\frac{26,200k_c}{\lambda^2} S_{22}, & \text{For welded shape,}
\end{array} \right. \quad \text{LRFD Table A-F1.1) } \]
Boxes

\[
\lambda = \begin{cases} 
\frac{b_f - 3t_w}{t_f}, & \text{For rolled shape,} \\
\frac{b_f - 2t_w}{t_f}, & \text{For welded shape,}
\end{cases} \quad (\text{LRFD B5.1, Table A-F1.1})
\]

\[
\lambda_p = \frac{190}{\sqrt{F_y}}, \quad (\text{LRFD B5.1, Table A-F1.1})
\]

\[
\lambda_r = \frac{238}{\sqrt{F_y}}, \quad (\text{LRFD B5.1, Table A-F1.1})
\]

\[
M_{r,33} = (F_y - F_s) S_{eff,33}, \quad (\text{LRFD Table A-F1.1})
\]

\[
M_{r,22} = (F_y - F_s) S_{eff,22}, \quad (\text{LRFD Table A-F1.1})
\]

\[
M_{cr,33} = F_y S_{eff,33} \left( S_{eff,33} / S_{33} \right), \quad (\text{LRFD Table A-F1.1})
\]

\[
M_{cr,22} = F_y S_{eff,22}, \quad (\text{LRFD Table A-F1.1})
\]

\[
F_r = \begin{cases} 
10 \text{ ksi}, & \text{For rolled shape,} \\
16.5 \text{ ksi}, & \text{For welded shape.}
\end{cases} \quad (\text{LRFD A-F1})
\]

\[S_{eff,33} = \text{effective major section modulus considering slenderness, and}
\]
\[S_{eff,22} = \text{effective minor section modulus considering slenderness.}
\]

**T-sections and Double Angles**

No local buckling is considered for T sections and Double angles in ETABS. If special consideration is required, the user is expected to analyze this separately.

**Single Angles**

The nominal strengths for Single-angles are calculated based on their principal axes of bending. The nominal major and minor bending strengths for Single-angles for the limit state of flange local buckling are given as follows (LRFD SAM 5.1.1):
\[
M_n = \begin{cases} 
F_y S_c, & \text{if } b/t \leq 0.382 \sqrt{\frac{E}{F_y}}, \\
1.25 F_y S_c, \left(1 - 1.49 \left(\frac{b/t}{0.382 \sqrt{\frac{E}{F_y}}} - 1\right)\right), & \text{if } 0.382 \sqrt{\frac{E}{F_y}} < b/t \leq 0.446 \sqrt{\frac{E}{F_y}}, \\
Q F_y S_c, & \text{if } b/t > 0.446 \sqrt{\frac{E}{F_y}},
\end{cases}
\]

where,

\( S_c \) = section modulus for compression at the tip of one leg,

\( t \) = thickness of the leg under consideration,

\( b \) = length of the leg under consideration, and

\( Q \) = strength reduction factor due to local buckling.

In calculating the bending strengths for Single-angles for the limit state of flange local buckling, the capacities are calculated for both the principal axes considering the fact that either of the two tips can be under compression. The minimum capacities are considered.

**Pipe Sections**

\[
\lambda = \frac{D}{t}, \quad \lambda_p = \frac{2.070}{F_y}, \quad \lambda_s = \frac{8.970}{F_y}
\]

(LRFD Table A-F1.1)

\[
M_{r33} = M_{r22} = \left(\frac{600}{D/t} + F_y\right) S, \quad M_{r33} = M_{r22} = \left(\frac{9.570}{D/t}\right) S.
\]

(LRFD Table A-F1.1)

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Circular, Rectangular, and General Sections

No consideration of local buckling is required for solid circular shapes, rectangular plates (LRFD Table A-F1.1). No local buckling is considered in ETABS for circular, rectangular, and general shapes. If special consideration is required, the user is expected to analyze this separately.

Web Local Buckling

The flexural design strengths are considered in ETABS for only the major axis bending (LRFD Table A-F1.1).

I Shapes, Channels, and Boxes

The flexural design strength for the major axis bending, \( M_n \), of Noncompact and Slender beams for the limit state of Web Local Buckling is calculated as follows (LRFD A-F1-1, A-F1-3, A-G2-2):

\[
M_n = \begin{cases} 
M_{p33}, & \text{if } \lambda \leq \lambda_p, \\
M_{p33} - (M_{p33} - M_{r33}) \left( \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right), & \text{if } \lambda_p < \lambda \leq \lambda_r, (A-F1,A-G1) \\
S_{33} R_{PG} R_e F_{cr}, & \text{if } \lambda > \lambda_r, 
\end{cases}
\]

where,

- \( M_{n33} \) = Nominal major bending strength,
- \( M_{p33} \) = Major plastic moment, \( Z_{33} F_y \leq 1.5 S_{33} F_y \) (LRFD F1.1)
- \( M_{r33} \) = Major limiting buckling moment, \( R_e S_{33} F_y \) (LRFD Table A-F1.1)
- \( \lambda \) = Web slenderness parameter,
- \( \lambda_p \) = Largest value of \( \lambda \) for which \( M_n = M_p \),
- \( \lambda_r \) = Largest value of \( \lambda \) for which buckling is inelastic,
- \( R_{PG} \) = Plate girder bending strength reduction factor,
- \( R_e \) = Hybrid girder factor, and
- \( F_{cr} \) = Critical compression flange stress, ksi.

The web slenderness parameters are computed as follows, where the value of \( P_u \) is taken as positive for compression and zero for tension:

\[
\lambda_c = \frac{h}{t_w},
\]
The parameters $R_{PG}$, $R_e$, and $F_{cr}$ for slender web sections are calculated in ETABS as follows:

\[
\lambda_p = \begin{cases} 
\frac{640}{\sqrt{F_y}} \left(1 - 2.75 \frac{P_u}{\varphi_b P_y}\right), & \text{for } \frac{P_u}{\varphi_b P_y} \leq 0.125, \\
\frac{191}{\sqrt{F_y}} \left(2.33 - \frac{P_u}{\varphi_b P_y}\right) \geq \frac{253}{\sqrt{F_y}}, & \text{for } \frac{P_u}{\varphi_b P_y} > 0.125, 
\end{cases}
\]

\[
\lambda_v = \frac{970}{\sqrt{F_y}} \left(1 - 0.74 \frac{P_u}{\varphi_b P_y}\right).
\]

The parameters $R_{PG}$, $R_e$, and $F_{cr}$ for slender web sections are calculated in ETABS as follows:

\[
R_{PG} = 1 - \frac{a_r}{1,200 + 300a_r} \left(\frac{a_r}{h_c} \frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}}\right) \leq 1.0, \quad \text{(LRFD A-G2-3)}
\]

\[
R_e = \frac{12 + a_r \left(2m - m^3\right)}{12 + 2a_r} \leq 1.0 \quad \text{(for hybrid sections),} \quad \text{(LRFD A-G2)}
\]

\[
R_e = 1.0, \quad \text{(for non-hybrid section), where} \quad \text{(LRFD A-G2)}
\]

\[
a_r = \frac{\text{web area}}{\text{compression flange area}} \leq 1.0, \quad \text{and} \quad \text{(LRFD A-G2)}
\]

\[
m = \frac{F_y}{\min(F_{cr}, F_y)}, \quad \text{taken as 1.0.} \quad \text{(LRFD A-G2)}
\]

In the above expressions, $R_e$ is taken as 1, because currently ETABS deals with only non-hybrid girders.

The critical compression flange stress, $F_{cr}$, for slender web sections is calculated for limit states of lateral-torsional buckling and flange local buckling for the corresponding slenderness parameter $\eta$ in ETABS as follows:

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The parameters \( \eta, \eta_p, \eta_r, \) and \( C_{PG} \) for lateral-torsional buckling for slender web I, Channel and Box sections are given below:

\[
F_{cr} = \begin{cases} 
F_y, & \text{if } \eta \leq \eta_p, \\
C_{PG} \left\lfloor \frac{1}{2} \frac{\eta - \eta_p}{\eta_r - \eta_p} \right\rfloor F_y, & \text{if } \eta_p < \eta \leq \eta_r, \\
\frac{C_{PG}}{\eta^2}, & \text{if } \eta > \eta_r,
\end{cases} \tag{LRFD A-G2-4, 5, 6}
\]

The parameters \( \eta, \eta_p, \eta_r, \) and \( C_{PG} \) for flange local buckling for slender web I, Channel and Box sections are given below:

\[
\eta = \frac{L_o}{r_y}, \tag{LRFD A-G2-7}
\]

\[
\eta_p = \frac{300}{\sqrt{F_y}}, \tag{LRFD A-G2-8}
\]

\[
\eta_r = \frac{756}{\sqrt{F_y}}, \tag{LRFD A-G2-9}
\]

\[
C_{PG} = 286,000 \ C_b, \ \text{and} \tag{LRFD A-G2-10}
\]

\( r_y = \) radius of gyration of the compression flange plus one-third of the compression portion of the web, and it is taken as \( b_f/\sqrt{12} \) in ETABS.

\( C_b = \) a factor which depends on span moment. It is calculated using the equation given in page 66.

The parameters \( \eta, \eta_p, \eta_r, \) and \( C_{PG} \) for flange local buckling for slender web I, Channel and Box sections are given below:

\[
\eta = \frac{b}{t}, \tag{LRFD A-G2-11}
\]

\[
\eta_p = \frac{65}{\sqrt{F_y}}, \tag{LRFD A-G2-12}
\]

\[
\eta_r = \frac{230}{\sqrt{F_y/k_c}}, \tag{LRFD A-G2-13}
\]

\[
C_{PG} = 26,200k_c, \ \text{and} \tag{LRFD A-G2-14}
\]

\( C_b = 1. \tag{LRFD A-G2-15} \)
T-sections and Double Angles

No local buckling is considered for T-sections and Double-angles in ETABS. If special consideration is required, the user is expected to analyze this separately.

Single Angles

The nominal major and minor bending strengths for Single-angles for the limit state of web local buckling are the same as those given for flange local buckling (LRFD SAM 5.1.1). No additional check is considered in ETABS.

Pipe Sections

The nominal major and minor bending strengths for Pipe sections for the limit state of web local buckling are the same as those given for flange local buckling (LRFD Table A-F1.1). No additional check is considered in ETABS.

Circular, Rectangular, and General Sections

No web local buckling is required for solid circular shapes and rectangular plates (LRFD Table A-F1.1). No web local buckling is considered in ETABS for circular, rectangular, and general shapes. If special consideration is required, the user is expected to analyze them separately.

Shear Capacities

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

Major Axis of Bending

The nominal shear strength, $V_{n2}$, for major direction shears in I-shapes, boxes and channels is evaluated as follows:

For $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_y}}$, 

$$V_{n2} = 0.6F_yA_w,$$  

(LRFD F2-1)

for $\frac{418}{\sqrt{F_y}} < \frac{h}{t_w} \leq \frac{523}{\sqrt{F_y}}$, 

$$V_{n2} = \frac{h}{t_w} \frac{418}{\sqrt{F_y}}.$$
The nominal shear strength for all other sections is taken as:

\[ V_{n2} = 0.6 \frac{F_y}{F_y} A_w \frac{418}{\sqrt{F_y}} \frac{h}{t_w}, \quad \text{and} \quad \text{(LRFD F2-2)} \]

for \( \frac{523}{\sqrt{F_y}} < \frac{h}{t_w} \leq 260 \),

\[ V_{n2} = 132000 \frac{A_w}{[h/t_w]^2}. \quad \text{(LRFD F2-3 and A-F2-3)} \]

The nominal shear strength for minor direction shears is assumed as:

\[ V_{n3} = 0.6 F_{y}, A_{v3} \]

\section*{Minor Axis of Bending}

The nominal shear strength for minor direction shears is assumed as:

\[ V_{n3} = 0.6 F_{y}, A_{v3} \]

\section*{Calculation of Capacity Ratios}

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

During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.

\section*{Axial and Bending Stresses}

The interaction ratio is determined based on the ratio \( \frac{P}{P_a} \) (LRFD SAM 6). If \( P_a \) is tensile, \( P_a \) is the nominal axial tensile strength and \( \varphi = \varphi_y = 0.9 \); and if \( P_a \) is compressive, \( P_a \) is the nominal axial compressive strength and \( \varphi = \varphi_c = 0.85 \), except for angle sections \( \varphi = \varphi_c = 0.90 \) (LRFD SAM 6). In addition, the resistance factor for bending, \( \varphi_b = 0.9 \).
For $P \geq 0.2 \phi_{P_n}$, the capacity ratio is given as

$$\frac{P}{\phi_{P_n}} + \frac{8}{9} \left( \frac{M_{u33}}{\phi_b M_{n33}} + \frac{M_{u22}}{\phi_b M_{n22}} \right).$$

(LRFD H1-1a, SAM 6-1a)

For $P < 0.2 \phi_{P_n}$, the capacity ratio is given as

$$\frac{P}{2\phi_{P_n}} + \left( \frac{M_{u33}}{\phi_b M_{n33}} + \frac{M_{u22}}{\phi_b M_{n22}} \right).$$

(LRFD H1-1b, SAM 6-1a)

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

For Single-angle sections, the combined stress ratio is calculated based on the properties about the principal axis (LRFD SAM 5.3, 6). For I, Box, Channel, T, Double angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes are determined in ETABS. For general sections it is assumed that the section properties are given in terms of the principal directions.

Shear Stresses

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

$$\frac{V_{u2}}{\phi_{V_n} V_{n2}}$$, and

$$\frac{V_{u3}}{\phi_{V_n} V_{n3}},$$

where $\phi_{V_n} = 0.9$.

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

Check/Design for UBC-ASD97


Chapter 22, Division III, of UBC adopted the American Institute of Steel Construction’s Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design, June 1, 1989 with Commentary (AISC 1989a), which has been implemented in the AISC-ASD89 code in ETABS. The ETABS implementation of AISC-ASD89 is described in Chapter III “Design/Check for AISC-ASD89” of this manual. The current chapter frequently refers to Chapter III. It is suggested that the user read Chapter III before continuing to read this chapter.

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

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Various notations used in this chapter are described in Table III-1.

When using the UBC-ASD97 option, the following Framing Systems are recognized (UBC 1627, 2213):

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

By default the frame type is taken as Special Moment-Resisting Frame (SMRF) in the program. However, the frame type can be overwritten in the Preference form to change the default and in the Overwrites form on a member by member basis. If any member is assigned with a frame type, the change of the frame type in the Preference will not modify the frame type of the individual member for which it is assigned.

When using the UBC-ASD97 option, a frame is assigned to one of the following five Seismic Zones (UBC 2213, 2214):

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

By default the Seismic Zone is taken as Zone 4 in the program. However, the frame type can be overwritten in the Preference form to change the default.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

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

The design load combinations are the various combinations of the load cases for which the structural members and joints needs to be designed or checked. For the UBC-ASD97 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, then the following load combinations may have to be defined (UBC 1612.3):

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

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

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

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

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

- 1.0 DL + 0.7 LL ± Ω₀ EL (UBC 2213.5.1.1)
- 0.85 DL ± Ω₀ EL (UBC 2213.5.1.2)

where, Ω₀ is the seismic force amplification factor which is required to account for structural overstrength. The default value of Ω₀ is taken as 2.8 in the program. However, Ω₀ can be overwritten in the Preference form to change the default and in the Overwrites form on a member by member basis. If any member is assigned a
value for $\Omega_0$, the change of $\Omega_0$ in the Preference form will not modify the $\Omega_0$ of the individual member for which $\Omega_0$ is assigned. The guideline for selecting a reasonable value can be found in UBC 1630.3.1 and UBC Table 16-N. There are other similar special loading combinations which are described latter in this chapter.

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

### Member Design

A member is recognized in the program as either a beam, column, or brace. In the calculation of the axial and bending stress ratios, first, for each station along the length of the member, the actual stresses are calculated for each load combination. Then the corresponding allowable stresses are calculated. Then, the stress ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling stress ratio is then obtained, along with the associated station and load combination. A stress ratio greater than 1.0 indicates an overstress. Similarly, a shear capacity ratio is also calculated separately. In addition, if required for seismic design, members are checked for special loading combinations, $l/r$ ratio, section slenderness ratio, etc.

### Classification of Sections

The allowable stresses for axial compression and flexure depend upon the classification of sections. The sections are classified in UBC-ASD97 as either Compact, Noncompact, Slender or Too Slender in the same way as described in section “Classification of Sections” of Chapter III with some exceptions as described in the next paragraph. ETABS classifies the individual sections according to the limiting width/thickness ratios given in Table III-2 (UBC 2208, 2212, 2213, ASD B5.1, F3.1, F5, G1, A-B5-2). The definition of the section properties required in this table is given in Figure III-1 and Table III-1 of Chapter III.

In general the design sections need not necessarily be Compact to satisfy UBC-ASD97 codes (UBC 2213.4.2). However, for certain special seismic cases they have to be Compact and have to satisfy special slenderness requirements. See subsection “Seismic Requirements” later in this chapter. The sections which do sat-
<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Width-Thickness Ratio $\lambda$</th>
<th>SEISMIC (Special requirements in seismic design $\lambda_p$)</th>
<th>Section References</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>$b_j/2t_j$ (beam)</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$</td>
<td>UBC 2213.7.3 (SMRF) UBC 2213.10.2 (EBF)</td>
</tr>
<tr>
<td></td>
<td>$b_j/2t_j$ (column)</td>
<td>$\leq \frac{110}{\sqrt{F_y}}$</td>
<td>UBC 2213.7.3 (SMRF), UBC 2213.9.5 (SCBF)</td>
</tr>
<tr>
<td>BOX</td>
<td>$b/t_i$ and $h_i/t_o$ (column)</td>
<td>$\leq \frac{110}{\sqrt{F_y}}$</td>
<td>UBC 2213.8.2.5 (BF), UBC 2213.9.2.4 (SCBF)</td>
</tr>
<tr>
<td>ANGLE</td>
<td>$b/t$ (brace)</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$</td>
<td>UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)</td>
</tr>
<tr>
<td>DOUBLE-ANGLE</td>
<td>$b/t$ (brace)</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$</td>
<td>UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)</td>
</tr>
<tr>
<td>PIPE</td>
<td>$D/t$ (brace)</td>
<td>$\leq \frac{1300}{F_y}$</td>
<td>UBC 2213.8.2.5 (BF) UBC 2213.9.2.4 (SCBF)</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$b_j/t_j$ and $h_j/t_o$ (brace)</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$b_j/2t_j$ (brace)</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
</tbody>
</table>

**Table V-1**

*Limiting Width-Thickness Ratios for Classification of Sections when Special Seismic Conditions Apply as per UBC-ASD*
isfy these additional requirements are classified and reported as “SEISMIC” in ETABS. These special requirements for classifying the sections as “SEISMIC” in ETABS ( “Compact” in UBC) are given in Table V-1 (UBC 2213.7.3, 2213.8.2.5, 2213.9.2.4, 2213.9.5, 2213.10.2). If these criteria are not satisfied, when the code requires them to be satisfied, the user must modify the section property. In this case ETABS gives a warning message in the output file.

**Calculation of Stresses**

The axial, flexural, and shear stresses at each of the previously defined stations for each load combination in UBC-ASD97 are calculated in the same way as described in section “Calculation of Stresses” of Chapter III without any exception (UBC 2208, ASD A-B5.2d). For nonslender sections, the stresses are based on the gross cross-sectional areas (ASD A-B5.2c), for slender sections the stresses are based on effective section properties (ASD A-B5.2c), and for Single-angle sections the stresses are based on the principal properties of the sections (ASD SAM 6.1.5).

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

The shear stresses for Single-angle sections are calculated for directions along the geometric axes. For all other sections the shear stresses are calculated along the geometric/principle axes.

**Calculation of Allowable Stresses**

The allowable stresses in compression, tension, bending, and shear for Compact, Noncompact, and Slender sections according to the UBC-ASD97 are calculated in the same way as described in section “Calculation of Allowable Stresses” of Chapter III without any exception (UBC 2208, ASD A-B5.2d). The allowable stresses for Seismic sections are calculated in the same way as for Compact sections.

The allowable flexural stresses for all shapes of sections are calculated based on their principal axes of bending. For the I, Box, Channel, Circular, Pipe, T, Double-angle and Rectangular sections, the principal axes coincide with their geometric axes. For the Angle sections, the principal axes are determined and all computations related to flexural stresses are based on that.
The allowable shear stress is calculated along the geometric axes for all sections. For I, Box, Channel, T, Double angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes do not coincide with the geometric axes.

All limitations and warnings related to allowable stress calculation in AISC-ASD89 also apply in this code.

*If the user specifies nonzero allowable stresses for one or more elements in the ETABS “Allowable Stress Overwrites” form, these values will override the above mentioned calculated values for those elements. The specified allowable stresses should be based on the principal axes of bending.*

**Calculation of Stress Ratios**

The stress ratios in UBC-ASD97 are calculated in the same way as described in section “Calculation of Stress Ratios” of Chapter III with some modifications as described below.

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

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

**Axial and Bending Stresses**

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

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

\[
\frac{f_a}{F_a} + \frac{C_{m33} f_{b33}}{F_{b33}} + \frac{C_{m22} f_{b22}}{F_{b22}}, \quad \text{and (ASD H1-1, SAM 6.1)}
\]
\[
\frac{f_a}{Q(0.60 F_a)} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{where} \quad \text{(ASD H1-2, SAM 6.1)}
\]

\(f_a, f_{b33}, f_{b22}, F_a, F_{b33}, F_{b22}, \) and \(F_a'\) are defined earlier in Chapter III. A factor of 4/3 is NOT applied on \(F_a'\) and 0.6\(F_a\) if the load combination includes any wind load or seismic load (UBC 1612.3.1).

\(C_{m33}\) and \(C_{m22}\) are coefficients representing distribution of moment along the member length. They are calculated in the same way as in Chapter III.

