SAP2000®

Integrated
Finite Element Analysis
and
Design of Structures

STEEL DESIGN MANUAL

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Berkeley, California, USA

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Table of Contents

Chapter I

Introduction

Overview

SAP2000 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 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 (1994),
- U.S. AASHTO LRFD (1997),

- Canadian CAN/CSA-S16.1-94 (1994),
- British BS 5950 (1990), and
- Eurocode 3 (ENV 1993-1-1).

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 SAP2000. If the default load combinations are acceptable, no definition of additional load combination is required.

In the design 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.

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 overstress. Backup design information produced by the program is also provided for convenient verification of the results.

Special requirements for seismic design are not implemented in the current version of SAP2000.

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 SAP2000 program. This chapter describes the common terminology of steel design as implemented in SAP2000.

Each of six subsequent chapters gives a detailed description of a specific code of practice as interpreted by and implemented in SAP2000. Each chapter describes the design loading combinations to be considered; allowable stress or capacity calculations for tension, compression, bending, and shear; calculations of demand/capacity ratios; and other special considerations required by the code.

- Chapter III gives a detailed description of the AISC ASD code (AISC 1989) as implemented in SAP2000.
- Chapter IV gives a detailed description of the AISC LRFD code (AISC 1994) as implemented in SAP2000.
- Chapter V gives a detailed description of the AASHTO LRFD steel code (AASHTO 1997) as implemented in SAP2000.
- Chapter VI gives a detailed description of the Canadian code (CISC 1994) as implemented in SAP2000.
- Chapter VII gives a detailed description of the British code BS 5950 (BSI 1990) as implemented in SAP2000.
- Chapter VIII gives a detailed description of the Eurocode 3 (CEN 1992) as implemented in SAP2000.

Chapter IX outlines various aspects of the tabular and graphical output from SAP2000 related to steel design.

Recommended Reading

It is recommended that the user read Chapter II “Design Algorithms” and one of six subsequent chapters corresponding to the code of interest to the user. Finally the user should read “Design Output” in Chapter IX for understanding and interpreting SAP2000 output related to steel design.

A steel design tutorial is presented in the chapter “Steel Design Tutorial” in the *SAP2000 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 SAP2000 program. The steel design and check may be performed according to one of the following codes of practice.

- American Institute of Steel Construction's "Allowable Stress Design and Plastic Design Specification for Structural Steel Buildings", **AISC-ASD** (AISC 1989).
- American Institute of Steel Construction's "Load and Resistance Factor Design Specification for Structural Steel Buildings", **AISC-LRFD** (AISC 1994).
- American Association of State Highway and Transportation Officials' "AASHTO-LRFD Bridge Design Specifications", **AASHTO-LRFD** (AASHTO 1997).
- Canadian Institute of Steel Construction's "Limit States Design of Steel Structures", **CAN/CSA-S16.1-94** (CISC 1995).
- British Standards Institution's "Structural Use of Steelwork in Building", **BS 5950** (BSI 1990).
- European Committee for Standardization's "Eurocode 3: Design of Steel Structures C Part 1.1: General Rules and Rules for Buildings", **ENV 1993-1-1** (CEN 1992).

Details of the algorithms associated with each of these codes as implemented and interpreted in SAP2000 are described in subsequent chapters. However, this chapter provides a background which is common to all the design codes.

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.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code. For example, all references to the AASHTO-LRFD code carry the prefix of “**AASHTO**”. Similarly,

– 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 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**”

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 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. The number of segments in an element is requested by the user before the analysis is made. The user can refine the design along the length of an element by requesting more segments.

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.

P-Δ Effects

The SAP2000 design algorithms require that the analysis results include the P-Δ effects. The P-Δ effects are considered differently for “braced” or “nonsway” 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 SAP2000, it is assumed that “braced” or “nonsway” 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 AASHTO-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 code.

For lateral drift effects of unbraced or sway frames, SAP2000 assumes that the amplification is already included in the results because P-Δ effects are considered for all but AISC-ASD code.

The users of SAP2000 should be aware that the default analysis option in SAP2000 is turned OFF for P-Δ effect. 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 code. For further reference, the user is referred to *SAP2000 Analysis Reference Manual* (CSI 1997).

The user is also cautioned that SAP2000 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 SAP2000 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.