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

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

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

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

\[
\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{and} \quad \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{(ASD H2-1, SAM 6.2)}
\]

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

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

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

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

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

Shear Stresses

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

\[ \frac{f_{v2}}{F_v} \text{, and} \]

\[ \frac{f_{v3}}{F_v} . \]

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

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

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

Ordinary Moment Frames

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

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

\[
\begin{align*}
1.0 \text{ DL} + 0.7 \text{ LL} & \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.5.1.1)} \\
0.85 \text{ DL} & \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.5.1.2)}
\end{align*}
\]

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

Special Moment-Resisting Frames

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

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

\[
\begin{align*}
1.0 \text{ DL} + 0.7 \text{ LL} & \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.5.1.1)} \\
0.85 \text{ DL} & \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.5.1.2)}
\end{align*}
\]

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

- In Seismic Zones 3 and 4, the I-shaped beams, I-shaped columns, and Box shaped columns are additionally checked for compactness criteria as described in Table V-1 (UBC 2213.7.3). Compact I-shaped beam sections are additionally checked for $b_t / 2t_f$ to be less than $52 \sqrt{F_y}$. Compact I-shaped column sections are additionally checked for $b_t / 2t_f$ to be less than the numbers given for plastic sections in Table V-1. Compact box shaped column sections are additionally checked for $b/t_f$ and $d/t_w$ to be less than $110 \sqrt{F_y}$. If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

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

**Braced Frames**

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

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

\[
\begin{align*}
1.0 \text{DL} + 0.7 \text{LL} & \pm \Omega_0 \text{EL} \quad \text{(UBC 2213.5.1.1)} \\
0.85 \text{DL} + \Omega_0 \text{EL} & \quad \text{(UBC 2213.5.1.2)}
\end{align*}
\]

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

- In Seismic Zones 3 and 4, the program checks the laterally unsupported length of beams to be less than $96 r_y$. If the check is not satisfied, it is noted in the output (UBC 2213.8.1, 2213.7.8).
• In Seismic Zones 3 and 4, the maximum \( l/r \) ratio of the braces is checked not to exceed \( 720 / \sqrt{F_y} \). If this check is not met, it is noted in the output (UBC 2213.8.2.1).

• In Seismic Zones 3 and 4, the Angle, Double-angle, Box, and Pipe shaped braces are additionally checked for compactness criteria as described in Table V-1 (UBC 2213.8.2.5). For angles and double-angles \( b/t \) is limited to \( 52 / \sqrt{F_y} \), for box sections \( b/t_f \) and \( d/t_w \) is limited to \( 110 / \sqrt{F_y} \), for pipe sections \( D/t \) is limited to \( 1300 / F_y \). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

• In Seismic Zones 3 and 4, the allowable compressive stress for braces is reduced by a factor, \( B \), where

\[
B = \frac{1}{1 + \frac{Kl/r}{2C_c}}
\]

(UBC 2213.8.2.2)

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

\[
B \geq 0.8
\]

(UBC 2214.6.2.1)

• In Seismic Zones 3 and 4, Chevron braces are designed for 1.5 times the specified loading combinations (UBC 2213.8.4.1).

**Eccentrically Braced Frames**

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

• In all Seismic Zones except Zone 0, the I-shaped beam sections are additionally checked for compactness criteria as described in Table V-1. Compact I-shaped beam sections are additionally checked for \( b_f / 2t_f \) to be less than \( 52 / \sqrt{F_y} \). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property (UBC 2213.10.2). Other sections meeting this criteria are also reported as SEISMIC.
In all Seismic Zones except Zone 0, the link beam strength in shear 
\( V_s = 0.55 F_y d_t_w \) and moment \( M_z = Z F_y \) are calculated. If \( V_s \leq 2.0 M_z / e \), the 
link beam strength is assumed to be governed by shear and is so reported. If the 
above condition is not satisfied, the link beam strength is assumed to be gov-
erned by flexure and is so reported. When link beam strength is governed by 
shear, the axial and flexural properties (area, \( A \) and section modulus, \( S \)) for use 
in the interaction equations are calculated based on the beam flanges only 
(UBC 2213.10.3).

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

\[
e \leq 1.6 \frac{M_p}{V_p} \tag{UBC 2213.10.12}
\]

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

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

\[
\theta \leq 0.090, \quad \text{where link beam clear length, } e \leq 1.6 \frac{M_z}{V_s}, \\
\theta \leq 0.030, \quad \text{where link beam clear length, } e \geq 3.0 \frac{M_z}{V_s}, \text{ and} \\
\theta \leq \text{value interpolated between 0.090 and 0.030 as the link beam clear} 
\text{length varies from 1.6 } \frac{M_z}{V_s} \text{ to } 3.0 \frac{M_z}{V_s}.
\]

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

In all Seismic Zones except Zone 0, the brace strength is checked to be at least 
1.5 times the axial force corresponding to the controlling link beam strength 
(UBC 2213.10.13). The controlling link beam strength is either the shear 
strength, \( V_s \), as \( V_s = 0.55 F_y d_t_w \), or the reduced flexural strength, \( M_{rs} \), whichever 
produces the lower brace force. The value of \( M_{rs} \) is taken as 
\( M_{rs} = Z(F_y - f_a) \) (UBC 2213.10.3), where \( f_a \) is the lower of the axial stress 
in the link beam corresponding to yielding of the link beam web in shear or the 
link beam flanges in flexure. The correspondence between brace force and link 
beam force is obtained from the associated load cases, whichever has the high-
est link beam force of interest.
In all Seismic Zones except Zone 0, the column is checked not to become inelastic for gravity loads plus 1.25 times the column forces corresponding to the controlling link beam strength (UBC 2213.10.14). The controlling link beam strength and the corresponding forces are as obtained by the process described above. If this condition governs, the column axial allowable stress in compression is taken to be $1.7F_a$ instead of $F_a$ and the column axial allowable stress in tension is taken to be $F_y$ instead of $F_a$.

Figure V-1
Eccentrically Braced Frame Configurations

- In all Seismic Zones except Zone 0, the column is checked not to become inelastic for gravity loads plus 1.25 times the column forces corresponding to the controlling link beam strength (UBC 2213.10.14). The controlling link beam strength and the corresponding forces are as obtained by the process described above. If this condition governs, the column axial allowable stress in compression is taken to be $1.7F_a$ instead of $F_a$ and the column axial allowable stress in tension is taken to be $F_y$ instead of $F_a$. 
In all Seismic Zones except Zone 0, axial forces in the beams are included in checking of the beams (UBC 2211.10.17). The user is reminded that using a rigid diaphragm model will result in zero axial forces in the beams. The user must disconnect some of the column lines from the diaphragm to allow beams to carry axial loads. It is recommended that only one column line per eccentrically braced frame be connected to the rigid diaphragm or a flexible diaphragm model be used.

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

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

### Special Concentrically Braced Frames

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

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

\[
\begin{align*}
1.0 \text{DL} + 0.7 \text{LL} & \pm \Omega_o \, \text{EL} \\
0.85 \text{DL} & \pm \Omega_o \, \text{EL}
\end{align*}
\]

(UBC 2213.5.1.1) (UBC 2213.5.1.2)

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

- In all Seismic Zones except Zone 0, the I-shaped and Box shaped columns are additionally checked for compactness criteria as described in Table V-1. Compact I-shaped column sections are additionally checked for \( b_f/2t_f \) to be less than the numbers given for plastic sections in Table V-1. Compact box shaped column sections are additionally checked for \( b/t_f \) and \( d/t_w \) to be less than \( 110/\sqrt{F_y} \). If this criteria is satisfied the section is reported as SEISMIC as

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described earlier under section classifications. If this criteria is not satisfied the user must modify the section property (UBC 2213.9.5, 2213.7.3).

- In all Seismic Zones except Zone 0, bracing members are checked to be compact and are so reported. The Angle, Box, and Pipe sections used as braces are additionally checked for compactness criteria as described in Table V-1. For angles $b/t_i$ is limited to $52/\sqrt{F_y}$, for box sections $b/t_j$ and $d/t_w$ is limited to $110/\sqrt{F_y}$, for pipe sections $D/t$ is limited to $1300/F_y$. If this criteria is satisfied the section is reported as SEISMIC. If this criteria is not satisfied the user must modify the section property (UBC 2213.9.2.4).

- In all Seismic Zones except Zone 0, the maximum $K/I/r$ ratio of the braces is checked not to exceed $1000/\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2213.9.2.4).

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

\[
1.2 \text{DL} + 0.5 \text{LL} \pm P_b
\]
\[
0.9 \text{DL} \pm P_b
\]

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

### Joint Design

When using UBC-ASD97 design code, the structural joints are checked and/or designed for the following:

- Check for the requirement of continuity plate and determination of its area
- Check for the requirement of doubler plate and determination of its thickness
- Check for the ratio of beam flexural strength to column flexural strength
- Reporting the beam connection shear
- Reporting the brace connection force

94 Joint Design
Design of Continuity Plates

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

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

To achieve a beam/column moment connection, continuity plates such as shown in Figure II-4 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.

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

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

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

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

- The depth of the column clear of the fillets, i.e. \(d_c - 2k_c\), is less than or equal to:
The thickness of the column flange, $t_{fc}$, is greater than or equal to:

$$0.4 \frac{P_{bf}}{F_{yc}}$$

where

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

$f_b$ is the bending stress calculated from the larger of $5/3$ of loading combinations with gravity loads only and $4/3$ of the loading combinations with lateral loads (ASD K1.2). For special seismic design, $f_b$ is specified to be beam flange strength.

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

- The thickness of the stiffeners is at least $0.5 t_{fb}$, or

$$t_{cp}^{\text{min}} = 0.5 \ t_{fb}$$

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

$$b_{cp}^{\text{min}} = 2 \left( b_{fb} - \frac{t_{wc}}{2} \right)$$

- So that the minimum area is given by:

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

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

Where

- $A_{bf}$ = Area of beam flange
- $A_{cp}$ = Required continuity plate area
- $F_{yb}$ = Yield stress of beam material
- $F_{yc}$ = Yield stress of the column and continuity plate material
The special seismic requirements additionally checked by the program are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2213 for frames in Seismic Zones 3 and 4 and on UBC Section 2214 for frames in Seismic Zones 1 and 2 (UBC 2204.2, 2205.2, 2213, 2214). No special requirement is checked for frames in Seismic Zone 0.

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

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

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

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

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

  $$P_{bf} = f_{sb} A_{bf}$$  
  (UBC 2213.10.12, 2213.10.19)
Design of Doubler Plates

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

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

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending upon the loading and the geometry of the steel beams that frame into the column, either along the column major direction, or at an angle so that the beams have components along the column major direction. See Figure II-5. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames only. No check is made for braced frames.

The shear force in the panel zone, is given by

\[ V_p = P - V_c, \]  

or

\[ V_p = \sum_{n=1}^{m} \frac{M_{bn} \cos \theta_n}{d_n - t_{f_n}} - V_c \]

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

\[ t_r = \frac{V_p}{F_v d_c} \geq \frac{h}{\frac{380}{\sqrt{F_{yc}}} \sqrt{F_{yc}}} \] (ASD F4)

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

\[ t_{dp} = t_r - t_{we} \], where,

\[ F_v = 0.40 F_{yc} \] (ASD F4)

\[ F_{yc} = \text{Yield stress of the column and doubler plate material} \]

\[ t_r = \text{Required column web thickness} \]
\[ t_{dp} = \text{Required doubler plate thickness} \]
\[ t_{fn} = \text{Thickness of flange of the } n\text{-th beam connecting to column} \]
\[ t_{wc} = \text{Column web thickness} \]
\[ V_p = \text{Panel zone shear} \]
\[ V_c = \text{Column shear in column above} \]
\[ P = \text{Beam flange forces} \]
\[ n_b = \text{Number of beams connecting to column} \]
\[ d_n = \text{Depth of } n\text{-th beam connecting to column} \]
\[ h = d_n - 2t_{jc} \text{ if welded, } d_n - 2k_c \text{ if rolled,} \]
\[ \theta_n = \text{Angle between } n\text{-th beam and column major direction} \]
\[ d_c = \text{Depth of column} \]
\[ M_{bn} = \text{Calculated factored beam moment from the corresponding loading combination} \]

The largest calculated value of \( V_p \), calculated for any of the load combinations based upon the factored beam moments is used to calculate doubler plate areas.

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

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

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

\[
V_p = 0.55 \frac{F_{yc}}{d_c} d_c t_r \left(1 + \frac{3 b_c t_{cf}^2}{d_b d_c t_r}\right) \quad \text{(UBC 2213.7.2.1)}
\]

giving the required panel zone thickness as

\[
t_r = \frac{V_p}{0.55 \frac{F_{yc}}{d_c}} - \frac{3 b_c t_{cf}^2}{d_b d_c} \geq \frac{h}{380 \sqrt{F_{yc}}} \quad \text{(UBC 2213.7.2.1, ASD F4)}
\]
and the required doubler plate thickness as

\[ t_{dp} = t_r - t_{we} \]

where

- \( b_c \) = width of column flange,
- \( h \) = \( d_c - 2t_{fc} \) if welded, \( d_c - 2k_c \) if rolled,
- \( t_{cf} \) = thickness of column flange, and
- \( d_b \) = depth of deepest beam framing into the major direction of the column.

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

\[ t_{we} \geq \frac{(d_c - 2t_{fc}) + (d_b - 2t_{fb})}{90} \]  

(UBC 2213.7.2.2)

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

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

### Beam/Column Plastic Moment Capacity Ratio

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

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

\[ R_{maj} = \frac{\sum_{n=1}^{n_b} M_{pbn} \cos \theta_n}{M_{pax} + M_{pchx}} \]  

(UBC 2213.7.5)
For the minor direction of any column the beam to column strength ratio is obtained as

\[ R_{\text{min}} = \frac{\sum_{n=1}^{n_b} M_{\text{pbn}} \sin \theta_n}{M_{\text{pcay}} + M_{\text{pcbby}}} \]  

(UBC 2213.7.5)

where,

- \( R_{\text{maj, min}} \) = Plastic moment capacity ratios, in the major and minor directions of the column, respectively,
- \( M_{\text{pbn}} \) = Plastic moment capacity of \( n \)-th beam connecting to column,
- \( \theta_n \) = Angle between the \( n \)-th beam and the column major direction,
- \( M_{\text{pcax, y}} \) = Major and minor plastic moment capacities, reduced for axial force effects, of column above story level.
- \( M_{\text{pcbax, y}} \) = Major and minor plastic moment capacities, reduced for axial force effects, of column below story level, and
- \( n_b \) = Number of beams connecting to the column.

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

\[ M_{\text{pc}} = Z_c (F_{\text{yc}} - f_a) \]  

(UBC 2213.7.5)

where,

- \( Z_c \) = Plastic modulus of column,
- \( F_{\text{yc}} \) = Yield stress of column material, and
- \( f_a \) = Maximum axial stress in the column.

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

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

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

- In all Seismic Zones except Zone 0, for Ordinary Moment Frames, the beam connection shears reported are the maximum of the specified loading combinations and the following additional loading combination (UBC 2213.6.2, 2214.4.2):
  \[ 1.0 \, DL + 1.0 \, LL \, \pm \, \Omega_0 \, EL \] (UBC 2213.6.2, 2214.4.2)

- In all Seismic Zones except Zone 0, for Special Moment-Resisting Frames, the beam connection shears that are reported allow for the development of the full plastic moment capacity of the beam (UBC 2213.7.1, 2214.5.1.1). Thus:
  \[ V = \frac{C \, M_{pb}}{L} + V_{DL,LL} \] (UBC 2213.7.1.1, 2214.5.1.1)

  where,
  \[ V \quad \text{Shear force corresponding to END I or END J of beam,} \]
  \[ C \quad \begin{array}{l} = 0 \text{ if beam ends are pinned, or for cantilever beam,} \\ = 1 \text{ if one end of the beam is pinned,} \\ = 2 \text{ if no ends of the beam are pinned,} \end{array} \]
  \[ M_{pb} \quad \text{Plastic moment capacity of the beam,} \, Z F_y, \]
  \[ L \quad \text{Clear length of the beam, and} \]
  \[ V_{DL,LL} \quad \text{Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the dead load and live load combinations only.} \]

- In all Seismic Zones except Zone 0, for Eccentrically Braced Frames, the beam connection shears reported are the maximum of the specified loading combinations and the following additional loading combination:
  \[ 1.0 \, DL + 1.0 \, LL \, \pm \, \Omega_0 \, EL \]
Evaluation of Brace Connection Forces

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

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

- In all Seismic Zones except Zone 0, for ordinary Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace \( F_y \) and the following special loading combination (UBC 2213.8.3.1, 2214.6.3.1):

  \[
  1.0 \text{ DL} + 1.0 \text{ LL} \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.8.3.1, 2214.6.3.1)}
  \]

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

  \[
  1.0 \text{ DL} + 1.0 \text{ LL} \pm \Omega_0 \text{ EL} \quad \text{(UBC 2213.9.3.1, 2214.7)}
  \]

- In all Seismic Zones except Zone 0, for Eccentrically Braced Frames, the bracing connection force is reported as at least the brace strength in compression which is computed as \( 1.7 F_y A \) (UBC 2213.10.6, 2214.8). \( 1.7 F_y A \) is limited not to exceed \( F_y A \).
Check/Design for UBC-LRFD97

This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the UBC-LRFD97 design code. The UBC-LRFD97 design code in ETABS implements the International Conference of Building Officials’ 1997 Uniform Building Code: Volume 2: Structural Engineering Design Provisions, Chapter 22, Division II, “Design Standard for Load and Resistance Factor Design Specification for Structural Steel Buildings” (ICBO 1997).

Chapter 22, Division III, of UBC adopted the American Institute of Steel Construction’s Load and Resistance Factor Design Specification for Structural Steel Buildings (AISC 1993), which has been implemented in the AISC-LRFD93 code in ETABS. The ETABS implementation of UBC-LRFD97 is described in Chapter IV “Check/Design for AISC-LRFD93” of this manual. The current chapter frequently refers to Chapter IV. It is suggested that the user read Chapter IV before continuing to read this chapter.

For referring to pertinent sections and equations of the UBC code, a unique prefix “UBC” is assigned. For referring to pertinent sections and equations of the UBC-LRFD code, a unique prefix “LRFD” is assigned. However, all references to the “Specifications for Load and Resistance Factored Design of Single-Angle Members” (AISC 1994) carry the prefix of “LRFD SAM”. Moreover, all sections of the “Seismic Provisions for Structural Steel Buildings June 15, 1992” (AISC
1994) are referred to as Section 2211.4 of the UBC code. In this manual, all sections and subsections referenced by “UBC 2211.4” or “UBC 2211.4.x” refer to the LRFD Seismic Provisions after UBC amendments through UBC Section 2210. Various notations used in this chapter are described in Table IV-1.

When using the UBC-LRFD97 option, the following Framing Systems are recognized (UBC 1627, 2210):

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

By default the frame type is taken as Special Moment-Resisting Frame (SMRF) in the program. However, the frame type can be overwritten in the Preference form to change the default and in the Overwrites form on a member by member basis. If any member is assigned with a frame type, the change of the frame type in the Preference will not modify the frame type of the individual member for which it is assigned.

When using the UBC-LRFD97 option, a frame is assigned to one of the following five Seismic Zones (UBC 2210):

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

By default the Seismic Zone is taken as Zone 4 in the program. However, the frame type can be overwritten in the Preference form to change the default.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

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

The design load combinations are the various combinations of the load cases for which the structural members and joints needs to be designed or checked. For the UBC-LRFD97 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, then the following load combinations may have to be defined (UBC 2204.1, 2206, 2207.3, 2210.3, 1612.2.1):

- $1.4 \text{DL}$
- $1.2 \text{DL} + 1.4 \text{LL}$
- $1.2 \text{DL} \pm 0.8 \text{WL}$
- $0.9 \text{DL} \pm 1.3 \text{WL}$
- $1.2 \text{DL} + 0.5 \text{LL} \pm 1.3 \text{WL}$
- $1.2 \text{DL} \pm 1.0 \text{EL}$
- $0.9 \text{DL} \pm 1.0 \text{EL}$
- $1.2 \text{DL} + 0.5 \text{LL} \pm 1.0 \text{EL}$

These are also the default design load combinations in ETABS whenever the UBC-LRFD97 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

When using the UBC-LRFD97 code, ETABS design assumes that a P-Δ analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-Δ analysis be done at the factored load level of $1.2 \text{DL} + 0.5 \text{LL}$ (White and Hajjar 1991).

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

- $0.9 \text{DL} \pm \Omega_0 \text{EL}$
- $1.2 \text{DL} + 0.5 \text{LL} \pm \Omega_0 \text{EL}$

where, $\Omega_0$ is the seismic force amplification factor which is required to account for structural overstrength. The default value of $\Omega_0$ is taken as 2.8 in the program.
However, $\Omega_0$ can be overwritten in the Preference form to change the default and in the Overwrites form on a member by member basis. If any member is assigned a value for $\Omega_0$, the change of $\Omega_0$ in the Preference form will not modify the $\Omega_0$ of the individual member for which $\Omega_0$ is assigned. The guideline for selecting a reasonable value can be found in UBC 1630.3.1 and UBC Table 16-N. There are other similar special loading combinations which are described latter in this chapter.

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

**Member Design**

A member is recognized in the program as either a beam, column, or brace. In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this chapter. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates over-stress. Similarly, a shear capacity ratio is also calculated separately.

**Classification of Sections**

The nominal strengths for axial compression and flexure are dependent on the classification of the section as Compact, Noncompact, Slender or Too Slender. The sections are classified in UBC-LRFD97 as either Compact, Noncompact, Slender or Too Slender in the same way as described in section “Classification of Sections” of Chapter IV with some exceptions as described in the next paragraph. ETABS classifies individual members according to the limiting width/thickness ratios given in Table IV-2 and Table IV-3 (UBC 2204.1, 2205, 2206, and 2210; LRFD B5.1, A-G1, and Table A-F1.1). The definition of the section properties required in these tables is given in Figure IV-1 and Table IV-1 of Chapter IV. The same limitations apply.