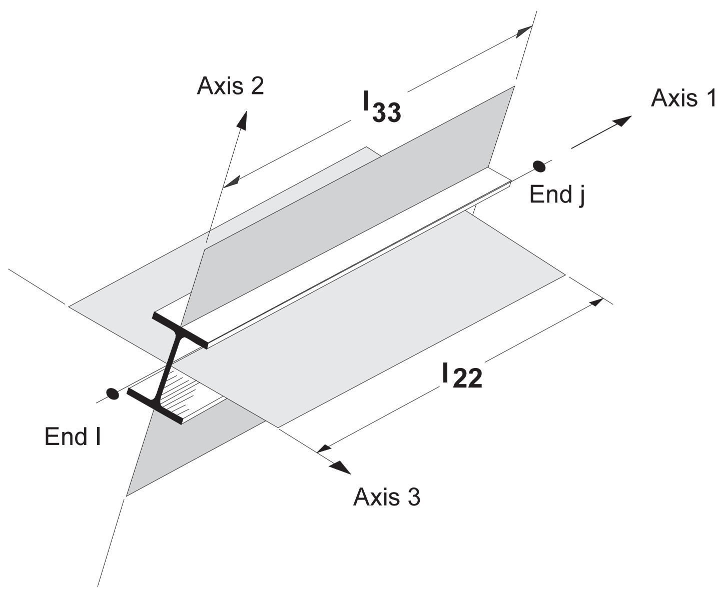


Figure II-1
Major and Minor Axes of Bending

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.

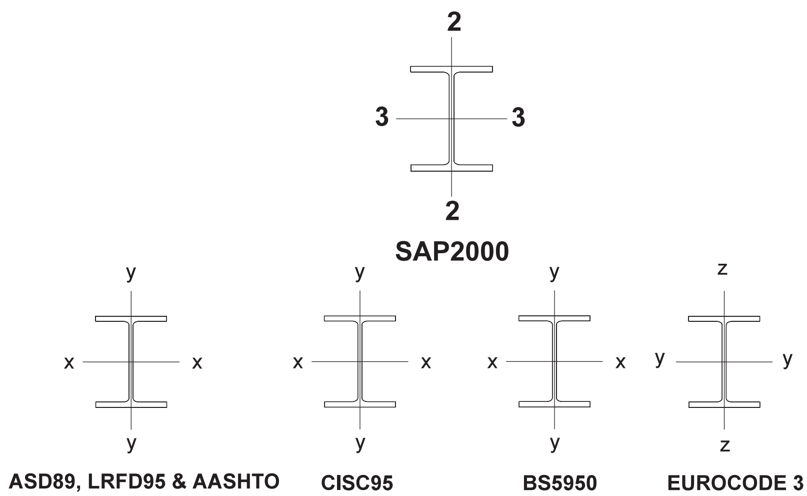


Figure II-2
Correspondence between SAP2000 Axes and Code Axes

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.

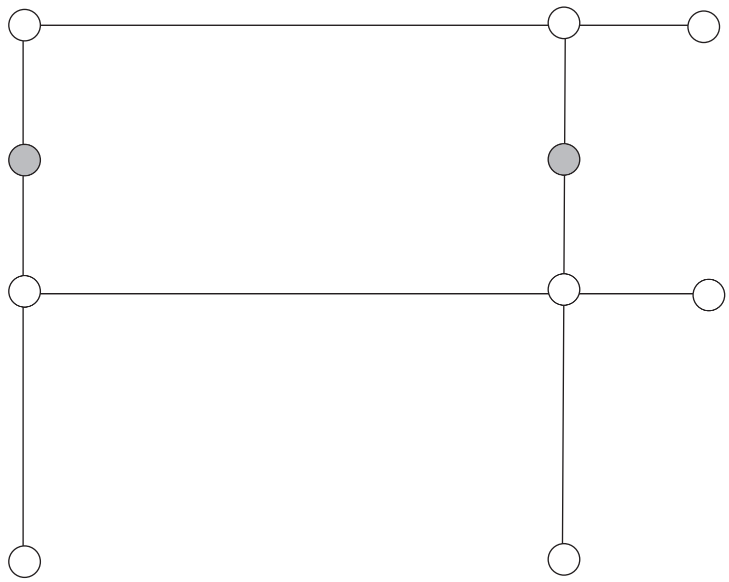


Figure II-3
Unsupported Lengths are Affected by Intermediate Nodal Points

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:

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

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

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

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

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

If a rotational release exists at a particular end (and direction) of an 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 α:

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

from which $K = \pi / \alpha$. This relationship is the mathematical formulation for the evaluation of *K* factors for moment-resisting frames assuming sidesway to be uninhibited. For other structures, such as braced frame structures, trusses, space frames, transmission towers, etc., 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.