In general the design sections need not necessarily be Compact to satisfy UBC-LRFD97 codes (UBC 2213.2). However, for certain special seismic cases
<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Width-Thickness Ratio $\lambda$</th>
<th>SEISMIC (Special requirements in seismic design) $\left(\lambda_p, \varphi_p\right)$</th>
<th>Section References</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>$\frac{b}{2t}$ or $\frac{h}{t}$</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$</td>
<td>UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)</td>
</tr>
<tr>
<td></td>
<td>$\frac{b}{2t}$ or $\frac{h}{t}$</td>
<td>$\leq \frac{520}{\left(1 - 1.54 \frac{P}{P_u}\right) \varphi P_y}$ For $P_y / \varphi P_y &gt; 0.125$ $\leq \left{ \frac{191}{\sqrt{F_y}} \left(2.33 \cdot \frac{P}{P_u} \right) \right} \geq \left(253 \right)$</td>
<td>UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)</td>
</tr>
<tr>
<td>BOX</td>
<td>$\frac{b}{t}$ or $\frac{h}{t}$</td>
<td>$\leq 110 / \sqrt{F_y}$ (Beam and column in SMRF, column in SCBF, Braces in BF)</td>
<td>UBC 2210.8 (SMRF) UBC 2210.10.g (SCBF) UBC 2211.4.9.2.d (BF)</td>
</tr>
<tr>
<td></td>
<td>$\frac{b}{t}$ or $\frac{h}{t}$</td>
<td>$\leq 100 / \sqrt{F_y}$ (Braces in SCBF)</td>
<td>UBC 2210.10.c (SCBF)</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$\frac{b}{t}$ or $\frac{h}{t}$</td>
<td>Same as I-Shapes Same as I-Shapes</td>
<td>UBC 2211.4.8.4.b (SMRF) UBC 2211.4 Table 8-1 (SMRF)</td>
</tr>
<tr>
<td>ANGLE</td>
<td>$\frac{b}{t}$</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$ (Braces in SCBF)</td>
<td>UBC 2210.10.c (SCBF) UBC 2211.4.9.2.d (SCBF)</td>
</tr>
<tr>
<td>DOUBLE-ANGLE</td>
<td>$\frac{b}{t}$</td>
<td>$\leq \frac{52}{\sqrt{F_y}}$ (Braces in SCBF)</td>
<td>UBC 2210.10.c (SCBF) UBC 2211.4.9.2.d (SCBF)</td>
</tr>
<tr>
<td>PIPE</td>
<td>$\frac{D}{t}$</td>
<td>$\leq 1300 / F_y$</td>
<td>UBC 2210.10.c (Braces in SCBF) UBC 2211.4.9.2.d (Braces in BF)</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$\frac{b}{2t}$ or $\frac{d}{t}$</td>
<td>No special requirement No special requirement</td>
<td></td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td>No special requirement</td>
<td></td>
</tr>
</tbody>
</table>

Table VI-1

Limiting Width-Thickness Ratios for Classification of Sections when Special Seismic Conditions Apply as per UBC-LRFD
they have to be Compact and have to satisfy special slenderness requirements. See subsection “Seismic Requirements” later in this section. The sections which do satisfy these additional requirements are classified and reported as “SEISMIC” in ETABS. These special requirements for classifying the sections as “SEISMIC” in ETABS (“Compact” in UBC) are given in Table VI-1 (UBC 2210.8, 2210.10c, 2211.4.8.4.b, 2211.9.2.d, 2210.10g, 2211.4.10.6.e). If these criteria are not satisfied, when the code requires them to be satisfied, the user must modify the section property. In this case ETABS gives a warning message in the output file.

**Calculation of Factored Forces**

The factored member loads that are calculated for each load combination are $P_u$, $M_{u33}$, $M_{u22}$, $V_{u2}$ and $V_{u3}$ corresponding to factored values of the axial load, the major moment, the minor moment, the major direction shear force and the minor direction shear force, respectively. These factored loads are calculated at each of the previously defined stations for each load combination. They are calculated in the same way as described in section “Calculation of Factored Forces” of Chapter IV without any exception (UBC 2204.1 2205.2, 2205.3, 2206, 2210).

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

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

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

$$M_u = B_1 M_{u1} + B_2 M_{u2},$$

(LRFD C1-1, SAM 6)

where $M_{u1}$, $M_{u2}$, $B_1$, and $B_2$ are defined in Chapter IV. $B_1$ and $B_2$ are moment magnification factors. $B_1$ is calculated in the same way as in Chapter IV. Similarly to AISC-LRFD93, ETABS design assumes the analysis includes P-Δ effects in this code too, therefore $B_2$ is taken as unity for bending in both directions. If the program assumptions are not satisfactory for a particular structural model or member,
the user has a choice of explicitly specifying the values of $B_1$ and $B_2$ for any member.

When using the UBC-LRFD97 code, ETABS design assumes that a P-\(\Delta\) analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P-\(\Delta\) analysis be done at the factored load level of 1.2 DL plus 0.5 LL (White and Hajjar 1991).

The same conditions and limitations as AISC-LRFD93 apply.

**Calculation of Nominal Strengths**

The nominal strengths in compression, tension, bending, and shear for Seismic, Compact, Noncompact, and Slender sections according to the UBC-LRFD97 are calculated in the same way as described in section “Calculation of Nominal Strengths” of Chapter IV without any exception (UBC 2204.1 2205.2, 2205.3, 2206, 2210.2, 2210.3). The nominal strengths for Seismic sections are calculated in the same way as for Compact sections.

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

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

*If the user specifies nonzero factored strengths for one or more elements in the “Capacity Overwrites” form, these values will override the above mentioned calculated values for those elements. The specified factored strengths should be based on the principal axes of bending.*

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

- $\varphi_t =$ Resistance factor for tension, 0.9 \quad (LRFD D1, H1, SAM 2, 6)
- $\varphi_c =$ Resistance factor for compression, 0.85 \quad (LRFD E2, E3, H1)
- $\varphi_c =$ Resistance factor for compression in angles, 0.90 \quad (LRFD SAM 4, 6)
- $\varphi_b =$ Resistance factor for bending, 0.9 \quad (LRFD F1, H1, A-F1, A-G2, SAM 5)
- $\varphi_s =$ Resistance factor for shear, 0.9 \quad (LRFD F2, A-F2, A-G3, SAM 3)
All limitations and warnings related to nominal strengths calculation in AISC-LRFD93 also apply in this code.

**Calculation of Capacity Ratios**

The capacity ratios in UBC-LRFD97 are calculated in the same way as described in section “Calculation of Capacity Ratios” of Chapter IV with some modifications as described below.

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

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

**Axial and Bending Stresses**

The interaction ratio is determined based on the ratio $\frac{P_u}{\phi P_n}$. If $P_u$ is tensile, $P_n$ is the nominal axial tensile strength and $\phi = \phi_y = 0.9$; and if $P_u$ is compressive, $P_n$ is the nominal axial compressive strength and $\phi = \phi_c = 0.85$, except for angle sections $\phi = \phi_c = 0.90$ (LRFD SAM 6). In addition, the resistance factor for bending, $\phi_b = 0.9$.

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

$$\frac{P_u}{\phi P_n} + \frac{8}{9} \left( \frac{M_{u33}}{\phi_b M_{n33}} + \frac{M_{u22}}{\phi_b M_{n22}} \right).$$

(LRFD H1-1a, SAM 6-1a)

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

$$\frac{P_u}{2\phi P_n} + \left( \frac{M_{u33}}{\phi_b M_{n33}} + \frac{M_{u22}}{\phi_b M_{n22}} \right).$$

(LRFD H1-1b, SAM 6-1a)
For circular sections an SRSS (Square Root of Sum of Squares) combination is first made of the two bending components before adding the axial load component instead of the simple algebraic addition implied by the above formulas.

For Single-angle sections, the combined stress ratio is calculated based on the properties about the principal axes (LRFD SAM 5.3, 6). For I, Box, Channel, T, Double angle, Pipe, Circular and Rectangular sections, the principal axes coincide with their geometric axes. For Single-angle sections, principal axes are determined in ETABS. For general sections it is assumed that all section properties are given in terms of the principal directions and consequently no effort is made to determine the principal directions.

Shear Stresses

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

\[
\frac{V_{u2}}{\phi_s V_{n2}}, \quad \text{and} \quad \frac{V_{u3}}{\phi_s V_{n3}},
\]

where \( \phi_s = 0.9 \).

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

The special seismic requirements checked by the program for member design are dependent on the type of framing used and are described below for each type of framing (UBC 2204.1, 2205.2, 2205.3). The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seismic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

Ordinary Moment Frames

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

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

$$0.9 \, DL \pm \Omega_0 \, EL$$

$$1.2 \, DL + 0.5 \, LL \pm \Omega_0 \, EL$$

(UBC 2210.3, 2211.4.3.1)

Special Moment-Resisting Frames

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

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

$$0.9 \, DL \pm \Omega_0 \, EL$$

$$1.2 \, DL + 0.5 \, LL \pm \Omega_0 \, EL$$

(UBC 2210.3, 2211.4.3.1)

- In Seismic Zones 3 and 4, the I-shaped beams or columns, Channel-shaped beams or columns, and Box shaped columns are additionally checked for compactness criteria as described in Table VI-1 (UBC 2210.8, 2211.4.8.4.b, Table 2211.4.8-1). Compact I-shaped beam and column sections are additionally...
checked for \( b / t_f \) to be less than \( 52 / \sqrt{F_y} \). Compact Channel-shaped beam and column sections are additionally checked for \( b / t_f \) to be less than \( 52 / \sqrt{F_y} \). Compact I-shaped and Channel-shaped column sections are additionally checked for web-slenderness \( h / t_w \) to be less than the numbers given in Table VI-1. Compact box shaped column sections are additionally checked for \( b / t_f \) and \( d / t_w \) to be less than \( 110 / \sqrt{F_y} \). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the program checks the laterally unsupported length of beams to be less than \( 2500 / F_{yy} \). If the check is not satisfied, it is noted in the output (UBC 2211.4.8.8).

### Braced Frames

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

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

\[
\begin{align*}
0.9 \, DL & \pm \Omega_0 \, EL \\
1.2 \, DL & + 0.5 \, LL \pm \Omega_0 \, EL
\end{align*}
\]

(UBC 2210.3, 2211.4.3.1)

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

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

\[
P_u \leq 0.8 \phi \, P_u
\]

(UBC 2211.4.9.2.b)

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all braces are checked to be either Compact or Noncompact according to Table IV-2 (UBC 2211.4.9.2.d). The Box and Pipe shaped braces are additionally checked for compactness criteria as described in Table VI-1 (UBC
For box sections \( b/t_f \) and \( d/t_w \) is limited to \( 110/\sqrt{F_y} \), for pipe sections \( D/t \) is limited to \( 1300/\sqrt{F_y} \). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

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

**Eccentrically Braced Frames**

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

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

\[
\begin{align*}
0.9 \text{DL} \pm \Omega_0 \text{ EL} & \quad \text{(UBC 2210.3, 2211.4.3.1)} \\
1.2 \text{DL} + 0.5 \text{LL} \pm \Omega_0 \text{ EL} & \quad \text{(UBC 2210.3, 2211.4.3.1)}
\end{align*}
\]

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the I-shaped and Channel-shaped beams are additionally checked for compactness criteria as described in Table VI-1 (UBC 2211.4.10.2.a, 2210.8, 2211.4.8.4.b, Table 2211.4.8-1). Compact I-shaped and Channel-shaped beam sections are additionally checked for \( b_f/2t_f \) to be less than \( 52/\sqrt{F_y} \). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

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

\[
F_y \leq 50 \text{ ksi} \quad \text{(UBC 2211.4.10.2.b)}
\]

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

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

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\[ V_u \leq \varphi V_u, \quad \text{(UBC 2211.4.10.2.d)} \]

where,

\[
\varphi V_u = \min\left(\varphi V_{pu}, \varphi 2M_{pu} / e \right) , \quad \text{(UBC 2211.4.10.2.d)}
\]

\[
V_{pu} = V_p \sqrt{1 - \left( \frac{P_u}{P_y} \right)^2} , \quad \text{(UBC 2211.4.10.2.f)}
\]

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

\[
V_p = 0.6 F_y (d - 2t_f) t_w , \quad \text{(UBC 2211.4.10.2.d)}
\]

\[
M_p = Z F_y , \quad \text{(UBC 2211.4.10.2.d)}
\]

\[
\varphi = \varphi_v, \quad \text{(default is 0.9)}, \quad \text{(UBC 2211.4.10.2.d, 2211.4.10.2.f)}
\]

\[
P_y = A_y F_y . \quad \text{(UBC 2211.4.10.2.e)}
\]

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

\[
e \leq \begin{cases} 
1.15 - 0.5 \rho \frac{A_w}{A_y} & \text{if } \rho \frac{A_w}{A_y} \geq 0.3 , \\
1.6 \frac{M_p}{V_p} & \text{if } \rho \frac{A_w}{A_y} < 0.3 , 
\end{cases} \quad \text{(UBC 2211.4.10.2.f)}
\]

where,

\[
A_w = (d - 2t_f) t_w , \quad \text{and} \quad \text{(UBC 2211.4.10.2.f)}
\]

\[
\rho = P_u / V_u . \quad \text{(UBC 2211.4.10.2.f)}
\]

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

- The link beam rotation, \( \theta \), of the individual bay relative to the rest of the beam is calculated as the story drift \( \Delta_{sy} \) times bay length divided by the total lengths of link beams in the bay. In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the link beam rotation, \( \theta \), is checked as follows (UBC 2211.4.10.2.g).
\[ \theta \leq 0.090, \text{ where link beam clear length, } e \leq 1.6 \frac{M_s}{V_s}, \]
\[ \theta \leq 0.030, \text{ where link beam clear length, } e \geq 2.6 \frac{M_s}{V_s}, \text{ and} \]
\[ \theta \leq \text{value interpolated between 0.090 and 0.030 as the link beam clear length varies from } 1.6 \frac{M_s}{V_s} \text{ to } 2.6 \frac{M_s}{V_s}. \]

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

\[ \min \left( \varphi_{V_{pu}} \cdot \varphi_{V_{pa}} \cdot \frac{2M_{pa}}{e} \right), \quad \text{(UBC 2211.4.10.2.d)} \]

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

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

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

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

**Note:** The beam strength in flexure of the beam outside the link, is **NOT** currently checked to be at least 1.25 times the moment corresponding to the controlling link beam strength (UBC 2211.4.10.6.b). Users need to check for this requirement.
Special Concentrically Braced Frames

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

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, whenever $P_a/\phi P_a > 0.5$ in columns due to the prescribed loading combinations, the Special Seismic Load Combinations as described below are checked (UBC 2210.2, 2211.4.2.2.e, 2211.4.2.3.e, 2210.5, 2211.4.6.1).

Figure VI-1
Eccentrically Braced Frame Configurations
In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all columns are checked to be Compact according to Table IV-2. Compact box shaped column sections are additionally checked for $b/t_f$ and $d/t_w$ to be less than $110 / \sqrt{F_y}$ as described in Table VI-1 (UBC 2211.4.12.5.a). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property (UBC 2210.10.g, 2211.4.12.5.a).

In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, all braces are checked to be Compact according to Table IV-2 (UBC 2210.10.c, 2211.4.12.2.d). The Angle, Double-angle, Box and Pipe shaped braces are additionally checked for compactness criteria as described in Table VI-1 (UBC 2210.10.c, 2211.4.12.2.d). For box sections $b/t_f$ and $d/t_w$ is limited to $100 / \sqrt{F_y}$, for pipe sections $D/t$ is limited to $1300 / F_y$. If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the section property.

In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, the compressive strength for braces is taken as $\varphi_c P_n$ (UBC 2210.10.b, 2211.4.12.2.b). Unlike Braced Frames, no reduction is required.

$$P_u \leq \varphi_c P_n \quad \text{(UBC 2211.4.12.2.b)}$$

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

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

$$1.0 \text{DL} + 0.7 \text{LL} \pm P_b \quad \text{(UBC 2210.10.e, 2211.4.12.4.a.3)}$$
$$0.9 \text{DL} \pm P_b \quad \text{(UBC 2210.10.e, 2211.4.12.4.a.3)}$$

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

When using UBC-LRFD97 design code, the structural joints are checked and/or designed for the following:

- Check for the requirement of continuity plate and determination of its area
- Check for the requirement of doubler plate and determination of its thickness
- Check for the ratio of beam flexural strength to column flexural strength
- Reporting the beam connection shear
- Reporting the brace connection force

Design of Continuity Plates

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

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

To achieve a beam/column moment connection, continuity plates such as shown in Figure II–4 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.

For connection conditions described in the last two steps above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam. However, for the connection condition described by the first step above, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends upon the magnitude of the beam-flange force and the properties of the column. This is the condition that the program investigates. Columns of I-sections only are investigated. The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e. parallel to the column major direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.
The program first evaluates the need for continuity plates. Continuity plates will be required if any of the following four conditions are not satisfied:

- The column flange design strength in bending must be larger than the beam flange force, i.e.,
  \[ \varphi R_u = (0.9)6.25 t_{fc}^2 F_{ye} \geq P_{bf} \] (LRFD K1-1)

- The design strength of the column web against local yielding at the toe of the fillet must be larger than the beam flange force, i.e.,
  \[ \varphi R_u = (1.0) (5.0 k_e + t_{fb}) F_{ye} t_{we} \geq P_{bf} \] (LRFD K1-2)

- The design strength of the column web against crippling must be larger than the beam flange force, i.e.,
  \[ \varphi R_u = (0.75)68 t_{we} \left[ 1 + \frac{3}{d_e} \left( \frac{t_{we}}{t_{fc}} \right)^{1.5} \right] \sqrt{\frac{F_{ye}}{F_{yc}}} \geq P_{bf} \] (LRFD K1-5a)

- The design compressive strength of the column web against buckling must be larger than the beam flange force, i.e.,
  \[ \varphi R_u = (0.9) \frac{4100 t_{we}^3}{d_e} \sqrt{\frac{F_{ye}}{F_{yc}}} \geq P_{bf} \] (LRFD K1-8)

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

\[ A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{ye})} - 12 t_{we}^2 \] (LRFD K1.9, E2)

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

The formula above assumes the continuity plate plus a width of web equal to \( 12t_{we} \) act as a compression member to resist the applied load (LRFD K1.9). The formula also assumes \( \varphi = 0.85 \) and \( F_{cy} = 0.9F_{ye} \). This corresponds to an assumption of \( \lambda = 0.5 \) in the column formulas (LRFD E2-2). The user should choose the continuity plate cross-section such that this is satisfied. As an example when using \( F_{ye} = 50 \) ksi and assuming the effective length of the stiffener as a column to be \( 0.75h \) (LRFD K1.9) the required minimum radius of gyration of the stiffener cross-section would be \( r = 0.02h \) to obtain \( \lambda = 0.5 \) (LRFD E2-4).
If continuity plates are required, they must satisfy a minimum area specification defined as follows:

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

\[
  t_{cp}^{min} = \max\left\{0.5 \ t_{fb}, \ \frac{F_y}{95} b_{fb}\right\} \quad (LRFD \ K1.9.2)
\]

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

\[
b_{cp}^{min} = 2 \left( \frac{b_{fb}}{3} - \frac{t_{wc}}{2} \right) \quad (LRFD \ K1.9.1)
\]

- So that the minimum area is given by:

\[
A_{cp}^{min} = t_{cp}^{min} b_{cp}^{min} \quad (LRFD \ K1.9.1)
\]

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

In the equations above,

\[
\begin{align*}
A_{cp} &= \text{Required continuity plate area} \\
F_{yc} &= \text{Yield stress of the column and continuity plate material} \\
d_{b} &= \text{Beam depth} \\
d_{c} &= \text{Column depth} \\
h &= \text{Clear distance between flanges of column less fillets for rolled shapes} \\
k_{c} &= \text{Distance between outer face of the column flange and web toe of its fillet.} \\
M_{u} &= \text{Factored beam moment} \\
P_{bf} &= \text{Beam flange force, assumed as } M_{u}/(d_{b} - t_{fb}) \\
R_{n} &= \text{Nominal strength} \\
t_{fb} &= \text{Beam flange thickness} \\
t_{fc} &= \text{Column flange thickness} \\
t_{wc} &= \text{Column web thickness} \\
\varphi &= \text{Resistance factor}
\end{align*}
\]

The special seismic requirements additionally checked by the program are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2211.4.2.1 for frames in Seis-
mic Zones 0 and 1 and Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1), on UBC Section 2211.4.2.2 for frames in Seismic Zone 2 with Importance factor greater than 1 (UBC 2210.2, UBC 2211.4.2.2), and on UBC Section 2211.4.2.3 for frames in Seismic Zones 3 and 4 (UBC 2210.2, UBC 2211.4.2.3). No special requirement is checked for frames in Seismic Zones 0 and 1 and in Seismic Zone 2 with Importance factor equal to 1 (UBC 2210.2, UBC 2211.4.2.1).

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

$$P_{bf} = \frac{M_{pb}}{(d_b - t_{fb})} \quad (\text{UBC 2211.4.7.2.a, 2211.4.8.2.a.1})$$

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

In Seismic Zone 2 with Importance factor greater than 1, for Special Moment-Resisting Frames, for determining the need for continuity plates at joints due to tension transfer from the beam flanges, the force $P_{bf}$ is taken as $f_{yrb} A_{bf}$ (UBC 2211.4.8.2.a.1).

$$P_{bf} = 1.8 f_{yrb} A_{bf} \quad (\text{Zone 3 and 4}) \quad (\text{UBC 2211.4.8.5})$$

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

For design of the continuity plate the beam flange force is taken as $P_{bf} = \frac{M_{pb}}{(d_b - t_{fb})}$ (UBC 2211.4.8.2.a.1).

$$P_{bf} = \frac{M_{pb}}{(d_b - t_{fb})} \quad (\text{UBC 2211.4.8.2.a.1})$$

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

124 Joint Design
Design of Doubler Plates

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

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

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending upon the loading and the geometry of the steel beams that frame into the column, either along the column major direction, or at an angle so that the beams have components along the column major direction. See Figure II-5. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames only. No check is made for braced frames.

The program calculates the required thickness of doubler plates (see Figure II-5) for AISC-LRFD93 similar to the procedure described in Section “Design of Doubler Plates” in Chapter II except that the following algorithms are used. The shear force in the panel zone, is given by

\[
V_r = \sum_{n=1}^{N} \frac{M_{bn}}{d_n - t_{fn}} \cos \frac{\theta_n}{2} - V_c
\]

The nominal panel shear strength is given by

\[
R_y = 0.6F_y d_c t_r , \quad \text{for } P_u \leq 0.4P_y \text{ or if } P_u \text{ is tensile, and } \quad (LRFD \text{ K1-9})
\]

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

By using \( V_p = \phi R_y \), with \( \phi = 0.9 \), the required column web thickness \( t_r \) can be found.

The extra thickness, or thickness of the doubler plate is given by
\[ t_{dp} = t_r - t_w \geq \frac{h}{418 \sqrt{F_y}}, \]  

(LRFD F2-1)

where,

- \( F_y \) = Column and doubler plate yield stress
- \( t_r \) = Required column web thickness
- \( t_{dp} \) = Required doubler plate thickness
- \( t_w \) = Column web thickness
- \( h \) = \( d_e - 2t_f \) if welded, \( d_e - 2k_e \) if rolled,
- \( V_p \) = Panel zone shear
- \( V_c \) = Column shear in column above
- \( F_y \) = Beam flange forces
- \( n_b \) = Number of beams connecting to column
- \( d_n \) = Depth of \( n \)-th beam connecting to column
- \( \theta_n \) = Angle between \( n \)-th beam and column major direction
- \( d_c \) = Depth of column clear of fillets, equals \( d - 2k \)
- \( M_{bm} \) = Calculated factored beam moment from the corresponding loading combination
- \( R_v \) = Nominal shear strength of panel
- \( P_u \) = Column factored axial load
- \( P_y \) = Column axial yield strength, \( F_y A \)

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

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

- In Seismic Zones 3 and 4, for Special Moment-Resisting Frames, the panel zone doubler plate requirements that are reported will develop the lesser of beam moments equal to 0.9 of the plastic moment capacity of the beam
The capacity of the panel zone in resisting this shear is taken as (UBC 2211.8.3.a):

\[
\varphi_v V_n = 0.60 \varphi_v F_y d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p}\right)
\]

(UBC 2211.4.8.3.a)

giving the required panel zone thickness as

\[
t_p = \frac{V_p}{0.6 \varphi_v F_y d_c} - \frac{3 b_{cf} t_{cf}^2}{d_b d_c} \geq \frac{h}{2418\sqrt{F_y}},
\]

(UBC 2211.4.8.3, LRFD F2-1)

and the required doubler plate thickness as

\[
t_{dp} = t_p - t_{wc}
\]

where,

\[
\begin{align*}
\varphi_v &= 0.75, \\
b_{cf} &= \text{width of column flange}, \\
t_{cf} &= \text{thickness of column flange}, \\
t_p &= \text{required column web thickness}, \\
h &= d_c - 2t_{fc} \text{ if welded, } d_c - 2k_c \text{ if rolled, and} \\
d_b &= \text{depth of deepest beam framing into the major direction of the column.}
\end{align*}
\]

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

\[
t_{wc} \geq \frac{(d_c - 2t_{fc}) + (d_c - 2t_{f})}{90}
\]

(UBC 2211.4.8.3.b)

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

- In Seismic Zones 3 and 4, for Eccentrically Braced Frames, the doubler plate requirements are checked similar to the doubler plate checks for special Moment-Resisting frames as discussed above (UBC 2211.4.10.7).
Weak Beam Strong Column Measure

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

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

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

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

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

where,

- $R_{maj, min} =$ Plastic moment capacity ratios, in the major and minor directions of the column, respectively
- $M_{pbn} =$ Plastic moment capacity of $n$-th beam connecting to column
- $\theta_n =$ Angle between the $n$-th beam and the column major direction
- $M_{pcax, y} =$ Major and minor plastic moment capacities, reduced for axial force effects, of column above story level
- $M_{pcby, y} =$ Major and minor plastic moment capacities, reduced for axial force effects, of column below story level
- $n_b =$ Number of beams connecting to the column

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

$$M_{pc} = Z_e \left( F_{ye} - \frac{P_{ae}}{A_e} \right)$$  \hspace{1cm} (UBC 2211.4.8.6 8-3)
where,

\[ Z_c = \text{Plastic modulus of column}, \]
\[ F_{yc} = \text{Yield stress of column material}, \]
\[ P_{uc} = \text{Maximum axial strength in the column in compression, } P_{uc} \geq 0, \text{ and} \]
\[ A_{gc} = \text{Gross area of column}. \]

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

**Evaluation of Beam Connection Shears**

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

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

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

  \[ 0.9 \text{ DL } \pm \Omega_0 \text{ EL} \]  
  \[ 1.2 \text{ DL } + 0.5 \text{ LL } \pm \Omega_0 \text{ EL} \]  

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

\[ V_s = \frac{CM_{pb}}{L} + 1.2V_{DL} + 0.5V_{LL} \]  

(UBC 2211.4.8.2.b)
where

\[ V = \text{Shear force corresponding to END I or END J of beam}, \]
\[ C = 0 \text{ if beam ends are pinned, or for cantilever beam,} \]
\[ = 1 \text{ if one end of the beam is pinned,} \]
\[ = 2 \text{ if no ends of the beam are pinned,} \]
\[ M_{pb} = \text{Plastic moment capacity of the beam, } Z F_y, \]
\[ L = \text{Clear length of the beam,} \]
\[ V_{DL} = \text{Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the dead load only, and} \]
\[ V_{LL} = \text{Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the live load only.} \]

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

### Evaluation of Brace Connection Forces

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

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

- In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for ordinary Braced Frames, the bracing connection force is reported at least as the smaller of the tensile strength of the brace \((F_A)\) (UBC 2211.4.9.3.a.1) and the following special loading combinations (UBC 2211.4.9.3.a.2):
In Seismic Zones 3 and 4 and in Seismic Zone 2 with Importance factor greater than 1, for Eccentrically Braced Frames, the bracing connection force is reported as at least the nominal strength of the brace (UBC 2211.4.10.6.d).

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

\[
\begin{align*}
0.9 \text{ DL} &\pm \Omega_0 \text{ EL} & \text{(UBC 2210.3, 2211.4.3.1)} \\
1.2 \text{ DL} + 0.5 \text{ LL} &\pm \Omega_0 \text{ EL} & \text{(UBC 2210.3, 2211.4.3.1)}
\end{align*}
\]
This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the CAN/CSA-S16.1-94 design code (CISC 1995). Various notations used in this chapter are described in Table VII-1.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this section. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ratio is also calculated separately.

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to Newton-Millimeter-Second units unless otherwise noted.
### Table VII-1

**CISC 94 Notations**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Cross-sectional area, mm$^2$</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross cross-sectional area, mm$^2$</td>
</tr>
<tr>
<td>$A_{a_2}, A_{a_3}$</td>
<td>Major and minor shear areas, mm$^2$</td>
</tr>
<tr>
<td>$A_w$</td>
<td>Shear area, mm$^2$</td>
</tr>
<tr>
<td>$C_f$</td>
<td>Euler buckling strength, N</td>
</tr>
<tr>
<td>$C_r$</td>
<td>Factored compressive axial load, N</td>
</tr>
<tr>
<td>$C_s$</td>
<td>Factored compressive axial strength, N</td>
</tr>
<tr>
<td>$C_w$</td>
<td>Warping constant, mm$^6$</td>
</tr>
<tr>
<td>$C_y$</td>
<td>Compressive axial load at yield stress, $A_y F_y$, N</td>
</tr>
<tr>
<td>$D$</td>
<td>Outside diameter of pipes, mm</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity, MPa</td>
</tr>
<tr>
<td>$F_y$</td>
<td>Specified minimum yield stress, MPa</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus, MPa</td>
</tr>
<tr>
<td>$I_{33}, I_{22}$</td>
<td>Major and minor moment of inertia, mm$^4$</td>
</tr>
<tr>
<td>$J$</td>
<td>Torsional constant for the section, mm$^4$</td>
</tr>
<tr>
<td>$K$</td>
<td>Effective length factor</td>
</tr>
<tr>
<td>$K_{33}, K_{22}$</td>
<td>Effective length $K$-factors in the major and minor directions (assumed as 1.0 unless overwritten by user)</td>
</tr>
<tr>
<td>$L$</td>
<td>Laterally unbraced length of member, mm</td>
</tr>
<tr>
<td>$M_{33}, M_{22}$</td>
<td>Factored major and minor bending loads, N-mm</td>
</tr>
<tr>
<td>$M_{33}, M_{p22}$</td>
<td>Major and minor plastic moments, N-mm</td>
</tr>
<tr>
<td>$M_{33}, M_{r22}$</td>
<td>Factored major and minor bending strengths, N-mm</td>
</tr>
<tr>
<td>$M_o$</td>
<td>Critical elastic moment, N-mm</td>
</tr>
<tr>
<td>$M_{33}, M_{y22}$</td>
<td>Major and minor yield moments, N-mm</td>
</tr>
<tr>
<td>$S_{33}, S_{22}$</td>
<td>Major and minor section moduli, mm$^3$</td>
</tr>
<tr>
<td>$T_f$</td>
<td>Factored tensile axial load, N</td>
</tr>
<tr>
<td>$T_s$</td>
<td>Factored tensile axial strength, N</td>
</tr>
<tr>
<td>$U_1$</td>
<td>Moment magnification factor to account for deformation of member between ends</td>
</tr>
<tr>
<td>$U_2$</td>
<td>Moment magnification factor (on sidesway moments) to account for $P_\Delta$</td>
</tr>
<tr>
<td>$V_{33}, V_{33}$</td>
<td>Factored major and minor shear loads, N</td>
</tr>
<tr>
<td>$V_{33}, V_{33}$</td>
<td>Factored major and minor shear strengths, N</td>
</tr>
<tr>
<td>$Z_{33}, Z_{22}$</td>
<td>Major and minor plastic moduli, mm$^3$</td>
</tr>
</tbody>
</table>
Chapter VII  Check/Design for CISC94

\[ b = \text{Nominal dimension of longer leg of angles} \]
\[ (b_f - 2t_w) \text{ for welded} \]
\[ (b_f - 3t_f) \text{ for rolled box sections, mm} \]
\[ b_f = \text{Flange width, mm} \]
\[ d = \text{Overall depth of member, mm} \]
\[ h = \text{Clear distance between flanges, taken as} (d - 2t_f) \text{, mm} \]
\[ k = \text{Web plate buckling coefficient, assumed as 5.34 (no stiffeners)} \]
\[ k = \text{Distance from outer face of flange to web toe of fillet, mm} \]
\[ l = \text{Unbraced length of member, mm} \]
\[ l_{33}, l_{22} = \text{Major and minor direction unbraced member lengths, mm} \]
\[ r = \text{Radius of gyration, mm} \]
\[ r_{33}, r_{22} = \text{Radii of gyration in the major and minor directions, mm} \]
\[ r_z = \text{Minimum Radius of gyration for angles, mm} \]
\[ t = \text{Thickness, mm} \]
\[ t_f = \text{Flange thickness, mm} \]
\[ t_w = \text{Web thickness, mm} \]
\[ \lambda = \text{Slenderness parameter} \]
\[ \phi = \text{Resistance factor, taken as 0.9} \]
\[ \omega_1 = \text{Moment Coefficient} \]
\[ \omega_{33}, \omega_{12} = \text{Major and minor direction moment coefficients} \]
\[ \omega_2 = \text{Bending coefficient} \]

Table VII-1

| CISC 94 Notations (cont.) |
Design Loading Combinations

The design load combinations are the various combinations of the load cases for which the structure needs to be checked. For the CAN/CSA-S16.1-94 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake induced load (EL), and considering that wind and earthquake forces are reversible, then the following load combinations may have to be defined (CISC 7.2):

1.25 DL
1.25 DL + 1.50 LL (CISC 7.2.2)
1.25 DL ± 1.50 WL
0.85 DL ± 1.50 WL
1.25 DL + 0.7 (1.50 LL ± 1.50 WL) (CISC 7.2.2)
1.00 DL ± 1.00 EL
1.00 DL + 0.50 LL ± 1.00 EL (CISC 7.2.6)

These are also the default design load combinations whenever the CISC Code is used. In generating the above default loading combinations, the importance factor is taken as 1.

The user should use other appropriate loading combinations if roof live load is separately treated, other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

When using the CISC code, ETABS design assumes that a P-Δ analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is suggested that the P-Δ analysis be done at the factored load level of 1.25 DL plus 1.05 LL. See also White and Hajjar (1991).

For the gravity load case only, the code (CISC 8.6.2) requires that notional lateral loads be applied at each story, equal to 0.005 times the factored gravity loads acting at each story. If extra load cases are used for such analysis, they should be included in the loading combinations with due consideration to the fact that the notional lateral forces can be positive or negative.
Classification of Sections

For the determination of the nominal strengths for axial compression and flexure, the sections are classified as either Class 1 (Plastic), Class 2 (Compact), Class 3 (Noncompact), or Class 4 (Slender). The program classifies the individual sections according to Table VII-2 (CISC 11.2). According to this table, a section is classified as either Class 1, Class 2, or Class 3 as applicable.

If a section fails to satisfy the limits for Class 3 sections, the section is classified as Class 4. Currently ETABS does not check stresses for Class 4 sections.

Calculation of Factored Forces

The factored member forces for each load combination are calculated at each of the previously defined stations. These member forces are $T_f$ or $C_f$, $M_{f33}$, $M_{f22}$, $V_{f2}$ corresponding to factored values of the tensile or compressive axial load, the major moment, the minor moment, the major direction shear, and the minor direction shear, respectively.

Because ETABS design assumes that the analysis includes P-Δ effects, any magnification of sidesway moments due to the second order effects are already included, therefore $U_2$ for both directions of bending is taken as unity. It is suggested that the P-Δ analysis be done at the factored load level of 1.25 DL plus 1.05 LL. See also White and Hajjar (1991).

However, the user can overwrite the values of $U_2$ for both major and minor direction bending. In this case $M_f$ in a particular direction is taken as:

$$M_f = M_{f_k} + U_2M_{f_t}, \text{ where} \quad \text{(CISC 8.6.1)}$$

$$U_2 = \text{Moment magnification factor for sidesway moments},$$

$$M_{f_k} = \text{Factored moments not causing translation},$$

$$M_{f_t} = \text{Factored moments causing sidesway}.$$
<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Ratio of Section Checked</th>
<th>Class 1 (Plastic)</th>
<th>Class 2 (Compact)</th>
<th>Class 3 (Noncompact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>b _ / 2_t _</td>
<td>( \leq 145 / \sqrt{F_y} )</td>
<td>( \leq 170 / \sqrt{F_y} )</td>
<td>( \leq 200 / \sqrt{F_y} )</td>
</tr>
<tr>
<td></td>
<td>h / t_w</td>
<td>( \leq 1100 \sqrt{F_y} \left(1 - 0.39 \frac{C_I}{C_I} \right) )</td>
<td>( \leq 1700 \sqrt{F_y} \left(1 - 0.61 \frac{C_I}{C_I} \right) )</td>
<td>( \leq 1900 \sqrt{F_y} \left(1 - 0.65 \frac{C_I}{C_I} \right) )</td>
</tr>
<tr>
<td>BOX</td>
<td>b / t _</td>
<td>( \leq 420 / \sqrt{F_y} ) (rolled)</td>
<td>( \leq 525 / \sqrt{F_y} ) (welded)</td>
<td>( \leq 670 / \sqrt{F_y} )</td>
</tr>
<tr>
<td></td>
<td>h / t_w</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>b _ / t _</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>( \leq 200 / \sqrt{F_y} ) As for I-shapes</td>
</tr>
<tr>
<td></td>
<td>h / t_w</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>b _ / 2_t _</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>( \leq 200 / \sqrt{F_y} ) ( \leq 340 / \sqrt{F_y} )</td>
</tr>
<tr>
<td></td>
<td>d / t_w</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>DOUBLE ANGLE</td>
<td>b / t _</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>( \leq 200 / \sqrt{F_y} )</td>
</tr>
<tr>
<td></td>
<td>d / t_w</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>ANGLE</td>
<td>b / t _</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>( \leq 200 / \sqrt{F_y} )</td>
</tr>
<tr>
<td></td>
<td>d / t_w</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>PIPE (Flexure)</td>
<td>D / t _</td>
<td>( \leq 13000 / F_y )</td>
<td>( \leq 18000 / F_y )</td>
<td>( \leq 66000 / F_y )</td>
</tr>
<tr>
<td>PIPE (Axial)</td>
<td>D / t _</td>
<td>—</td>
<td>—</td>
<td>( \leq 23000 / F_y )</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td></td>
<td></td>
<td>Assumed Class 2</td>
</tr>
<tr>
<td>RECTANGULAR</td>
<td>—</td>
<td></td>
<td></td>
<td>Assumed Class 2</td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td></td>
<td></td>
<td>Assumed Class 3</td>
</tr>
</tbody>
</table>

Table VII-2

Limiting Width-Thickness Ratios for Classification of Sections based on CISC 94
CISC95: Axes Conventions

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

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

Figure VII-1
CISC 94 Definition of Geometric Properties
Calculation of Factored Strengths

The factored strengths in compression, tension, bending, and shear are computed for Class 1, 2, and 3 sections in ETABS. The strength reduction factor, \( \varphi \), is taken as 0.9 (CISC 13.1).

For Class 4 (Slender) sections and any singly symmetric and unsymmetric sections requiring consideration of local buckling, flexural-torsional and torsional buckling, or web buckling, reduced nominal strengths may be applicable. The user must separately investigate this reduction if such elements are used.

If the user specifies nonzero factored strengths for one or more elements in the “Capacity Overwrites” form, these values will override the above mentioned calculated values for those elements.

Compression Strength

The factored axial compressive strength value, \( C_r \), for Class 1, 2, or 3 sections depends on a factor, \( \lambda \), which eventually depends on the slenderness ratio, \( KL/r \), which is the larger of \( (K_{33}/l_{33})/r_{33} \) and \( (K_{22}/l_{22})/r_{22} \), and is defined as

\[
\lambda = \frac{KL}{rFy\sqrt{E}}.
\]

For single angles \( r_z \) is used in place of \( r_{33} \) and \( r_{22} \). For members in compression, if \( KL/r \) is greater than 200, a message is printed (CISC 10.2.1).

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

\[
C_r = \varphi AF_y\left(1 + \lambda^{2n}\right)\frac{1}{\pi}, \quad \text{where} \quad \text{(CISC 13.3.1)}
\]

\( n \) is an exponent and it takes three possible values to match the strengths related to three SSRC curves. The default \( n \) is 1.34 which is assigned to W-shapes rolled in Canada, fabricated boxes and I shapes, and cold-formed non-stress relieved (Class C) hollow structural sections (HSS) (CISC 13.3.1, CISC C13.3, Manual Page 4-12, Manual Table 6-2). The WWF sections produced in Canada from plate with flame-cut edges and hot-formed or cold-relieved (Class H) HSS are assigned to a favorable value of \( n = 2.24 \) (CISC 13.3.1, CISC C13.3, Manual Page 4-12). For heavy sections, a smaller value of \( n (n = 0.98) \) is considered appropriate (CISC C13.3). ETABS assumes the value of \( n \) as follows:
\[ n = \begin{cases} 
2.24, & \text{for WWF, HS (Class H) and HSS (Class H) sections,} \\
1.34, & \text{for W, L, and 2L sections and normal HS and HSS sections,} \\
1.34, & \text{for other sections with thickness less than 25.4 mm,} \\
0.98, & \text{for other sections with thickness larger than or equal to 25.4 mm.} 
\end{cases} \]

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

### Tension Strength

The factored axial tensile strength value, \( T_r \), is taken as \( \phi A_g F_y \) (CISC 13.2.(a).(i)). For members in tension, if \( l/r \) is greater than 300, a message is printed accordingly (CISC 10.2.2).

\[ T_r = \phi A_g F_y \quad (\text{CISC 13.2}) \]

### Bending Strengths

The factored bending strength in the major and minor directions is based on the geometric shape of the section, the section classification for compactness, and the unbraced length of the member. The bending strengths are evaluated according to CISC as follows (CISC 13.5 and 13.6):

For laterally supported members, the moment capacities are considered to be as follows:

For Class 1 and 2,
\[ M_r = \phi ZF_y, \quad \text{and} \]

For Class 3,
\[ M_r = \phi SF_y. \quad (\text{CISC 13.5}) \]

Special considerations are required for laterally unsupported members. The procedure for the determination of moment capacities for laterally unsupported members (CISC 13.6) is described in the following subsections.

If the capacities (\( M_{r22} \) and \( M_{r33} \)) are overwritten by the user, they are used in the interaction ratio calculation when strengths are required for actual unbraced lengths. None of these overwritten capacities are used for strengths in laterally supported case.
I-shapes and Boxes

Major Axis of Bending

For Class 1 and 2 sections of I-shapes and boxes bent about the major axis,

when \( M_u > 0.67 M_{p33} \),

\[
M_{r3} = 1.15 \phi M_{p33} \left( 1 - 0.28 \frac{M_{p33}}{M_u} \right) \leq \phi M_{p33} \text{, and} \quad (CISC 13.6)
\]

when \( M_u \leq 0.67 M_{p33} \),

\[
M_{r33} = \phi M_u \text{, where} \quad (CISC 13.6)
\]

\[
M_{r33} = \text{Factored major bending strength}, \\
M_{p33} = \text{Major plastic moment, } Z_{33} F_{33}, \\
M_u = \text{Critical elastic moment,} \\
\frac{\omega_2 \pi}{L} \sqrt{EI_{33} GJ + \left( \frac{\pi E}{L} \right)^2 I_{22} C_w}, \quad (CISC 13.6)
\]

\[
L = \text{Laterally unbraced length, } l_{22}, \\
C_w = \text{Warping constant assumed as 0.0 for boxes, pipes,} \\
\text{rectangular and circular bars, and} \\
\omega_2 = 1.75 + 1.05 \left( \frac{M_u}{M_b} \right) + 0.30 \left( \frac{M_u}{M_b} \right)^2 \leq 2.5. \quad (CISC 13.6)
\]

\( M_u \) and \( M_b \) are end moments of the unbraced segment and \( M_u \) is less than \( M_b \), \( M_u/M_b \) being positive for double curvature bending and negative for single curvature bending. If any moment within the segment is greater than \( M_b \), \( \omega_2 \) is taken as 1.0. The program defaults \( \omega_2 \) to 1.0 if the unbraced length, \( l \) of the member is overwritten by the user (i.e. it is not equal to the length of the member). \( \omega_2 \) should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. The user can overwrite the value of \( \omega_2 \) for any member by specifying it.

For Class 3 sections of I-shapes, channels, boxes bent about the major axis,

when \( M_u > 0.67 M_{y33} \),
\[ M_{r33} = 1.15 \varphi \frac{M_{y33}}{1 - 0.28 \frac{M_{y33}}{M_u}} \leq \varphi M_{y33}, \text{ and} \quad (\text{CISC 13.6}) \]

when \( M_u \leq 0.67 M_{y33} \),

\[ M_{r33} = \varphi M_u, \quad \text{where} \quad (\text{CISC 13.6}) \]

\( M_{r33} \) and \( M_u \) are as defined earlier for Class 1 and 2 sections and \( M_{y33} \) is the major yield moment, \( S_{y3} F_y \).