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 SAP2000.

Chapter III

Check/Design for AISC-ASD89

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 when the user selects the AISC-ASD89 design code (AISC 1989). 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” 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.

A	=	Cross-sectional area, in^2
A_e	=	Effective cross-sectional area for slender sections, in^2
A_f	=	Area of flange , in^2
A_g	=	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	=	Major and minor shear areas, in^2
A_w	=	Web shear area, dt_w, in^2
C_b	=	Bending Coefficient
C_m	=	Moment Coefficient
C_w	=	Warping constant, in^6
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
F_a	=	Allowable axial stress, ksi
F_b	=	Allowable bending stress, ksi
F_{b33}, F_{b22}	=	Allowable major and minor bending stresses, ksi
F_{cr}	=	Critical compressive stress, ksi
F'_{e33}	=	$\dfrac{12\pi^2 E}{23(K_{33}l_{33}/r_{33})^2}$
F'_{e22}	=	$\dfrac{12\pi^2 E}{23(K_{22}l_{22}/r_{22})^2}$
F_v	=	Allowable shear stress, ksi
F_y	=	Yield stress of material, ksi
K	=	Effective length factor
K_{33}, K_{22}	=	Effective length K-factors in the major and minor directions
M_{33}, M_{22}	=	Major and minor bending moments in member, kip-in
M_{ob}	=	Lateral-torsional moment for angle sections, kip-in
P	=	Axial force in member, kips
P_e	=	Euler buckling load, kips
Q	=	Reduction factor for slender section, = $Q_a Q_s$
Q_a	=	Reduction factor for stiffened slender elements
Q_s	=	Reduction factor for unstiffened slender elements
S	=	Section modulus, in^3
S_{33}, S_{22}	=	Major and minor section moduli, in^3

Table III-1
AISC-ASD Notations

$S_{eff,33}, S_{eff,22}$	=	Effective major and minor section moduli for slender sections, in^3
S_c	=	Section modulus for compression in an angle section, in^3
V_2, V_3	=	Shear forces in major and minor directions, kips
b	=	Nominal dimension of plate in a section, in longer leg of angle sections, $b_f - 2t_w$ for welded and $b_f - 3t_w$ for rolled box sections, etc.
b_e	=	Effective width of flange, in
b_f	=	Flange width, in
d	=	Overall depth of member, in
f_a	=	Axial stress either in compression or in tension, ksi
f_b	=	Normal stress in bending, ksi
f_{b33}, f_{b22}	=	Normal stress in major and minor direction bending, ksi
f_v	=	Shear stress, ksi
f_{v2}, f_{v3}	=	Shear stress in major and minor direction bending, ksi
h	=	Clear distance between flanges for I shaped sections ($d - 2t_f$), in
h_e	=	Effective distance between flanges less fillets, in
k	=	Distance from outer face of flange to web toe of fillet , in
k_c	=	Parameter used for classification of sections, $\frac{4.05}{[h/t_w]^{0.46}}$ if $h/t_w > 70$, 1 if $h/t_w \leq 70$.
l_{33}, l_{22}	=	Major and minor direction unbraced member lengths, in
l_c	=	Critical length, in
r	=	Radius of gyration, in
r_{33}, r_{22}	=	Radii of gyration in the major and minor directions, in
r_z	=	Minimum Radius of gyration for angles, in
t	=	Thickness of a plate in I, box, channel, angle, and T sections, in
t_f	=	Flange thickness, in
t_w	=	Web thickness, in
β_w	=	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 ± WL (ASD A4.1)
DL + LL ± WL (ASD A4.1)

DL ± EL (ASD A4.1)
DL + LL ± EL (ASD A4.1)