**Minor Axis of Bending**

For Class 1 and 2 sections of I-shapes and boxes bent about their minor axis,

\[ M_{r22} = \varphi M_{p22} = \varphi Z_{22} F_y. \]

For Class 3 sections of I-shapes and boxes bent about their minor axis,

\[ M_{r22} = M_{y22} = S_{22} F_y. \]

**Rectangular Bar**

**Major Axis of Bending**

For Class 2 rectangular bars bent about their major axis,

when \( M_u > 0.67 M_{p33} \),

\[ M_{r33} = 1.15 \varphi M_{p33} \left( 1 - 0.28 \frac{M_{p33}}{M_u} \right) \leq \varphi M_{p33}, \text{ and} \quad (\text{CISC 13.6}) \]

when \( M_u \leq 0.67 M_{p33} \),

\[ M_{r33} = \varphi M_u. \quad (\text{CISC 13.6}) \]

**Minor Axis of Bending**

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

\[ M_{r22} = \varphi M_{p22} = \varphi Z_{22} F_y. \]

**Pipes and Circular Rods**

For pipes and circular rods bent about any axis
When \( M_u > 0.67 M_{\rho 33} \),

\[
M_{\rho 33} = 1.15 \varphi M_{\rho 33} \left( 1 - 0.28 \frac{M_{\rho 33}}{M_u} \right) \leq \varphi M_{\rho 33}, \text{ and (CISC 13.6)}
\]

when \( M_u \leq 0.67 M_{\rho 33} \),

\[
M_{\rho 33} = \varphi M_u. \text{ (CISC 13.6)}
\]

**Channel Sections**

**Major Axis of Bending**

For Class 3 channel sections bent about their major axis,

when \( M_u > 0.67 M_{y 33} \),

\[
M_{y 33} = 1.15 \varphi M_{y 33} \left( 1 - 0.28 \frac{M_{y 33}}{M_u} \right) \leq \varphi M_{y 33}, \text{ and (CISC 13.6)}
\]

when \( M_u \leq 0.67 M_{y 33} \),

\[
M_{y 33} = \varphi M_u.
\]

**Minor Axis of Bending**

For Class 3 channel sections bent about their minor axis,

\[
M_{y 22} = M_{y 22} = S_{y 22} F_y.
\]

**T-shapes and double angles**

**Major Axis of Bending**

For Class 3 sections of T-shapes and double angles the factored major bending strength is assumed to be (CISC 13.6d),

\[
M_{\rho 33} = \varphi \frac{\omega_2 \pi \sqrt{E I_{22} G J}}{L} \left[ B + \sqrt{1 + B^2} \right] \leq \varphi F_y S_{33}, \text{ where}
\]

\[
B = \pm 2.3 \frac{d}{L} \sqrt{I_{22} / J}.
\]
The positive sign for $B$ applies for tension in the stem of T-sections or the outstanding legs of double angles (positive moments) and the negative sign applies for compression in stem or legs (negative moments).

**Minor Axis of Bending**

For Class 3 sections of T-shapes and double angles the factored minor bending strength is assumed as,

$$M_{r,22} = \varphi F_y S_{22}.$$  

**Single Angle and General Sections**

For Class 3 single angles and for General sections, the factored major and minor direction bending strengths are assumed as,

$$M_{r,33} = \varphi F_y S_{33},$$ and $$M_{r,22} = \varphi F_y S_{22}.$$  

**Shear Strengths**

The factored shear strength, $V_{r,2}$, for major direction shears in I-shapes, boxes and channels is evaluated as follows (CISC 13.4.1.1):

- For $\frac{h}{t_w} \leq 439 \sqrt{\frac{k_v}{F_y}}$,

  $$V_{r,2} = \varphi A_w \left\{ 0.66F_y \right\}.$$  

(CISC 13.4.1.1)

- For $439 \sqrt{\frac{k_v}{F_y}} < \frac{h}{t_w} \leq 502 \sqrt{\frac{k_v}{F_y}}$,

  $$V_{r,2} = \varphi A_w \left\{ 290 \sqrt{\frac{k_v F_y}{h/t_w}} \right\}.$$  

(CISC 13.4.1.1)

- For $502 \sqrt{\frac{k_v}{F_y}} < \frac{h}{t_w} \leq 621 \sqrt{\frac{k_v}{F_y}}$,

  $$V_{r,2} = \varphi A_w \left\{ F_{cr} + F_i \right\},$$ where  

(CISC 13.4.1.1)
Assuming no stiffener is used, the value of $F_v$ is taken as zero.

- For $\frac{h}{t_w} > 621 \sqrt{\frac{k_v}{F_y}}$, 

$$V_{r2} = \phi A_y \left\{ F_{cre} + F_t \right\}, \text{ where }$$

$$F_{cre} = \frac{180000 k_v}{(h/t_w)^2}.$$ 

In the above equations, $k_v$ is the shear buckling coefficient, and it is defined as:

$$k_v = 4 + \frac{5.34}{(a/h)^2}, \quad a/h < 1$$

$$k_v = 5.34 + \frac{4}{(a/h)^2}, \quad a/h \geq 1$$

and the aspect ratio $a/h$ is the ratio of the distance between the stiffeners to web depth. Assuming no stiffener is used, the value of $k_v$ is taken as 5.34.

The factored shear strength for minor direction shears in I-shapes, boxes and channels is assumed as

$$V_{r2} = 0.66 \, \phi \, F_y A_{y,v}.$$  

(CISC 13.4.2)

The factored shear strength for major and minor direction shears for all other sections is assumed as (CISC 13.4.2):

$$V_{r2} = 0.66 \, \phi \, F_y A_{y,2}, \text{ and }$$

$$V_{r3} = 0.66 \, \phi \, F_y A_{y,3}.$$  

(CISC 13.4.2)
Calculation of Capacity Ratios

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

If the axial, flexural, and shear strengths of a section are overwritten by the user, the overwritten values are used in calculating the stress ratios. However, certain strengths can not be overwritten. If the axial and bending capacities are overwritten by the user, they are used in the interaction ratio calculation when strengths are required for actual unbraced lengths. None of these overwritten capacities are used for strengths in laterally supported case. More specific information is given in the following subsections as needed.

**During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.**

Axial and Bending Stresses

From the factored axial loads and bending moments at each station and the factored strengths for axial tension and compression and major and minor bending, an interaction capacity ratio is produced for each of the load combinations as follows:

**Compressive Axial Load**

If the axial load is compressive, the capacity ratio is given by:

\[
\frac{C_f}{C_r} + \frac{U_{13}}{M_{r33}} + \frac{U_{12}}{M_{r22}}, \text{ for all but Class 1 I-shaped sections (13.8.1)}
\]

\[
\frac{C_f}{C_r} + 0.85 \frac{U_{13}}{M_{r33}} + 0.6 \frac{U_{12}}{M_{r22}}, \text{ for Class 1 I-shaped sections (13.8.2)}
\]

The above ratios are calculated for each of the following conditions and the largest ratio is reported:
• **Cross-sectional Strength:**
  - The axial compression capacity is based on $\lambda = 0$.
    
    $C_r = \varphi A F_y$  \hspace{1cm} (CISC 13.3.1)
  
  - The $M_{r_{33}}$ and $M_{r_{22}}$ are calculated assuming that the member is laterally fully supported ($l_{22} = 0$ and $l_{33} = 0$) irrespective of its actual lateral bracing length (CISC 13.5), and
  
  - $U_{12}$ and $U_{13}$ are taken as 1.
    
    $U_{13} = U_{12} = 1.0$.  \hspace{1cm} (CISC 13.8.1, 13.8.2)
  
  If the capacities ($C_r$, $M_{r_{22}}$, and $M_{r_{33}}$) are overwritten by the user, they are assumed not to apply to this case and are ignored.

• **Overall Member Strength:**
  - The axial compression capacity is based on both major and minor direction buckling using both $K_{22} \frac{l_{22}}{r_{22}}$ and $K_{33} \frac{l_{33}}{r_{33}}$ as described in an earlier section (CISC 13.3.1).
  
  - $M_{r_{33}}$ and $M_{r_{22}}$ are calculated assuming that the member is laterally fully supported ($l_{22} = 0$ and $l_{33} = 0$) irrespective of its actual lateral bracing length (CISC 13.5), and
  
  - $U_{12}$ and $U_{13}$ are calculated using the expression given below for $U_1$. In this equation specific values for major and minor directions are to be used to calculate values of $U_{12}$ and $U_{13}$ (CISC 13.8.3).

  If the capacities ($C_r$, $M_{r_{22}}$, and $M_{r_{33}}$) are overwritten by the user, the only overwritten capacity used in this case is $C_r$.

• **Lateral-Torsional Buckling Strength:**
  - The axial compression capacity is based on weak-axis buckling only based on $K_{22} \frac{l_{22}}{r_{22}}$ (CISC 13.3.1),
  
  - $M_{r_{33}}$ and $M_{r_{22}}$ are calculated based on actual unbraced length (CISC 13.6), and
- \( U_{12} \) and \( U_{13} \) are calculated using the expression given below for \( U_1 \). In this equation specific values for major and minor directions are to be used to calculate values of \( U_{12} \) and \( U_{13} \) (CISC 13.8.3). Moreover, 
\[
U_{13} \geq 1 \text{ is enforced.} \quad \text{(CISC 13.3.1, 13.8.2)}
\]
If the capacities (\( C_r \), \( M_{r22} \), and \( M_{r33} \)) are overwritten by the user, all three overwritten capacities are used in this case.

In addition, For Class 1 I-shapes, the following ratio is also checked:
\[
\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}. \quad \text{(CISC 13.8.2)}
\]
If the capacities (\( M_{r22} \) and \( M_{r33} \)) are overwritten by the user, all these overwritten capacities are used in this case.

In the above expressions,
\[
U_1 = \frac{\omega_1}{1 - C_f/C_e}, \quad \text{(CISC 13.8.3)}
\]
\[
C_e = \frac{E I}{L^2},
\]
\[
\omega_1 = 0.6 - 0.4 \frac{M_a}{M_b} \geq 0.4, \quad \text{and}
\]
\( M_a/M_b \) is the ratio of the smaller to the larger moment at the ends of the member, \( M_a/M_b \) being positive for double curvature bending and negative for single curvature bending. \( \omega_1 \) is assumed as 1.0 for beams with transverse load and when \( M_b \) is zero.

The program defaults \( \omega_1 \) to 1.0 if the unbraced length, \( l \), of the member is redefined by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of \( \omega_1 \) for any member by specifying it. The factor \( U_1 \) must be a positive number. Therefore \( C_f \) must be less than \( C_e \). If this is not true, a failure condition is declared.

**Tensile Axial Load**

If the axial load is tensile the capacity ratio is given by the larger of two ratios. In the first case, the ratio is calculated as
assuming \( M_{r22} \) and \( M_{r33} \) are calculated based on fully supported member (\( l_{22} = 0 \) and \( l_{33} = 0 \)). If the capacities \((T_r, M_{r22}, M_{r33})\) are overwritten by the user, the only overwritten capacity used in this case is \( T_r \). \( M_{r22} \) and \( M_{r33} \) overwrites are assumed not to apply to this case and are ignored.

In the second case the ratio is calculated as

\[
\frac{T_r}{T_r} + \frac{M_{f33}}{M_{r33}} \left( \frac{M_{f22}}{M_{r22}} \right),
\]

(CISC 13.9)

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

For circular 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 above interaction formulas.

**Shear Stresses**

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

\[
\frac{V_{f2}}{V_{r2}} \quad \text{and} \quad \frac{V_{f3}}{V_{r3}},
\]
This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the BS 5950 design code (BSI 1990). Various notations used in this chapter are described in Table VIII-1.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this section. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ratio is also calculated separately.

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to Newton-Millimeter-Second units unless otherwise noted.
### Table VIII-1

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Cross-sectional area, mm$^2$</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross cross-sectional area, mm$^2$</td>
</tr>
<tr>
<td>$A_{a_1}, A_{a_2}$</td>
<td>Major and minor shear areas, mm$^2$</td>
</tr>
<tr>
<td>$B$</td>
<td>Breadth, mm</td>
</tr>
<tr>
<td>$D$</td>
<td>Depth of section, mm or outside diameter of pipes, mm</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity, MPa</td>
</tr>
<tr>
<td>$F_c$</td>
<td>Axial compression, N</td>
</tr>
<tr>
<td>$F_t$</td>
<td>Axial tension, N</td>
</tr>
<tr>
<td>$F_{v_1}, F_{v_2}$</td>
<td>Major and minor shear loads, N</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus, MPa</td>
</tr>
<tr>
<td>$H$</td>
<td>Warping constant, mm$^6$</td>
</tr>
<tr>
<td>$I_{33}$</td>
<td>Major moment of inertia, mm$^4$</td>
</tr>
<tr>
<td>$I_{22}$</td>
<td>Minor moment of inertia, mm$^4$</td>
</tr>
<tr>
<td>$J$</td>
<td>Torsional constant for the section, mm$^4$</td>
</tr>
<tr>
<td>$K$</td>
<td>Effective length factor</td>
</tr>
<tr>
<td>$K_{a_1}, K_{a_2}$</td>
<td>Major and minor effective length factors</td>
</tr>
<tr>
<td>$M$</td>
<td>Applied moment, N-mm</td>
</tr>
<tr>
<td>$M_{33}$</td>
<td>Applied moment about major axis, N-mm</td>
</tr>
<tr>
<td>$M_{22}$</td>
<td>Applied moment about minor axis, N-mm</td>
</tr>
<tr>
<td>$M_{a_{33}}$</td>
<td>Major maximum bending moment, N-mm</td>
</tr>
<tr>
<td>$M_{a_{22}}$</td>
<td>Minor maximum bending moment, N-mm</td>
</tr>
<tr>
<td>$M_b$</td>
<td>Buckling resistance moment, N-mm</td>
</tr>
<tr>
<td>$M_c$</td>
<td>Moment capacity, N-mm</td>
</tr>
<tr>
<td>$M_{33}$</td>
<td>Major moment capacity, N-mm</td>
</tr>
<tr>
<td>$M_{22}$</td>
<td>Minor moment capacity, N-mm</td>
</tr>
<tr>
<td>$M_E$</td>
<td>Elastic critical moment, N-mm</td>
</tr>
<tr>
<td>$P_c$</td>
<td>Compression resistance, N</td>
</tr>
<tr>
<td>$P_{c_{33}}, P_{c_{22}}$</td>
<td>Major and minor compression resistance, N</td>
</tr>
<tr>
<td>$P_t$</td>
<td>Tension capacity, N</td>
</tr>
<tr>
<td>$P_{v_1}, P_{v_2}$</td>
<td>Major and minor shear capacities, N</td>
</tr>
<tr>
<td>$S_{33}, S_{22}$</td>
<td>Major and minor plastic section moduli, mm$^3$</td>
</tr>
<tr>
<td>$T$</td>
<td>Thickness of flange or leg, mm</td>
</tr>
<tr>
<td>$Y_s$</td>
<td>Specified yield strength, MPa</td>
</tr>
<tr>
<td>$Z_{33}, Z_{22}$</td>
<td>Major and minor elastic section moduli, mm$^3$</td>
</tr>
</tbody>
</table>

**BS 5950 Notations**
Chapter VIII  Check/Design for BS 5950

Table VIII-1

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a )</td>
<td>Robertson constant</td>
</tr>
<tr>
<td>( b )</td>
<td>Outstand width, mm</td>
</tr>
<tr>
<td>( d )</td>
<td>Depth of web, mm</td>
</tr>
<tr>
<td>( h )</td>
<td>Story height, mm</td>
</tr>
<tr>
<td>( k )</td>
<td>Distance from outer face of flange to web toe of fillet, mm</td>
</tr>
<tr>
<td>( l )</td>
<td>Unbraced length of member, mm</td>
</tr>
<tr>
<td>( l_{33}, l_{22} )</td>
<td>Major and minor direction unbraced member lengths, mm</td>
</tr>
<tr>
<td>( l_{e33}, l_{e22} )</td>
<td>Major and minor effective lengths, mm ( (K_{33}l_{33}, K_{22}l_{22}) )</td>
</tr>
<tr>
<td>( m )</td>
<td>Equivalent uniform moment factor</td>
</tr>
<tr>
<td>( n )</td>
<td>Slenderness correction factor</td>
</tr>
<tr>
<td>( q_e )</td>
<td>Elastic critical shear strength of web panel, MPa</td>
</tr>
<tr>
<td>( q_{cr} )</td>
<td>Critical shear strength of web panel, MPa</td>
</tr>
<tr>
<td>( r_{33}, r_{22} )</td>
<td>Major and minor radii of gyration, mm</td>
</tr>
<tr>
<td>( r_c )</td>
<td>Minimum radius of gyration for angles, mm</td>
</tr>
<tr>
<td>( t )</td>
<td>Thickness, mm</td>
</tr>
<tr>
<td>( t_f )</td>
<td>Flange thickness, mm</td>
</tr>
<tr>
<td>( t_w )</td>
<td>Thickness of web, mm</td>
</tr>
<tr>
<td>( u )</td>
<td>Buckling parameter</td>
</tr>
<tr>
<td>( v )</td>
<td>Slenderness factor</td>
</tr>
<tr>
<td>( \beta )</td>
<td>Ratio of smaller to larger end moments</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>Constant ( \left( \frac{275}{\rho_y} \right)^{1/2} )</td>
</tr>
<tr>
<td>( \lambda )</td>
<td>Slenderness parameter</td>
</tr>
<tr>
<td>( \lambda_o )</td>
<td>Limiting slenderness</td>
</tr>
<tr>
<td>( \lambda_{LT} )</td>
<td>Equivalent slenderness</td>
</tr>
<tr>
<td>( \lambda_{Lo} )</td>
<td>Limiting equivalent slenderness</td>
</tr>
<tr>
<td>( \eta )</td>
<td>Perry factor</td>
</tr>
<tr>
<td>( \eta_{LT} )</td>
<td>Perry coefficient</td>
</tr>
<tr>
<td>( \rho_c )</td>
<td>Compressive strength, MPa</td>
</tr>
<tr>
<td>( \rho_E )</td>
<td>Euler strength, MPa</td>
</tr>
<tr>
<td>( \rho_y )</td>
<td>Yield strength, MPa</td>
</tr>
<tr>
<td>( \psi )</td>
<td>Monosymmetry index</td>
</tr>
</tbody>
</table>

BS 5950 Notations (cont.)
Design Loading Combinations

The design load combinations are the various combinations of the load cases for which the structure needs to be checked. According to the BS 5950 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), and earthquake load (EL), and considering that wind and earthquake forces are reversible, then the following load combinations may have to be considered (BS 2.4):

1.4 DL
1.4 DL + 1.6 LL (BS 2.4.1.1)

1.0 DL ± 1.4 WL
1.4 DL ± 1.4 WL
1.2 DL + 1.2 LL ± 1.2 WL (BS 2.4.1.1)

1.0 DL ± 1.4 EL
1.4 DL ± 1.4 EL
1.2 DL + 1.2 LL ± 1.2 EL

These are also the default design load combinations whenever BS 5950 Code is used. The user should use other appropriate loading combinations if roof live load is separately treated, other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

In addition to the above load combinations, the code requires that all buildings should be capable of resisting a notional design horizontal load applied at each floor or roof level. The notional load should be equal to the maximum of 0.01 times the factored dead load and 0.005 times the factored dead plus live loads (BS 2.4.2.3). The notional forces should be assumed to act in any one direction at a time and should be taken as acting simultaneously with the factored dead plus vertical imposed live loads. They should not be combined with any other horizontal load cases (BS 5.1.2.3). It is recommended that the user should define additional load cases for considering the notional load in ETABS and define the appropriate design combinations.

When using the BS 5950 code, ETABS design assumes that a P-Δ analysis has already been performed, so that moment magnification factors for the moments causing side-sway can be taken as unity. It is suggested that the P-Δ analysis be done at
the factored load level corresponding to 1.2 dead load plus 1.2 live load. See also White and Hajjar (1991).

**Classification of Sections**

The nominal strengths for axial compression and flexure are dependent on the classification of the section as Plastic, Compact, Semi-compact, or Slender. ETABS checks the sections according to Table VIII-2 (BS 3.5.2). The parameters $R$, $\gamma_e$, and $\varepsilon$ along with the slenderness ratios are the major factors in classification of section.

- $R$ is the ratio of mean longitudinal stress in the web to $\rho_y$ in a section. This implies that for a section in pure bending $R$ is zero. In calculating $R$, compression is taken as positive and tension is taken as negative. $R$ is calculated as follows:

$$R = \frac{P}{A_y \rho_y}$$

- $\alpha$ is given as $2\gamma_e/d$, where $\gamma_e$ is the distance from the plastic neutral axis to the edge of the web connected to the compression flange. For $\alpha > 2$, the section is treated as having compression throughout.

$$\alpha = \frac{\gamma_e}{d/2}$$

$$\gamma_e = \begin{cases} \left(\frac{D}{2} - T\right) - \frac{P}{2\rho_y t}, & \text{for I and Channel section} \\ \left(\frac{D}{2} - T\right) - \frac{P}{4\rho_y t}, & \text{for Box and Double Channel section} \end{cases}$$

In calculating $\gamma_e$, compression is taken as negative and tension is taken as positive.

- $\varepsilon$ is defined as follows:

$$\varepsilon = \left(\frac{275}{\rho_y}\right)^{1/2}$$

The section is classified as either Class 1 (Plastic), Class 2 (Compact), or Class 3 (Semi-compact) as applicable. **If a section fails to satisfy the limits for Class 3 (Semi-compact) sections, the section is classified as Class 4 (Slender). Currently ETABS does not check stresses for Slender sections.**
### Table VIII-2

**Limiting Width-Thickness Ratios for Classification of Sections based on BS 5950**

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Ratio Checked</th>
<th>Class 1 (Plastic)</th>
<th>Class 2 (Compact)</th>
<th>Class 3 (Semi-compact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$b/T$ (Rolled)</td>
<td>$\leq 8.5 \varepsilon$</td>
<td>$\leq 9.5 \varepsilon$</td>
<td>$\leq 15 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$b/T$ (welded)</td>
<td>$\leq 7.5 \varepsilon$</td>
<td>$\leq 8.5 \varepsilon$</td>
<td>$\leq 13 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$d/t$ webs ($\alpha &lt; 2$)</td>
<td>$\leq \frac{79 \varepsilon}{0.4 + 0.6 \alpha}$</td>
<td>$\leq \frac{98 \varepsilon}{\alpha}$</td>
<td>For $R &gt; 0$: $\leq \frac{120 \varepsilon}{1 + 1.5R}$ and $\leq \left(\frac{41}{R} - 13\right) \varepsilon$ (welded)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>For $R = 0$: $\leq 120 \varepsilon$ and $\leq \left(\frac{41}{R} - 2\right) \varepsilon$ (rolled)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>For $R &lt; 0$: $\leq \frac{120 \varepsilon}{(1 + R)^2}$ and $\leq 250 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$d/t$ webs ($\alpha \geq 2$) (rolled)</td>
<td>$\leq 39 \varepsilon$</td>
<td>$\leq 39 \varepsilon$</td>
<td>$\leq 39 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$d/t$ webs ($\alpha \geq 2$) (welded)</td>
<td>$\leq 28 \varepsilon$</td>
<td>$\leq 28 \varepsilon$</td>
<td>$\leq 28 \varepsilon$</td>
</tr>
<tr>
<td>BOX</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$b/T$ (Rolled)</td>
<td>$\leq 26 \varepsilon$</td>
<td>$\leq 32 \varepsilon$</td>
<td>$\leq 39 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$b/T$ (welded)</td>
<td>$\leq 23 \varepsilon$</td>
<td>$\leq 25 \varepsilon$</td>
<td>$\leq 28 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$d/t$</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>$b/T$ $d/t$</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
<td>As for I-shapes</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>$b/T$ $d/t$</td>
<td>$\leq 8.5 \varepsilon$</td>
<td>$\leq 9.5 \varepsilon$</td>
<td>$\leq 19 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\leq 8.5 \varepsilon$</td>
<td>$\leq 9.5 \varepsilon$</td>
<td>$\leq 19 \varepsilon$</td>
</tr>
<tr>
<td>DOUBLE ANGLE (separated)</td>
<td>$d/t$</td>
<td>$\leq 8.5 \varepsilon$</td>
<td>$\leq 9.5 \varepsilon$</td>
<td>$\leq 15 \varepsilon$</td>
</tr>
<tr>
<td></td>
<td>$(b + d)/t$</td>
<td>$\leq 23 \varepsilon$</td>
<td>$\leq 23 \varepsilon$</td>
<td>$\leq 23 \varepsilon$</td>
</tr>
</tbody>
</table>
Calculation of Factored Forces

The factored member loads that are calculated for each load combination are $F_t$, $F_c$, $M_{13}$, $M_{22}$, $F_{1v}$, and $F_{2v}$ corresponding to factored values of the tensile or compressive axial load, the major moment, the minor moment, the major direction shear load, and the minor direction shear load, respectively. These factored loads are calculated at each of the previously defined stations.

The moment magnification for non-sidesway moments is included in the overall buckling interaction equations.

$$M = M_g + \left\{ \frac{1}{1 - 200 \varphi_{s,\text{max}}} \right\} M_s, \text{ where}$$

$$M_{s,\text{max}} = \text{Maximum story-drift divided by the story-height,}$$
$$M_g = \text{Factored moments not causing translation, and}$$
$$M_s = \text{Factored moments causing sidesway.}$$

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Ratio Checked</th>
<th>Class 1 (Plastic)</th>
<th>Class 2 (Compact)</th>
<th>Class 3 (Semi-compact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANGLE</td>
<td>$b / t$</td>
<td>$\leq 8.5 \frac{e}{t}$</td>
<td>$\leq 9.5 \frac{e}{t}$</td>
<td>$\leq 15 \frac{e}{t}$</td>
</tr>
<tr>
<td></td>
<td>$(b + d) / t$</td>
<td>$\leq 23 \frac{e}{t}$</td>
<td>$\leq 23 \frac{e}{t}$</td>
<td>$\leq 23 \frac{e}{t}$</td>
</tr>
<tr>
<td>PIPE</td>
<td>$D / t$</td>
<td>$\leq 40 \frac{e^2}{t^2}$</td>
<td>$\leq 57 \frac{e^2}{t^2}$</td>
<td>$\leq 80 \frac{e^2}{t^2}$</td>
</tr>
<tr>
<td>SOLID CIRCLE</td>
<td>—</td>
<td>Assumed Compact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOLID RECTANGLE</td>
<td>—</td>
<td>Assumed Compact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td>Assumed Semi-compact</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table VIII-2 (cont.)

Limiting Width-Thickness Ratios for Classification of Sections based on BS 5950

Calculation of Factored Forces

The factored member loads that are calculated for each load combination are $F_t$, $F_c$, $M_{33}$, $M_{22}$, $F_{v,2}$, and $F_{v,3}$ corresponding to factored values of the tensile or compressive axial load, the major moment, the minor moment, the major direction shear load, and the minor direction shear load, respectively. These factored loads are calculated at each of the previously defined stations.

The moment magnification for non-sidesway moments is included in the overall buckling interaction equations.

$$M = M_g + \left\{ \frac{1}{1 - 200 \varphi_{s,\text{max}}} \right\} M_s, \text{ where}$$

$$M_{s,\text{max}} = \text{Maximum story-drift divided by the story-height,}$$
$$M_g = \text{Factored moments not causing translation, and}$$
$$M_s = \text{Factored moments causing sidesway.}$$
BS 5950: Axes Conventions

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

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

Figure VIII-1

BS 5950 Definition of Geometric Properties
The moment magnification factor for moments causing sidesway can be taken as unity if a P-Δ analysis is carried out. ETABS design assumes a P-Δ analysis has been done and, therefore, \( \phi_{\text{x},\text{max}} \) for both major and minor direction bending is taken as 0. It is suggested that the P-Δ analysis be done at the factored load level of 1.2 DL plus 1.2 LL. See also White and Hajjar (1991).

Calculation of Section Capacities

The strengths in compression, tension, bending, and shear are computed for Class 1, 2, and 3 sections according to the following subsections. By default, ETABS takes the design strength, \( \rho_{y} \), to be 1.0 times the minimum yield strength of steel, \( Y_{s} \), as specified by the user. In inputting values of the yield strength, the user should ensure that the thickness and the ultimate strength limitations given in the code are satisfied (BS 3.1.1).

\[
\rho_{y} = 1.0 Y_{s} \tag{BS 3.1.1}
\]

For Class 4 (Slender) sections and any singly symmetric and unsymmetric sections requiring special treatment, such as the consideration of local buckling, flexural-torsional and torsional buckling, or web buckling, reduced section capacities may be applicable. The user must separately investigate this reduction if such elements are used.

If the user specifies nonzero strengths for one or more elements in the “Capacity Overwrites” form, these values will override the above mentioned calculated values for those elements.

Compression Resistance

The compression resistance for plastic, compact, or semi-compact sections is evaluated as follows:

\[
P_{c} = A_{x} \rho_{c} \tag{BS 4.7.4}
\]

where \( \rho_{c} \) is the compressive strength given by

\[
\rho_{c} = \frac{\rho_{E} \rho_{y}}{\varphi + (\varphi^{2} - \rho_{E} \rho_{y})^{2/3}}, \tag{BS C.1}
\]

\[
\varphi = \rho_{y}^{2} + (\eta + 1) \rho_{E}\frac{\rho_{E}^{2}}{2} \tag{BS C.1}
\]
\[ \rho_E = \frac{\pi^2 E}{\lambda^2}, \]
\[ \eta = 0.001 \alpha (\lambda - \lambda_0) \geq 0, \quad (\text{BS C.2}) \]
\[ a = \text{Robertson constant from Table VIII-3, (BS C2, BS Table 25)} \]
\[ \lambda_0 = \text{Limiting slenderness, } 0.2 \left( \frac{\pi^2 E}{\rho_y} \right)^{\frac{1}{2}}, \quad (\text{BS C.2}) \]

\[ \lambda = \text{the slenderness ratio in either the major, } \lambda_{33} = l_{e33} / r_{33}, \text{ or } \]
\[ \text{in the minor, } \lambda_{22} = l_{e22} / r_{22} \text{ direction (BS 4.7.3.1).} \]

The larger of the two values is used in the above equations to calculate \( P_c \).

---

Table VIII-3
*Robertson Constant in BS 5950*

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Thickness (mm)</th>
<th>Axis of Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Major</td>
</tr>
<tr>
<td>I-SHAPE (rolled)</td>
<td>any</td>
<td>2.0</td>
</tr>
<tr>
<td>H-SHAPE (rolled)</td>
<td>( \leq 40 )</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>( &gt; 40 )</td>
<td>5.5</td>
</tr>
<tr>
<td>I-SHAPE (welded)</td>
<td>( \leq 40 )</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>( &gt; 40 )</td>
<td>3.5</td>
</tr>
<tr>
<td>BOX or Pipe (Rolled)</td>
<td>any</td>
<td>2.0</td>
</tr>
<tr>
<td>BOX (welded)</td>
<td>( \leq 40 )</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>( &gt; 40 )</td>
<td>5.5</td>
</tr>
<tr>
<td>CHANNEL, T-SHAPE, ANGLE</td>
<td>any</td>
<td>5.5</td>
</tr>
<tr>
<td>RECTANGULAR or CIRCLE</td>
<td>( \leq 40 )</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>( &gt; 40 )</td>
<td>5.5</td>
</tr>
<tr>
<td>GENERAL</td>
<td>any</td>
<td>5.5</td>
</tr>
</tbody>
</table>

---

160 Calculation of Section Capacities
For single angles \( r \) is used instead of \( r_{33} \) and \( r_{22} \). For members in compression, if \( \lambda \) is greater than 180, a message to that effect is printed (BS 4.7.3.2).

**Tension Capacity**

The tension capacity of a member is given by

\[
P_t = A \sigma_y.
\]

It should be noted that no net section checks are made. For main members in tension, the slenderness, \( \lambda \), should not be greater than 250 (BS 4.7.3.2). If \( \lambda \) is greater than 250, a message is displayed accordingly.

The user may have to separately investigate the members which are connected eccentrically to the axis of the member, for example angle sections.

**Moment Capacity**

The moment capacities in the major and minor directions, \( M_{c,33} \) and \( M_{c,22} \) are based on the design strength and the section modulus, the co-existent shear and the possibility of local buckling of the cross-section. Local buckling is avoided by applying a limitation to the width/thickness ratios of elements of the cross-section. The moment capacities are calculated as follows:

**Plastic and Compact Sections**

For plastic and compact sections, the moment capacities about the major and the minor axes of bending depend on the shear force, \( F_v \), and the shear capacity, \( P_v \).

For I, Box, Channel, and Double-Channel sections bending about the 3-3 axis the moment capacities considering the effects of shear force are computed as

\[
M_c = \sigma_y S \leq 1.2 \sigma_y Z, \quad \text{for} \quad F_v \leq 0.6P_v, \tag{BS 4.2.5}
\]

\[
M_c = \sigma_y (S - S_v \rho_y) \leq 1.2 \sigma_y Z, \quad \text{for} \quad F_v > 0.6P_v, \tag{BS 4.2.6}
\]

where

\[
S = \text{Plastic modulus of the gross section about the relevant axis,}
\]

\[
Z = \text{Elastic modulus of the gross section about the relevant axis,}
\]
S_v = Plastic modulus of the gross section about the relevant axis less the plastic modulus of that part of the section remaining after deduction of shear area i.e. plastic modulus of shear area. For example, for rolled I-shapes $S_{v2}$ is taken to be $tD^2/4$ and for welded I-shapes it is taken as $td^2/4$.

$P_v = $ The shear capacity described later in this chapter,

$\rho_v = \frac{2.5 F_v}{P_v} - 1.5$.

The combined effect of shear and axial forces is not being considered because practical situations do not warrant this. In rare cases, however, the user may have to investigate this independently, and if necessary, overwrite values of the section moduli.

For all other cases, the reduction of moment capacities for the presence of shear force is not considered. The user should investigate the reduced moment capacity separately. The moment capacity for these cases is computed in ETABS as

$$M_c = \rho_v S \leq 1.2 \rho_v Z.$$  \hspace{1cm} (BS 4.2.5)

**Semi-compact Sections**

Reduction of moment capacity due to coexistent shear does not apply for semi-compact sections.

$$M_c = \rho_v Z$$  \hspace{1cm} (BS 4.2.5)

**Lateral-Torsional Buckling Moment Capacity**

The lateral torsional buckling resistance moment, $M_b$, of a member is calculated from the following equations. The program assumes the members to be uniform (of constant properties) throughout their lengths. Furthermore members are assumed to be symmetrical about at least one axis.

For I, Box, T, Channel, and Double-Channel sections $M_b$ is obtained from

$$M_b = \frac{\rho_v S_{33} M_E}{\phi_b + (\phi_b^2 - \rho_v S_{33} M_E)^{1/2}}, \text{ where}$$  \hspace{1cm} (BS B2.1)
The Perry coefficient, $\eta_{LT}$, for rolled and welded sections is taken as follows:

For rolled sections

$$\eta_{LT} = \alpha_b \left\{ \lambda_{LT} - \lambda_{L0} \right\} \geq 0 \text{, and}$$

for welded sections

$$\eta_{LT} = 2\alpha_b \lambda_{L0} \geq 0 \text{, with } \alpha_b (\lambda_{LT} - \lambda_{L0}) \leq \eta_{LT} \leq 2\alpha_b (\lambda_{LT} - \lambda_{L0})$$

In the above definition of $\eta_{LT}$, $\lambda_{L0}$ and $\lambda_{LT}$ are the limiting equivalent slenderness and the equivalent slenderness, respectively, and $\alpha_b$ is a constant. $\alpha_b$ is taken as 0.007 (BS 2.3). For flanged members symmetrical about at least one axis and uniform throughout their length, $\lambda_{L0}$ is defined as follows:

$$\lambda_{L0} = 0.4 \sqrt{\frac{\pi^2 E}{\rho_y}}$$

For I, Channel, Double-Channel, and T sections $\lambda_{LT}$ is defined as

$$\lambda_{LT} = n u v \lambda$$

and for Box sections $\lambda_{LT}$ is defined as

$$\lambda_{LT} = 2.25 \ n \phi b \lambda^{1/2}$$

- $\lambda$ is the slenderness and is equivalent to $l_{e22}/r_{22}$.
- $n$ is the slenderness correction factor. For flanged members in general, not loaded between adjacent lateral restraints, and for cantilevers without intermediate lateral restraints, $n$ is taken as 1.0. For members with equal flanges loaded between adjacent lateral restraints, the value of $n$ is conservatively taken as given by the following formula. This, however, can be overwritten by the user for any member by specifying it (BS Table 13).

$$n = \frac{1}{\sqrt{C_b}} \leq 1.0$$

where

$$C_b = \frac{S_{33}}{\rho_y}$$
\[ C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_C}, \] and

\[ M_{\text{max}}, M_A, M_B, \text{ and } M_C \text{ are absolute values of maximum moment, 1/4 point, center of span and 3/4 point major moments respectively, in the member. The program also defaults } C_b \text{ to 1.0 if the unbraced length, } l, \text{ of the member is redefined by the user (i.e. it is not equal to the length of the member). } C_b \text{ should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. The user can overwrite the value of } C_b \text{ for any member.}

- \( u \) is the buckling parameter. It is conservatively taken as 0.9 for rolled I-shapes and channels. For any other section, \( u \) is taken as 1.0 (BS 4.3.7.5). For I, Channel, and Double-Channel sections,

\[ u = \left( \frac{4S_{33}^2 \gamma}{A^2(D-T)^2} \right)^{1/4}, \text{ for I, Channel, and Double-Channel, (BS B2.5b)} \]

\[ u = \left( \frac{I_{22}S_{33}^2 \gamma}{A^2H} \right)^{1/4}, \text{ for T section, where} \]

\[ \gamma = \left( 1 - \frac{I_{22}}{I_{33}} \right). \text{ (BS B2.5b)} \]

- \( v \) is the slenderness factor. For I, Channel, Double-Channel, and T sections, it is given by the following formula.

\[ v = \frac{1}{\left[ 4N(N-1) + \frac{1}{20} \left( \frac{\psi}{N} \right)^2 + \psi^2 \right]^{1/2}}, \text{ where} \]

\[ N = \begin{cases} 0.5, & \text{for I, Channel, Double-Channel sections,} \\ 1.0, & \text{for T sections with flange in compression,} \\ 0.0, & \text{for T sections with flange in tension, and} \\
0.0, & \text{for I, Channel, Double-Channel sections,} \\ 0.8, & \text{for T sections with flange in compression, and} \\ -1.0, & \text{for T sections with flange in tension.} \end{cases} \text{ (BS B2.5d)} \]
• $\phi_b$ is the buckling index for box section factor. It is given by the following formula. (BS B2.6.1).

$$\phi_b = \left( \frac{S_{33}^2 \gamma'}{A^2 J} \right)^{1/2},$$

where

$$\gamma' = \left( 1 - \frac{I_{22}}{I_{33}} \right) \left( 1 - \frac{J}{2.6 I_{33}} \right).$$

(Bo S B2.6.1)

For all other sections, lateral torsional buckling is not considered. The user should investigate moment capacity considering lateral-torsional buckling separately.

**Shear Capacities**

The shear capacities for both the major and minor direction shears in I-shapes, boxes or channels are evaluated as follows:

$$P_{r2} = 0.6 \rho_y A_{r2},$$

and

$$P_{r3} = 0.6 \rho_y A_{r3}.$$

(BS 4.2.3)

The shear areas $A_{r3}$ and $A_{r2}$ are given in Table VIII-4.

Moreover, the shear capacity computed above is valid only if $d/t \leq 63 \varepsilon$, strictly speaking. For $d/t > 63 \varepsilon$, the shear buckling of the thin members should be checked independently by the user in accordance with the code (BS 4.4.5).

**Calculation of Capacity Ratios**

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

During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.
### Table VIII-4
*Shear Area in BS 5950*

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Condition</th>
<th>Axis of Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Major</td>
</tr>
<tr>
<td><strong>I-SHAPE</strong></td>
<td>Rolled</td>
<td>$tD$</td>
</tr>
<tr>
<td></td>
<td>Welded</td>
<td>$td$</td>
</tr>
<tr>
<td><strong>CHANNEL</strong></td>
<td>Rolled</td>
<td>$tD$</td>
</tr>
<tr>
<td></td>
<td>Welded</td>
<td>$td$</td>
</tr>
<tr>
<td><strong>DOUBLE CHANNEL</strong></td>
<td>Rolled</td>
<td>$2.0 \times tD$</td>
</tr>
<tr>
<td></td>
<td>Welded</td>
<td>$2.0 \times td$</td>
</tr>
<tr>
<td><strong>BOX</strong></td>
<td>—</td>
<td>$D$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$D + B$</td>
</tr>
<tr>
<td><strong>T-SHAPE</strong></td>
<td>Rolled</td>
<td>$td$</td>
</tr>
<tr>
<td></td>
<td>Welded</td>
<td>$t(d - T)$</td>
</tr>
<tr>
<td><strong>DOUBLE ANGLE</strong></td>
<td>—</td>
<td>$2td$</td>
</tr>
<tr>
<td><strong>ANGLE</strong></td>
<td>—</td>
<td>$td$</td>
</tr>
<tr>
<td><strong>RECTANGULAR</strong></td>
<td>—</td>
<td>$0.9 \times A$</td>
</tr>
<tr>
<td><strong>CIRCLE</strong></td>
<td>—</td>
<td>$0.9 \times A$</td>
</tr>
<tr>
<td><strong>PIPE</strong></td>
<td>—</td>
<td>$0.6 \times A$</td>
</tr>
<tr>
<td><strong>GENERAL</strong></td>
<td>—</td>
<td>$0.9 \times A$</td>
</tr>
</tbody>
</table>
Local Capacity Check

For members under axial load and moments, local capacity ratios are calculated as follows:

**Under Axial Tension**

A simplified approach allowed by the code is used to check the local capacity for plastic and compact sections.

\[
\frac{F_t}{A_g \rho_y} + \frac{M_{33}}{M_{c33}} + \frac{M_{22}}{M_{c22}} \quad \text{(BS 4.8.2)}
\]

**Under Axial Compression**

Similarly, the same simplified approach is used for axial compression.

\[
\frac{F_c}{A_g \rho_y} + \frac{M_{33}}{M_{c33}} + \frac{M_{22}}{M_{c22}} \quad \text{(BS 4.8.3.2)}
\]

**Overall Buckling Check**

In addition to local capacity checks, which are carried out at section level, a compression member with bending moments is also checked for overall buckling in accordance with the following interaction ratio:

\[
\frac{F_c}{A_g \rho_c} + \frac{m_3 M_{33}}{M_b} + \frac{m_{22} M_{22}}{\rho_c Z_{22}} \quad \text{(BS 4.8.3.3.1)}
\]

The equivalent uniform moment factor, \( m \), for members of uniform section and with flanges, not loaded between adjacent lateral restraints, is defined as

\[
m = 0.57 + 0.33\beta + 0.10\beta^2 \geq 0.43 \quad \text{(BS Table 18)}
\]

For other members, the value of \( m \) is taken as 1.0. The program defaults \( m \) to 1.0 if the unbraced length, \( l \), of the member is overwritten by the user (i.e. if it is not equal to the length of the member). The user can overwrite the value of \( m \) for any member by specifying it. \( \beta \) is the ratio of the smaller end moment to the larger end moment on a span equal to the unrestrained length, being positive for single curvature bending and negative for double curvature bending.
Shear Capacity Check

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

\[
\frac{F_{v_2}}{P_{v_2}}, \quad \text{and} \quad \frac{F_{v_3}}{P_{v_3}}.
\]
This chapter describes the details of the structural steel design and stress check algorithms that are used by ETABS when the user selects the Eurocode 3 design code (CEN 1992). The program investigates the limiting states of strength and stability but does not address the serviceability limit states. Various notations used in this chapter are described in Table IX-1.

The design is based on user-specified loading combinations. But the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures.

In the evaluation of the axial force/biaxial moment capacity ratios at a station along the length of the member, first the actual member force/moment components and the corresponding capacities are calculated for each load combination. Then the capacity ratios are evaluated at each station under the influence of all load combinations using the corresponding equations that are defined in this section. The controlling capacity ratio is then obtained. A capacity ratio greater than 1.0 indicates exceeding a limit state. Similarly, a shear capacity ratio is calculated separately.