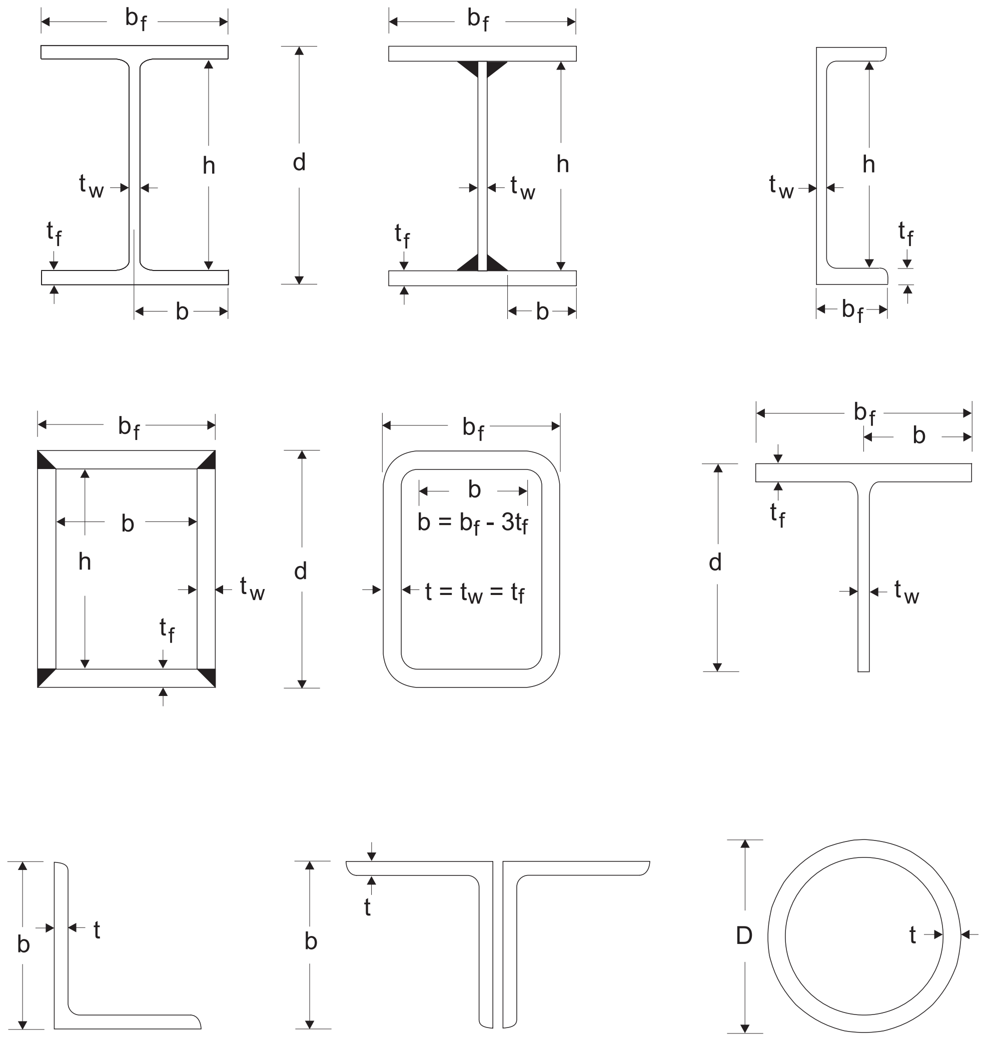
These are also the default design load combinations in SAP2000 whenever the AISC-ASD89 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

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. SAP2000 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 of the section properties required in this table is given in Figure III-1 and Table III-1.



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.

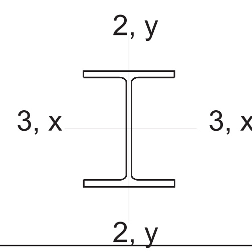


Figure III-1

AISC-ASD Definition of Geometric Properties

Section Description	Ratio Checked	Compact Section	Noncompact Section	Slender Section
I-SHAPE	$b_f / 2t_f$ (rolled)	$\leq 65 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y}$	No limit
	$b_f / 2t_f$ (welded)	$\leq 65 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y / k_c}$	No limit
	d / t_w	For $f_a / F_y \leq 0.16$ $\leq \frac{640}{\sqrt{F_y}}(1 - 3.74 \frac{f_a}{F_y})$, For $f_a / F_y > 0.16$ $\leq 257/\sqrt{F_y}$.	No limit	No limit
	h / t_w	No limit	If compression only, $\leq 253 / \sqrt{F_y}$ otherwise $\leq 760 / \sqrt{F_b}$	$\leq \frac{14000}{\sqrt{F_y(F_y + 16.5)}}$ ≤ 260
BOX	b / t_f	$\leq 190 / \sqrt{F_y}$	$\leq 238 / \sqrt{F_y}$	No limit
	d / t_w	As for I-shapes	No limit	No limit
	h / t_w	No limit	As for I-shapes	As for I-shapes
	Other	$t_w \geq t_f / 2$, $d_w \leq 6b_f$	None	None
CHANNEL	b / t_f	As for I-shapes	As for I-shapes	No limit
	d / t_w	As for I-shapes	No limit	No limit
	h / t_w	No limit	As for I-shapes	As for I-shapes
	Other	No limit	No limit	If welded $b_f / d_w \leq 0.25$, $t_f / t_w \leq 3.0$ If rolled $b_f / d_w \leq 0.5$, $t_f / t_w \leq 2.0$