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to Newton-Millimeter-Second units unless otherwise noted.
$A = \text{Gross cross-sectional area, } \text{mm}^2$

$A_{12}, A_{3} = \text{Areas for shear in the 2- and 3-directions, } \text{mm}^2$

$C_1 = \text{Bending coefficient}$

$E = \text{Modulus of elasticity, MPa}$

$G = \text{Shear modulus, MPa}$

$I_1 = \text{Torsion constant, } \text{mm}^4$

$I_w = \text{Warping constant, } \text{mm}^6$

$I_{33} = \text{Major moment of inertia, } \text{mm}^4$

$I_{22} = \text{Minor moment of inertia, } \text{mm}^4$

$K = \text{Effective length factor}$

$L = \text{Length, span, mm}$

$K_{33}, K_{22} = \text{Major and minor effective length factors}$

$M_{p, Rd} = \text{Design buckling resistance moment, N-mm}$

$M_{cr} = \text{Elastic critical moment for lateral-torsional buckling, N-mm}$

$M_{3, Sd} = \text{Design moments not causing sidesway}, \text{N-mm}$

$M_{2, Sd} = \text{Design moments causing sidesway, N-mm}$

$M_{2, Sd} = \text{Design moment resistance after considering shear, N-mm}$

$M_{22, Sd} = \text{Design value of moment about the major axis, N-mm}$

$M_{22, Rd} = \text{Design moment resistance about the major axis, N-mm}$

$M_{22, Rd} = \text{Design moment resistance about the minor axis, N-mm}$

$N_{c, Rd} = \text{Design buckling resistance of a compression member, N}$

$N_{c, Sd} = \text{Design value of compressive force, N}$

$N_{c, Rd} = \text{Design compression resistance, N}$

$N_{t, Rd} = \text{Design value of tensile force, N}$

$N_{t, Sd} = \text{Design tension resistance, N}$

$N_{p, Rd} = \text{Design plastic shear resistance, N}$

$N_{v, Rd} = \text{Design value of shear force in the major direction, N}$

$N_{v, Sd} = \text{Design shear resistance in the major direction, N}$

Table IX-1

Eurocode 3 Notations
Chapter IX  Check/Design for EUROCODE 3

$V_{3,rd}$ = Design shear resistance in the minor direction, N

$W_{el,33}, W_{el,22}$ = Major and minor elastic section moduli, mm$^3$

$W_{pl,33}, W_{pl,22}$ = Major and minor plastic section moduli, mm$^3$

$b$ = Width, mm

c = Distance, mm

d = Depth of web, mm

$f_y$ = Nominal yield strength of steel, MPa

$h$ = Overall depth, mm

$l_{33}, l_{22}$ = Major and minor direction unbraced member lengths, mm

$i_{33}, i_{22}$ = Major and minor radii of gyration, mm

$i_c$ = Minimum radius of gyration for angles, mm

$k_{33}, k_{22}$ = Factors applied to the major and minor design moments in the interaction equations

$k_{LT}$ = Factor applied to the major design moments in the interaction equation checking for failure due to lateral-torsional buckling

$t$ = Thickness, mm

$t_f$ = Flange thickness, mm

$t_w$ = Web thickness, mm

$\alpha$ = Ratio used in classification of sections

$\gamma_{M0}, \gamma_{M1}$ = Material partial safety factors

$\varepsilon = \left[ \frac{235}{f_y} \right]^{\frac{1}{2}} (f_y$ in MPa)

$\rho$ = Reduction factor

$\tau_{bu}$ = Post-critical shear strength, MPa

$\chi_{33}, \chi_{22}$ = Reduction factors for buckling about the 3-3 and 2-2 axes

$\chi_{LT}$ = Reduction factor for lateral-torsional buckling

$\psi$ = Ratio of smaller to larger end moment of unbraced segment

$\psi_s$ = Amplification factor for sway moments

Table IX-1

Eurocode 3 Notations (cont.)
Design Loading Combinations

The design loading combinations define the various factored combinations of the load cases for which the structure is to be checked. The design loading combinations are obtained by multiplying the characteristic loads with appropriate partial factors of safety. If a structure is subjected to dead load (DL) and live load (LL) only, the design will need only one loading combination, namely 1.35 DL + 1.5 LL.

However, in addition to the dead load and live load, if the structure is subjected to wind (WL) or earthquake induced forces (EL), and considering that wind and earthquake forces are subject to reversals, the following load combinations may have to be considered (EC3 2.3.3):

\[
\begin{align*}
1.35 \text{ DL} \\
1.35 \text{ DL} + 1.50 \text{ LL} & \quad (\text{EC3 2.3.3}) \\
1.35 \text{ DL} + 1.50 \text{ WL} \\
1.00 \text{ DL} + 1.50 \text{ WL} & \quad \text{ (EC3 2.3.3)} \\
1.35 \text{ DL} + 1.35 \text{ LL} + 1.35 \text{ WL} \\
1.00 \text{ DL} + 1.00 \text{ EL} \\
1.00 \text{ DL} + 1.5*0.3 \text{ LL} + 1.0 \text{ EL} & \quad (\text{EC3 2.3.3}) \\
\end{align*}
\]

In fact, these are the default load combinations which can be used or overwritten by the user to produce other critical design conditions. These default loading combinations are produced for persistent and transient design situations (EC3 2.3.2.2) by combining forces due to dead, live, wind, and earthquake loads for ultimate limit states. See also section 9.4 of Eurocode 1 (CEN 1994) and Table 1, 3, and 4 and section 4 of United Kingdom National Application Document (NAD).

The default load combinations will usually suffice for most building design. The user should use other appropriate loading combinations if roof live load is separately treated, other types of loads are present, or if pattern live loads are to be considered.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

In addition to the loads described earlier, equivalent lateral load cases for geometric imperfection should be considered by the user. This equivalent load is similar to the notional load of the British code, and depends on the number of stories and number of columns in any floor (EC3 5.2.4.3). Additional load combinations are also needed for these load cases.
When using Eurocode 3, ETABS design assumes that a P-Δ analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is suggested that the P-Δ analysis should be done at the factored load level corresponding to 1.35 dead load plus 1.35 live load. See also White and Hajjar (1991).

Classification of Sections

The design strength of a cross-section subject to compression due to moment and/or axial load depends on its classification as Class 1 (Plastic), Class 2 (Compact), Class 3 (Semi-compact), or Class 4 (Slender). According to Eurocode 3, the classification of sections depends on the classification of flange and web elements. The classification also depends on whether the compression elements are in pure compression, pure bending, or under the influence of combined axial force and bending (EC3 5.3.2).

ETABS conservatively classifies the compression elements according to Table IX-2 and Table IX-3. Table IX-2 is used when the section is under the influence of axial compression force only or combined axial compression force and bending. Table IX-3 is used when the section is in pure bending or under the influence of combined axial tensile force and bending. The section dimensions used in the tables are given in Figure IX-1. If the section dimensions satisfy the limits shown in the tables, the section is classified as Class 1, Class 2, or Class 3 as applicable. A cross-section is classified by reporting the highest (least favorable) class of its compression elements.

If a section fails to satisfy the limits for Class 3 sections, the section is classified as Class 4. Currently ETABS does not check stresses for Class 4 sections.

One of the major factors in determining the limiting width-thickness ratio is $\varepsilon$. This parameter is used to reflect the influence of yield stress on the section classification.

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

(EC3 5.3.2)

In classifying I, Box, Channel, Double-Channel, and T sections, two other factors $\alpha, \psi$ are defined as follows:
<table>
<thead>
<tr>
<th>Section</th>
<th>Element</th>
<th>Ratio Checked</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>web</td>
<td>$d/t_w$</td>
<td>If $\alpha &gt; 0.5$, $\frac{396\varepsilon}{(13\alpha - 1)}$, else if $\alpha \geq 0.5$, $\frac{36\varepsilon}{\alpha}$.</td>
<td>If $\alpha &gt; 0.5$, $\frac{456\varepsilon}{(13\alpha - 1)}$, else if $\alpha \geq 0.5$, $\frac{41.5\varepsilon}{\alpha}$.</td>
<td>If $\psi &gt; -1$, $\frac{42\varepsilon}{0.67 + 0.33\psi}$, else if $\psi \geq -1$, $\frac{62\varepsilon(1 - \psi)}{\sqrt{-\psi}}$.</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$c/t_f$ (rolled)</td>
<td>10 $\varepsilon$</td>
<td>11 $\varepsilon$</td>
<td>15 $\varepsilon$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$c/t_f$ (welded)</td>
<td>9 $\varepsilon$</td>
<td>10 $\varepsilon$</td>
<td>14 $\varepsilon$</td>
</tr>
<tr>
<td>BOX</td>
<td>web</td>
<td>$d/t_w$</td>
<td>Same as I-Shape</td>
<td>Same as I-Shape</td>
<td>Same as I-Shape</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$(b - 3t_f)/t_f$ (rolled)</td>
<td>42 $\varepsilon$</td>
<td>42 $\varepsilon$</td>
<td>42 $\varepsilon$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b/t_f$ (welded)</td>
<td>42 $\varepsilon$</td>
<td>42 $\varepsilon$</td>
<td>42 $\varepsilon$</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>web</td>
<td>$d/t_w$</td>
<td>Same as I-Shape</td>
<td>Same as I-Shape</td>
<td>Same as I-Shape</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$b/t_f$</td>
<td>10 $\varepsilon$</td>
<td>11 $\varepsilon$</td>
<td>15 $\varepsilon$</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>web</td>
<td>$d/t_w$</td>
<td>33 $\varepsilon$</td>
<td>38 $\varepsilon$</td>
<td>42 $\varepsilon$</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$h/2t_f$ (rolled)</td>
<td>10 $\varepsilon$</td>
<td>11 $\varepsilon$</td>
<td>15 $\varepsilon$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h/2t_f$ (welded)</td>
<td>9 $\varepsilon$</td>
<td>10 $\varepsilon$</td>
<td>14 $\varepsilon$</td>
</tr>
<tr>
<td>DOUBLE ANGLES</td>
<td>—</td>
<td>$h/t$ $(b + h)/[2\max(t,b)]$</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>15$\varepsilon$, 11.5$\varepsilon$</td>
</tr>
<tr>
<td>ANGLE</td>
<td>—</td>
<td>$h/t$ $(b + h)/[2\max(t,b)]$</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>15$\varepsilon$, 11.5$\varepsilon$</td>
</tr>
<tr>
<td>PIPE</td>
<td>—</td>
<td>$d/t$</td>
<td>50$\varepsilon^c$</td>
<td>70$\varepsilon^c$</td>
<td>90$\varepsilon^c$</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td>None</td>
<td>Assumed Class 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RECTANGLE</td>
<td>—</td>
<td>None</td>
<td>Assumed Class 2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table IX-2**

*Limiting Width-Thickness Ratios for Classification of Sections based on Eurocode 3 (Compression and Bending)*

174 Classification of Sections
<table>
<thead>
<tr>
<th>Section</th>
<th>Element</th>
<th>Ratio Checked</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE</td>
<td>web</td>
<td>$d/t_w$</td>
<td>72 ε</td>
<td>83 ε</td>
<td>124 ε</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$c/t_f$ (rolled)</td>
<td>10 ε</td>
<td>11 ε</td>
<td>15 ε</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$c/t_f$ (welded)</td>
<td>9 ε</td>
<td>10 ε</td>
<td>14 ε</td>
</tr>
<tr>
<td>BOX</td>
<td>web</td>
<td>$d/t_w$</td>
<td>72 ε</td>
<td>83 ε</td>
<td>124 ε</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$(b - 3t_f)/t_f$ (rolled)</td>
<td>33 ε</td>
<td>38 ε</td>
<td>42 ε</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b/t_f$ (welded)</td>
<td>33 ε</td>
<td>38 ε</td>
<td>42 ε</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>web</td>
<td>$d/t_w$ (Major axis)</td>
<td>72 ε</td>
<td>83 ε</td>
<td>124 ε</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d/t_w$ (Minor axis)</td>
<td>33 ε</td>
<td>38 ε</td>
<td>42 ε</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$b/t_f$</td>
<td>10 ε</td>
<td>11 ε</td>
<td>15 ε</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>web</td>
<td>$d/t_w$</td>
<td>33 ε</td>
<td>38 ε</td>
<td>42 ε</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>$b/2t_f$ (rolled)</td>
<td>10 ε</td>
<td>11 ε</td>
<td>15 ε</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b/2t_f$ (welded)</td>
<td>9 ε</td>
<td>10 ε</td>
<td>14 ε</td>
</tr>
<tr>
<td>DOUBLE ANGLES</td>
<td>—</td>
<td>$(b + h)/\left[2 \max(t, b)\right]$</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>15.0 ε</td>
</tr>
<tr>
<td>ANGLE</td>
<td>—</td>
<td>$h/t$</td>
<td>Not applicable</td>
<td>Not applicable</td>
<td>15.0 ε</td>
</tr>
<tr>
<td>PIPE</td>
<td>—</td>
<td>$d/t$</td>
<td>50ε²</td>
<td>70ε²</td>
<td>90ε²</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>—</td>
<td>None</td>
<td>Assumed Class 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RECTANGLE</td>
<td>—</td>
<td>None</td>
<td>Assumed Class 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GENERAL</td>
<td>—</td>
<td>None</td>
<td>Assumed Class 3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table IX-3
Limiting Width-Thickness Ratios for Classification of Sections based on Eurocode 3 (Bending Only)
EUROCODE 3: Axes Conventions

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

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

Figure IX-1
Eurocode 3 Definition of Geometric Properties
\[ \alpha = \begin{cases} \frac{1}{2} - \frac{1}{2} \frac{N_{c,SD}}{ht_w f_f}, & \text{for I, Channel, and T sections,} \\ \frac{1}{2} - \frac{1}{2} \frac{N_{c,SD}}{2ht_w f_f}, & \text{for Box and Double-Channel sections, and} \end{cases} \]

\[ \psi = \left( 1 + 2 \frac{N_{c,SD}}{A f_y} \right), \]

0 < \alpha \leq 1.0,

-3.0 < \psi \leq 1.0.

In the above expression, \( N_{c,SD} \) is taken as positive for tension and negative for compression. \( \alpha \) equals 0.0 for full tension, 0.5 for pure bending and 1.0 for full compression. \( \psi \) equals -3.0 for full tension, -1.0 for pure bending and 1.0 for full compression.

**Calculation of Factored Forces**

The internal design loads which are calculated for each load combination are \( N_{c,SD} \) or \( N_{c,SD} \), \( M_{33,SD} \), \( M_{22,SD} \), \( V_{2,SD} \) and \( V_{3,SD} \) corresponding to design values of the tensile or compressive axial load, the major moment, the minor moment, the major direction shear and the minor direction shear respectively. These design loads are calculated at each of the previously defined stations of each frame element.

The design moments and forces need to be corrected for second order effects. This correction is different for the so called “sway” and “nonsway” components of the moments. The code requires that the additional sway moments introduced by the horizontal deflection of the top of a story relative to the bottom must be taken into account in the elastic analysis of the frame in one of the following ways (EC3 5.2.6.2):

- Directly — by carrying out the global frame analysis using P-Δ analysis. Member design can be carried out using in-plane buckling lengths for nonsway mode.
- Indirectly — by modifying the results of a linear elastic analysis using an approximate method which makes allowance for the second order effects. There are two alternative ways to do this — “amplified sway moment method” or “sway mode in-plane buckling method”.

Calculation of Factored Forces
The advantage of the direct second order elastic analysis is that this method avoids uncertainty in approximating the buckling length and also avoids splitting up moments into their “sway” and “nonsway” components.

**ETABS design** assumes that P-Δ effects are included in the analysis. Therefore any magnification of sidesway moments due to second order effects is already accounted for, i.e. $\psi_s$ in the following equation is taken as 1.0. It is suggested that the P-Δ analysis be done at the factored load level of 1.35 DL plus 1.35 LL. See also White and Hajjar (1991). However, the user can overwrite the values of $\psi_s$ for both major and minor direction bending in which case $M_{Sd}$ in a particular direction is taken as:

$$M_{Sd} = M_{gSd} + \psi_s M_{Sd}$$

where

- $M_{gSd}$ = Design moments not causing translation, and
- $M_{Sd}$ = Design moments causing sidesway.

Moment magnification for non-sidesway moments is included in the overall buckling interaction equations.

Sway moments are produced in a frame by the action of any load which results in sway displacements. The horizontal loads can be expected always to produce sway moments. However, they are also produced by vertical loads if either the load or the frame are unsymmetrical. In the case of a symmetrical frame with symmetrical vertical loads, the sway moments are simply the internal moments in the frames due to the horizontal loads (EC3 5.2.6.2).

### Calculation of Section Resistances

The factored strengths in compression, tension, bending, and shear are computed for Class 1, 2, and 3 sections according to the following subsections. The material partial safety factors used by the program are:

$\gamma_{M0} = 1.1$, and $\gamma_{M1} = 1.1$.

For Class 4 (Slender) sections and any singly symmetric and unsymmetric sections requiring special treatment, such as the consideration of local buckling, flexural-torsional and torsional buckling, or web buckling, reduced section capacities may be applicable. The user must separately investigate this reduction if such elements are used.
If the user specifies nonzero factored strengths for one or more elements in the “Capacity Overwrites” form, these values will override the above mentioned calculated values for those elements.

Tension Capacity

The design tension resistance for all classes of sections is evaluated in ETABS as follows:

\[ N_{t,Rd} = A f'_{y} / \gamma_{M0} \]  

(EC3 5.4.3)

It should be noted that the design ultimate resistance of the net cross-section at the holes for fasteners is not computed and checked. The user is expected to investigate this independently.

Compression Resistance

The design compressive resistance of the cross-section is taken as the smaller of the design plastic resistance of the gross cross-section \( N_{pl,Rd} \) and the design local buckling resistance of the gross cross-section \( N_{b,Rd} \).

\[ N_{c,Rd} = \min (N_{pl,Rd}, N_{b,Rd}) \]  

(EC3 5.4.4)

The plastic resistance of Class 1, Class 2, and Class 3 sections is given by

\[ N_{pl,Rd} = A f'_{y} / \gamma_{M0} \]  

(EC3 5.4.4)

The design buckling resistance of a compression member is taken as

\[ N_{b,Rd} = \chi_{mn} \beta_{A} A f'_{y} / \gamma_{M1}, \text{ where} \]

\[ \beta_{A} = 1, \text{ for Class 1, 2 or 3 cross-sections.} \]

\( \chi \) is the reduction factor for the relevant buckling mode. This factor is calculated below based on the assumption that all members are of uniform cross-section.

\[ \chi = \frac{1}{\varphi + \left[ \varphi^2 - \overline{\lambda}^2 \right]^{1/2}} \leq 1, \text{ in which} \]

\[ \varphi = 0.5 \left[ 1 + \alpha \left( \overline{\lambda} - 0.2 \right) + \overline{\lambda}^2 \right], \]

where \( \overline{\lambda} \) is the characteristic slenderness ratio.
### Table IX-4

The $\alpha$ factor for different sections and different axes of buckling

<table>
<thead>
<tr>
<th>Section</th>
<th>Limits</th>
<th>$\alpha$ (major axis)</th>
<th>$\alpha$ (minor axis)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-SHAPE (rolled) $h/b &gt; 1.2$</td>
<td>$t_f \leq 40\text{ mm}$</td>
<td>0.21</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 40\text{ mm}$</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td>I-SHAPE (rolled) $h/b \leq 1.2$</td>
<td>$t_f \leq 100\text{ mm}$</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 100\text{ mm}$</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td>I-SHAPE (welded)</td>
<td>$t_f \leq 40\text{ mm}$</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 40\text{ mm}$</td>
<td>0.49</td>
<td>0.76</td>
</tr>
<tr>
<td>BOX</td>
<td>Rolled</td>
<td>0.21</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>welded</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>CHANNEL</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>T-SHAPE</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>DOUBLE ANGLES</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>ANGLE</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>PIPE</td>
<td>any</td>
<td>0.21</td>
<td>0.21</td>
</tr>
<tr>
<td>ROUND BAR</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>RECTANGLE</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>GENERAL</td>
<td>any</td>
<td>0.49</td>
<td>0.49</td>
</tr>
</tbody>
</table>
The two values of $\lambda$ give $\chi_3$ and $\chi_2$. $\chi_{mn}$ is the lesser of the two.

$K = \frac{l}{L} \leq 1$. $K$ is conservatively taken as 1 in ETABS design (EC3 5.5.1.5).

The user can, however, override this default option if it is deemed necessary. An accurate estimate of $K$ can be obtained from the Annex E of the code. See also EC3 5.2.6.2(2).

$l$ is the buckling length,

$L$ is the length of the column,

$i$ is the radius of gyration about the neutral axis, and is determined using the properties of the gross cross-section,

$\lambda_1 = \pi \left[ \frac{E}{f_y} \right]^{\frac{1}{2}},$ and

$\alpha$ is an imperfection factor and is obtained from Table IX-4. Values of this factor for different types of sections, axes of buckling, and thickness of materials are obtained from Tables 5.5.1 and 5.5.3 of the code.

Angle, Channel, and T-sections in compression are subjected to an additional moment due to the shift of the centroidal axis of the effective cross-section (EC3 5.4.4). ETABS does not currently consider this eccentricity. The user is expected to investigate this issue separately.

**Shear Capacity**

The design shear resistance of a section is the minimum of the plastic shear capacity and the buckling shear capacity. For all types of sections, the plastic shear resistance is computed as

$$V_{Rd} = V_{pl,Rd} = \frac{A_r f_y}{\sqrt{3}} \cdot \gamma_{M0}, \quad \text{(EC3 5.4.6)}$$
where $A_v$ is the effective shear area for the section and the appropriate axis of bending.

The buckling shear capacities are only computed for the I, Box, and Channel sections if the width-thickness ratio is large ($d/t_w > 69\varepsilon$). The capacities are computed as

$$V_{Rd} = V_{bu,Rd} = d \, t_w \, \tau_{ba} / \gamma_{M1}, \quad \text{(for } \frac{d}{t_w} > 69 \varepsilon \text{)} \quad \text{(EC3 5.6.3)}$$

where, $\tau_{ba}$ is the simple post-critical shear strength which is determined as follows:

$$\tau_{ba} = \frac{f_{yw}}{\sqrt{3}}, \quad \text{for } \overline{\lambda}_w \leq 0.8, \quad \text{(EC3 5.6.3)}$$

$$\tau_{ba} = \left[1 - 0.625(\overline{\lambda}_w - 0.8)\right] \frac{f_{yw}}{\sqrt{3}}, \quad \text{for } 0.8 < \overline{\lambda}_w < 1.2, \text{ and} \quad \text{(EC3 5.6.3)}$$

$$\tau_{ba} = \left[0.9 / \overline{\lambda}_w\right] \frac{f_{yw}}{\sqrt{3}}, \quad \text{for } \overline{\lambda}_w \geq 1.2. \quad \text{(EC3 5.6.3)}$$

in which $\overline{\lambda}_w$ is the web slenderness ratio,

$$\overline{\lambda}_w = \frac{d}{37.4 \varepsilon \sqrt{k_\tau}}, \text{ and} \quad \text{(EC3 5.6.3)}$$

$k_\tau$ is the buckling factor for shear. For webs with transverse stiffeners at the supports but no intermediate transverse stiffeners,

$$k_\tau = 5.34. \quad \text{(EC3 5.6.3)}$$

**Moment Resistance**

The moment resistance in the major and minor directions is based on the section classification. Moment capacity is also influenced by the presence of shear force and axial force at the cross section. If the shear force is less than half of the shear capacity, the moment capacity is almost unaffected by the presence of shear force. If the shear force is greater than half of the shear capacity, additional factors need to be considered.

If $V_{sd} \leq 0.5V_{pl,Rd}$

- For Class 1 and Class 2 Sections

$$M_{c,Rd} = M_{pl,Rd} = W_{pl} \int_y / \gamma_{M0}. \quad \text{(EC3 5.4.5.2)}$$
For Class 3 Sections

\[ M_{c,Rd} = M_{el,Rd} = W_{el} \int_y / \gamma_{M1} \]  

(EC3 5.4.5.2)

If \( V_{sd} > 0.5V_{pl,Rd} \)

- For I, Box, and Channel sections bending about the 3-3 axis the moment capacities considering the effects of shear force are computed as

\[ M_{V,Rd} = \left[ W_{pl} - \frac{\rho A_w^2}{4t_w} \right] \frac{f_y}{\gamma_{M0}} \leq M_{c,Rd} \, , \text{ where} \]  

(EC3 5.4.7)

\[ \rho = \left[ 2 \frac{V_{sd}}{V_{pl,Rd}} - 1 \right] \]

- For all other cases, the reduction of moment capacities for the presence of shear force is not considered. The user should investigate the reduced moment capacity separately.

**Lateral-torsional Buckling**

For the determination of lateral-torsional buckling resistance, it is assumed that the section is uniform, doubly symmetric, and loaded through its shear center. The lateral-torsional buckling resistance of I, Box, and Double Channel sections is evaluated as,

\[ M_{b,Rd} = \chi_{LT} \beta_w W_{pl,33} f_y / \gamma_{M1} \, , \text{ where} \]  

(EC3 5.5.2)

\[ \beta_w = 1 \, , \text{ for Class 1 and Class 2 sections,} \]

\[ \beta_w = \frac{W_{el,33}}{W_{pl,33}} \, , \text{ for Class 3 sections,} \]

\[ \chi_{LT} = \frac{1}{\phi_{LT} + \left[ \phi_{LT}^2 - \chi_{LT}^2 \right]^{\frac{1}{2}}} \leq 1 \, , \text{ in which} \]

\[ \phi_{LT} = 0.5 \left[ 1 + \alpha_{LT} \left( \chi_{LT} - 0.2 \right) + \chi_{LT}^2 \right] \, , \text{ where} \]

\[ \alpha_{LT} = 0.21 \, , \text{ for rolled sections,} \]

\[ \alpha_{LT} = 0.49 \, , \text{ for welded sections, and} \]
\[ \bar{x}_{LT} = \left[ \frac{\beta_w W_{pl,33} f_y}{M_{cr}} \right]^{0.5}, \] where

\[ M_{cr} = C_1 \frac{\pi^2 E I_{22}}{L^2} \left[ \frac{I_w}{I_{22}} + \frac{L^2 G I_1}{\pi^2 EI_{22}} \right]^{0.5}, \] (EC3 F1.1)

\[ I_w = \text{The torsion constant}, \]
\[ I_w = \text{The warping constant}, \]
\[ L = \text{Laterally unbraced length for buckling about the minor axis. It is taken as } l_{22}, \]
\[ C_1 = 1.88 - 1.40 \psi + 0.52 \psi^2 \leq 2.7, \text{ and } \]
\[ \psi = \text{The ratio of smaller to larger end moment of unbraced segment, } \frac{M_a}{M_b}. \]

\( \psi \) varies between -1 and 1 \((-1 \leq \psi \leq 1)\). A negative value implies double curvature. \( M_a \) and \( M_b \) are end moments of the unbraced segment and \( M_a \) is less than \( M_b \), \( \left( \frac{M_a}{M_b} \right) \) being negative for double curvature bending and positive for single curvature bending. If any moment within the segment is greater than \( M_b \), \( C_1 \) is taken as 1.0. The program defaults \( C_1 \) to 1.0 if the unbraced length, \( l_{22} \) of the member is overwritten by the user (i.e. it is not equal to the length of the member). \( C_1 \) should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. The user can overwrite the value of \( C_1 \) for any member by specifying it.

If \( \bar{x}_{LT} \leq 0.4 \), no special consideration for lateral torsional buckling is made in the design.

The lateral-torsional buckling resistance of a Channel, T, Angle, Double-Angle and General sections is evaluated as,

\[ M_{b, Rd} = \frac{W_{el,33} f_y}{\gamma_{M1}} \]

and the lateral-torsional buckling resistance of Rectangle, Circle and Pipe sections is evaluated as,

\[ M_{b, Rd} = \frac{W_{pl,33} f_y}{\gamma_{M1}} \]
Currently ETABS does not consider other special considerations for computing buckling resistance of Rectangle, Circle, Pipe, Channel, T, Angle, Double Angle and General sections.

**Calculation of Capacity Ratios**

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

During the design, the effect of the presence of bolts or welds is not considered. Also, the joints are not designed.

**Bending, Axial Compression, and Low Shear**

When the design value of the coexisting shear, $V_{sd}$, is less than half of the corresponding capacities for plastic resistance, $V_{pl,Rd}$, and buckling resistance, $V_{ba,Rd}$, i.e.

\[
V_{sd} \leq 0.5V_{pl,Rd}, \quad \text{and} \quad V_{sd} \leq 0.5V_{ba,Rd},
\]

the capacity ratios are computed for different types of sections as follows:

For Class 1 and Class 2 sections, the capacity ratio is conservatively taken as

\[
\frac{N_{c,Sd}}{N_{pl,Rd}} + \frac{M_{33,Sd}}{M_{pl,33,Rd}} + \frac{M_{22,Sd}}{M_{pl,22,Rd}}.
\]  

For Class 3 sections, the capacity ratio is conservatively taken as

\[
\frac{N_{c,Sd}}{Af_{yd}} + \frac{M_{33,Sd}}{W_{el,33}f_{yd}} + \frac{M_{22,Sd}}{W_{el,22}f_{yd}}, \quad \text{where}
\]

\[
f_{yd} = \frac{f_y}{\gamma_{M,0}}.
\]
**Bending, Axial Compression, and High Shear**

When the design value of the coexisting shear, \( V_{sd} \), is more than half the corresponding capacities for plastic resistance, \( V_{pl,Rd} \), or buckling resistance, \( V_{bu,Rd} \), the shear is considered to be high, i.e. the shear is high if

\[
V_{sd} > 0.5 \cdot V_{pl,Rd} \quad \text{or} \quad V_{sd} > 0.5 \cdot V_{bu,Rd} \tag{EC3 5.4.9}
\]

Under these conditions, the capacity ratios are computed for different types of sections as follows (EC3 5.4.9):

For Class 1, 2, and 3 sections, the capacity ratio is conservatively taken as

\[
\frac{N}{N_{pl,Rd}} + \frac{M_{33,Sd}}{M_{33,Rd}} + \frac{M_{22,Sd}}{M_{22,Rd}} \quad \text{where} \quad (EC3 5.4.8.1)
\]

\( M_{33,Rd} \) and \( M_{22,Rd} \) are the design moment resistances about the major and the minor axes, respectively, considering the effect of high shear (see page 182).

**Bending, Compression, and Flexural Buckling**

For all members of Class 1, 2, and 3 sections subject to axial compression, \( N_{sd} \), major axis bending, \( M_{33,Sd} \), and minor axis bending, \( M_{22,Sd} \), the capacity ratio is given by

\[
\frac{N}{N_{b,\min,Rd}} + \frac{k_{33} \cdot M_{33,Sd}}{\eta M_{c,33,Rd}} + \frac{k_{22} \cdot M_{22,Sd}}{\eta M_{c,22,Rd}} \quad \text{where} \quad (EC3 5.5.4)
\]

\[
N_{b,\min,Rd} = \min \{ N_{b,33,Rd}, N_{b,22,Rd} \},
\]

\[
\eta = \frac{\gamma M_{0}}{\gamma M_{1}}
\]

\[
k_{33} = 1 - \frac{\mu_{33} \cdot N_{c,Sd}}{\chi_{33} \cdot A f_{y}} \leq 1.5,
\]

\[
k_{22} = 1 - \frac{\mu_{22} \cdot N_{c,Sd}}{\chi_{22} \cdot A f_{y}} \leq 1.5,
\]
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\[
\mu_{33} = \bar{\lambda}_{33} (2 \beta_{M,33} - 4) + \left[ \frac{W_{pl,33} - W_{el,33}}{W_{el,33}} \right] \leq 0.9, \quad \text{(Class 1 and Class 2)},
\]

\[
\mu_{22} = \bar{\lambda}_{22} (2 \beta_{M,22} - 4) + \left[ \frac{W_{pl,22} - W_{el,22}}{W_{el,22}} \right] \leq 0.9, \quad \text{(Class 1 and Class 2)},
\]

\[
\mu_{33} = \bar{\lambda}_{33} (2 \beta_{M,33} - 4) \leq 0.9, \quad \text{(for Class 3 sections)},
\]

\[
\mu_{22} = \bar{\lambda}_{22} (2 \beta_{M,22} - 4) \leq 0.9, \quad \text{(for Class 3 sections)},
\]

\( \beta_{M,33} \) = Equivalent uniform moment factor for flexural buckling about the 3-3 (major) axis between points braced in 2-2 direction, and

\( \beta_{M,22} \) = Equivalent uniform moment factor for flexural buckling about the 2-2 (minor) axis between points braced in 3-3 direction.

The equivalent uniform moment factors, \( \beta_{M,33} \) and \( \beta_{M,22} \), are determined from

\[
\beta_M = (1.8 - 0.7 \psi) + \frac{M_Q}{\Delta M} (0.7 \psi - 0.5), \quad \text{and}
\]

\( M_Q \) = Absolute maximum moment due to lateral load only assuming simple support at the ends,

\( \psi \) = Absolute value of the ratio of smaller to larger end moment.

\( \psi \) varies between -1 and 1 \((-1 \leq \psi \leq 1)\). A negative value implies double curvature.

\( \Delta M \) = Absolute maximum value of moment for moment diagram without change of sign, and

\( \Delta M \) = Sum of absolute maximum and absolute minimum value of moments for moment diagram with change of sign.

**Bending, Compression, and Lateral-Torsional Buckling**

For all members of Class 1, 2, and 3 sections subject to axial compression, \( N_{Sd} \), major axis bending, \( M_{33,Sd} \), and minor axis bending, \( M_{22,Sd} \), the capacity ratio is given by

\[
\frac{N_{c,Sd}}{N_{b,22,Rd}} + \frac{k_{LT} M_{33,Sd}}{M_{b,Rd}} + \frac{k_{22} M_{22,Sd}}{\eta M_{c,22,Rd}}, \quad \text{where}
\]

(EC3 5.5.4)
$k_{22}$ and $\eta$ are as defined in the previous subsection “Bending, Compression, and Flexural Buckling”,

$$k_{LT} = 1 - \frac{\mu_{LT} N_{c,Sd}}{\chi_{22} \Lambda f_y} \leq 1,$$

where

$$\mu_{LT} = 0.15 \bar{\kappa}_{22} \beta_{M,LT} - 0.15 \leq 0.9,$$

and

$$\beta_{M,LT} = \text{Equivalent uniform moment factor for lateral-torsional buckling. It is determined for bending about the y-y axis and between two points braced in the y-y direction.}$$

### Bending, Axial Tension, and Low Shear

When the design value of the coexisting shear, $V_{Sd}$, is less than half of the corresponding capacities for plastic resistance, $V_{pl,Rd}$, and buckling resistance, $V_{ba,Rd}$, i.e.

$$V_{Sd} \leq 0.5V_{pl,Rd}, \text{ and}$$

$$V_{Sd} \leq 0.5V_{ba,Rd},$$

the capacity ratios are computed for different types of sections as follows:

For Class 1 and Class 2 sections, the capacity ratio is conservatively taken as

$$\frac{N_{Sd}}{N_{Rd}} + \frac{M_{33,Sd}}{M_{33,Rd}} + \frac{M_{22,Sd}}{M_{22,Rd}}. \quad \text{(EC3 5.4.8.1)}$$

For Class 3 sections, the capacity ratio is conservatively taken as

$$\frac{N_{Sd}}{4f_{yd}} + \frac{M_{33,Sd}}{W_{el,33}f_{yd}} + \frac{M_{22,Sd}}{W_{el,22}f_{yd}}. \quad \text{(EC3 5.4.8.1)}$$

### Bending, Axial Tension, and High Shear

When the design values of the coexisting shear, $V_{Sd}$, is more than half the corresponding capacities for plastic resistance, $V_{pl,Rd}$ or buckling resistance, $V_{ba,Rd}$, the shear is considered to be high, i.e. the shear is high if

$$V_{Sd} > 0.5V_{pl,Rd}, \text{ or}$$

$$V_{Sd} > 0.5V_{ba,Rd}.$$
Under these conditions, the capacity ratios are computed for different types of sections as follows (EC3 5.4.9):

For Class 1, 2, and 3 sections, the capacity ratio is conservatively taken as

\[ \frac{N_{t,ld}}{N_{t,Rd}} + \frac{M_{33,ld}}{M_{33,Rd}} + \frac{M_{22,ld}}{M_{22,Rd}} \]  

(EC3 5.4.8.1)

### Bending, Axial Tension, and Lateral-Torsional Buckling

The axial tensile force has a beneficial effect for lateral-torsional buckling. In order to check whether the member fails under lateral-torsional buckling, the effective internal moment about the 3-3 axis is calculated as follows:

\[ M_{eff,33,ld} = M_{33,ld} - \psi_{vec} \frac{N_{t,ld} W_{com,33}}{A}, \]  

where (EC3 5.5.3)

\[ \psi_{vec} = 0.8 \text{ (according to the EC3 box value), and} \]

\[ W_{com,33} \] is the elastic section modulus for the extreme compression fiber.

For all members of Class 1, 2, and 3 sections subject to axial tension, \( N_{t,ld} \), major axis bending, \( M_{33,ld} \), and minor axis bending, \( M_{22,ld} \), the capacity ratio is taken as

\[ \frac{N_{t,ld}}{N_{t,Rd}} + k_{LT} \frac{M_{33,ld}}{M_{b,Rd}} + k_{22} \frac{M_{22,ld}}{M_{c,ld}} - \psi_{vec} k_{LT} \frac{N_{t,ld} W_{com,33}}{A M_{b,Rd}}, \]  

(EC3 5.5.4)

where \( k_{LT}, k_{22} \) and \( \eta \) are as defined in the previous subsections.

### Shear

From the design values of shear force at each station, for each of the load combinations and the shear resistance values, shear capacity ratios for major and minor directions are produced as follows:

\[ \frac{V_{2,ld}}{V_{2,Rd}} \text{ and } \frac{V_{3,ld}}{V_{3,Rd}} \]
Overview

ETABS creates design output in three different major formats: graphical display, tabular output, and member-specific detailed design information.

The graphical display of steel design output includes input and output design information. Input design information includes design section labels, $K$-factors, live load reduction factors, and other design parameters. The output design information includes axial and bending interaction ratios and shear stress ratios. All graphical output can be printed.

The tabular output can be saved in a file or printed directly. The tabular output includes most of the information which can be displayed. This is generated for added convenience to the designer.

The member-specific detailed design information shows the details of the calculation. It shows the design section dimensions, material properties, design and allowable stresses or factored and nominal strengths, and some intermediate results for all the load combinations at all the design sections of a specific frame member.

In the following sections, some of the typical graphical display, tabular output, and member-specific detailed design information are described. Some of the design in-

Overview
formation is specific to the chosen steel design codes which are available in the program. The AISC-ASD89 design code is described in the latter part of this chapter. For all other codes, the design outputs are similar.

Graphical Display of Design Input and Output

The graphical output can be produced as screen display. Moreover, the active screen display can be sent directly to the printer. The graphical display of design output includes input and output design information.

Input design information, for the AISC-ASD89 code, includes

- Design section labels,
- Framing type,
- Live Load Reduction Factors,
- Unbraced Length Ratios for major and minor directions of bending,
- $K$-factors for major and minor directions of buckling,
- $C_m$-factors for major and minor directions,
- $C_b$-factors,
- Axial allowable stresses,
- Allowable stresses in flexure, and
- Allowable stresses in shear.

The output design information which can be displayed is

- Color coded P-M interaction ratios with or without values, and
- Color coded shear stress ratios.

The graphical displays can be accessed from the Design menu. For example, the color coded P-M interaction ratios with values can be displayed by selecting the Design menu > Steel Frame Design > Display Design Info command. This will pop up a dialog box called Display Design Results. Then the user should switch on the Design Output option button (default) and select P-M Ratios Colors & Values in the drop-down box. Then clicking the OK button will show the interaction ratios in the active window.

The graphics can be displayed in either 3D or 2D mode. The ETABS standard view transformations are available for all steel design input and output displays. For switching between 3D or 2D view of graphical displays, there are several buttons
on the main toolbar. Alternatively, the view can be set by choosing Set 3D View, Set Plan View, or Set Elevation View from the View menu.

The graphical display in an active window can be printed in gray scaled black and white from the ETABS program. To send the graphical output directly to the printer, click on the Print Graphics button in the File menu. A screen capture of the active window can also be made by following the standard procedure provided by the Windows operating system.

Tabular Display of Design Input and Output

The tabular design output can be sent directly either to a printer or to a file. The printed form of tabular output is the same as that produced for the file output with the exception that for the printed output font size is adjusted.

The tabular design output includes input and output design information which depends on the design code of choice. For the AISC-ASD89 code, the tabular output includes the following. All tables have formal headings and are self-explanatory, so further description of these tables is not given.

Input design information includes the following:

- Material Analysis Property Data
  - Material label,
  - Modulus of elasticity,
  - Poisson’s ratio,
  - Coefficient of thermal expansion,
  - Weight per unit volume, and
  - Mass per unit volume.

- Material Design Property Data
  - Material label,
  - Governing design code (Steel or Concrete),
  - Yield strength.

- Frame Section Property Data (Referenced sections only)
  - Section label,
  - Associated material label,
  - Section name,
Section geometric properties (depth, web thickness, top flange width, top flange thickness, bottom flange width, bottom flange thickness), and
Section gross property (area, major and minor shear areas, major and minor shear moment of inertia, torsional inertia, major and minor section Moduli, major and minor plastic Moduli, major and minor radii of gyration).

• Load Combination Multipliers
  - Combination name,
  - Combination type,
  - Load factors,
  - Load types, and
  - Combination title.

• Beam or Column Steel Stress Check Element Information (code dependent)
  - Story level,
  - Beam bay or Column line,
  - Design Section ID,
  - Framing type,
  - Live Load Reduction Factors,
  - Unbraced Length Ratios, and
  - $K$-factors for major and minor direction of buckling.

The output design information includes the following:

• Beam or Column Steel Stress Check Output (code dependent)
  - Story level,
  - Beam bay or Column line,
  - Design Section ID,
  - Controlling load combination ID for P-M interaction,
  - Tension or compression indication ("T" or "C"),
  - Axial and bending interaction ratio with breakdown into axial, and major and minor bending,
  - Controlling load combination ID for major and minor shear forces, and
  - Shear stress ratios, and
  - Occasional warning messages.
The tabular output can be accessed by selecting the **File** menu > **Print Tables** > **Steel Frame Design** command. This will pop up a dialog box. The design information has been grouped into four categories: Preferences, Input Summary, Output Summary, and Detailed Output. The user can specify the design quantities for which the results are to be tabulated by checking the associated check boxes. By default, the output will be sent to the printer. If the user wants the output stream to be redirected to a file, he/she can check the **Print to File** box. This will provide a default filename. The default filename can be edited. Alternatively, a file list can be obtained by clicking the **File Name** button to choose a file from. If the user wants the output table to be appended to the existing text file, he/she should select the file from the file list and check the Append box. Then clicking the **OK** button will direct the tabular output to the requested file or to the requested printer.

For easy review of the file in which the tabular information has just been saved, the program provides an easy access to a text editor through the **File > Display Input/Output Text Files** command.

### Member Specific Information

The member specific design information shows the details of the calculation. It provides an access to the geometry and material input data, design section dimensions, design and allowable stresses, stress ratios, and some of the intermediate results for a member. The design detail information can be displayed for a specific load combination and for a specific station of a frame member.

The detailed design information can be accessed by **right clicking** on the desired frame member. This will pop up a dialog box called **Steel Stress Check Information** which includes the following tabulated information for the specific member.

- Story level,
- Beam bay or Column line,
- Analysis Section ID,
- Design Section ID,
- Load combination ID,
- Station location,
- Axial and bending interaction ratio, and
- Shear stress ratio along two axes.
Additional information can be accessed by clicking on the **Overwrites** and **Details** buttons in the dialog box. Additional information that is available by clicking on the **Overwrites** button is as follows:

- Current Design Section ID,
- Element Framing Type,
- Live Load Reduction Factor,
- Horizontal Earthquake Factor,
- Design Parameters (code dependent)
  - Unbraced Length Ratios for major and minor directions,
  - Effective length factors, \( K \), for major and minor directions of buckling,
  - \( C_m \)-factors for major and minor directions,
  - \( C_b \)-factors,
  - \( \delta_m \)-factors for major and minor directions,
  - \( \delta_b \)-factors for major and minor directions,
  - Yield stress,
  - \( \Omega_0 \)-factors,
  - Compressive and tensile allowable stresses,
  - Major and minor bending allowable stresses, and
  - Major and minor shear allowable stresses.

Additional information that is available by clicking on the **Details** button is given below.

- Design code name, Units,
- Story, Beam bay or Column line, Station, Section, and Element type,
- Section geometric information and graphical representation,
- Material properties of steel,
- Warning information,
- Load Combination ID,
- Moment and forces,
- Demand/Capacity ratios,
- Design and allowable stresses for axial force and biaxial moments, and
- Design and allowable stresses for shear.
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