Table III-2
Limiting Width-Thickness Ratios for
Classification of Sections Based on AISC-ASD

Section Description	Ratio Checked	Compact Section	Noncompact Section	Slender Section
T-SHAPE	$b_f / 2t_f$	$\leq 65 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y}$	No limit
	d / t_w	Not applicable	$\leq 127 / \sqrt{F_y}$	No limit
	Other	No limit	No limit	If welded $b_f / d_w \geq 0.5$, $t_f / t_w \geq 1.25$ If rolled $b_f / d_w \geq 0.5$, $t_f / t_w \geq 1.10$
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.)

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 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 SAP2000.**

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.:

$$f_a = P/A$$
$$f_{b33} = M_{33}/S_{33}$$
$$f_{b22} = M_{22}/S_{22}$$
$$f_{v2} = V_2/A_{v2}$$
$$f_{v3} = V_3/A_{v3}$$

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.

$$f_a = P/A \quad \text{(ASD A-B5.2d)}$$
$$f_{b33} = M_{33}/S_{eff,33} \quad \text{(ASD A-B5.2d)}$$
$$f_{b22} = M_{22}/S_{eff,22} \quad \text{(ASD A-B5.2d)}$$
$$f_{v2} = V_2/A_{v2} \quad \text{(ASD A-B5.2d)}$$
$$f_{v3} = V_3/A_{v3} \quad \text{(ASD A-B5.2d)}$$

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 SAP2000 "Redefine Element Design Data" form, these values ***will override the above mentioned calculated values for those elements*** *as defined in the following subsections. 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 $0.60\,F_y$.

$$F_a = 0.6 F_y \qquad \text{(ASD D1, ASD SAM 2)}$$

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

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

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

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:

$$F_a = \frac{\left\{1.0 - \frac{(Kl/r)^2}{2C_c^2}\right\}F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8\,C_c} - \frac{(Kl/r)^3}{8\,C_c^3}}, \quad \text{if } \frac{Kl}{r} \le C_c, \qquad \text{(ASD E2-1, SAM 4-1)}$$

$$F_a = \frac{12\,\pi^2\,E}{23(Kl/r)^2}, \quad \text{if } \frac{Kl}{r} > C_c. \qquad \text{(ASD E2-2, SAM 4-2)}$$

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:

$$F_a = Q\,\frac{\left\{1.0 - \frac{(Kl/r)^2}{2C_c'^2}\right\}F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8\,C_c'} - \frac{(Kl/r)^3}{8\,C_c'^3}}, \text{ if } \frac{Kl}{r} \le C_c', \text{(ASD A-B5-11, SAM 4-1)}$$

$$F_a = \frac{12\,\pi^2\,E}{23(Kl/r)^2}, \quad \text{if } \frac{Kl}{r} > C_c'. \text{(ASD A-B5-12, SAM 4-2)}$$

where,

$$C_c' = \sqrt{\frac{2\pi^2 E}{Q\,F_y}}. \qquad \text{(ASD A-B5.2c, ASD SAM 4)}$$

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:

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

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 (ASD A-B5.2.c, SAM 4)

Q_s = reduction factor for unstiffened slender elements, and (ASD A-B5.2.a)

Q_a = reduction factor for stiffened slender elements. (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.

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

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

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

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):

$$F_a = Q \frac{\left\{ 1.0 - \frac{\left(Kl/r \right)_e^2}{2 C_c'^2} \right\} F_y}{\frac{5}{3} + \frac{3 \left(Kl/r \right)_e}{8 C_c'} - \frac{\left(Kl/r \right)_e^3}{8 C_c'^3}}, \text{ if } \left(Kl/r \right)_e \leq C_c', \qquad \text{(E2-1, A-B5-11)}$$

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0 & \text{if} \quad b_f/2t_f \le 95\big/\sqrt{F_y/k_c}\,, \\ 1.293 - 0.00309\left[b_f/2t_f\right]\sqrt{F_y/k_c} & \text{if} \quad 95\big/\sqrt{F_y/k_c} < b_f/2t_f < 195\big/\sqrt{F_y/k_c}\,, \\ 26{,}200\,k_c\big/\left\{\left[b_f/2t_f\right]^2 F_y\right\} & \text{if} \quad b_f/2t_f \ge 195\big/\sqrt{F_y/k_c}\,. \end{cases}$	ASD A-B5-3, ASD A-B5-4
BOX	$Q_s = 1$	ASD A-B5.2c
CHANNEL	As for I-shapes with $b_f/2t_f$ replaced by b_f/t_f .	ASD A-B5-3, ASD A-B5-4
T-SHAPE	For flanges, as for flanges in I-shapes. For web see below. $Q_s \le \begin{cases} 1.0\,, & \text{if} \quad d/t_w \le 127\big/\sqrt{F_y}\,, \\ 1.908 - 0.00715\left[d/t_w\right]\sqrt{F_y}\,, & \text{if} \quad 127\big/\sqrt{F_y} < d/t_w < 176\big/\sqrt{F_y}\,, \\ 20{,}000\big/\left\{\left[d/t_w\right]^2 F_y\right\}, & \text{if} \quad d/t_w \ge 176\big/\sqrt{F_y}\,. \end{cases}$	ASD A-B5-3, ASD A-B5-4, ASD A-B5-5, ASD A-B5-6
DOUBLE-ANGLE	$Q_s = \begin{cases} 1.0\,, & \text{if} \quad b/t \le 76\big/\sqrt{F_y}\,, \\ 1.340 - 0.00447\left[b/t\right]\sqrt{F_y}\,, & \text{if} \quad 76\big/\sqrt{F_y} < b/t < 155\big/\sqrt{F_y}\,, \\ 15{,}500\big/\left\{\left[b/t\right]^2 F_y\right\}, & \text{if} \quad b/t \ge 155\big/\sqrt{F_y}\,. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
ANGLE	$Q_s = \begin{cases} 1.0\,, & \text{if} \quad b/t \le 76\big/\sqrt{F_y}\,, \\ 1.340 - 0.00447\left[b/t\right]\sqrt{F_y}\,, & \text{if} \quad 76\big/\sqrt{F_y} < b/t < 155\big/\sqrt{F_y}\,, \\ 15{,}500\big/\left\{\left[b/t\right]^2 F_y\right\}, & \text{if} \quad b/t \ge 155\big/\sqrt{F_y}\,. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
PIPE	$Q_s = 1$	ASD A-B5.2c
ROUND BAR	$Q_s = 1$	ASD A-B5.2c
RECTAN-GULAR	$Q_s = 1$	ASD A-B5.2c
GENERAL	$Q_s = 1$	ASD A-B5.2c

Table III-3

Reduction Factor for Unstiffened Slender Elements, Q_s

Section Type	Effective Width for Stiffened Sections	Equation Reference
I-SHAPE	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253\, t_w}{\sqrt{f}} \left[1 - \dfrac{44.3}{(h/t_w)\sqrt{f}} \right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \dfrac{P}{A_g}$)	ASD A-B5-8
BOX	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253\, t_w}{\sqrt{f}} \left[1 - \dfrac{44.3}{(h/t_w)\sqrt{f}} \right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \dfrac{P}{A_g}$)	ASD A-B5-8
	$b_e = \begin{cases} b, & \text{if } \dfrac{b}{t_f} \le \dfrac{183.74}{\sqrt{f}}, \\ \dfrac{253\, t_f}{\sqrt{f}} \left[1 - \dfrac{50.3}{(h/t_f)\sqrt{f}} \right], & \text{if } \dfrac{b}{t} > \dfrac{183.74}{\sqrt{f}}. \end{cases}$ (compr., flexure, $f = 0.6F_y$)	ASD A-B5-7
CHANNEL	$h_e = \begin{cases} h, & \text{if } \dfrac{h}{t_w} \le \dfrac{195.74}{\sqrt{f}}, \\ \dfrac{253\, t_w}{\sqrt{f}} \left[1 - \dfrac{44.3}{(h/t_w)\sqrt{f}} \right], & \text{if } \dfrac{h}{t_w} > \dfrac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \dfrac{P}{A_g}$)	ASD A-B5-8
T-SHAPE	$b_e = b$	ASD A-B5.2c
DOUBLE-ANGLE	$b_e = b$	ASD A-B5.2c
ANGLE	$b_e = b$	ASD A-B5.2c
PIPE	$Q_a = 1$, (However, special expression for allowable axial stress is given.)	ASD A-B5-9
ROUND BAR	Not applicable	—
RECTAN-GULAR	$b_e = b$	ASD A-B5.2c
GENERAL	Not applicable	—

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).

Table III-4
Effective Width for Stiffened Sections

$$F_a = \frac{12\, \pi^2\, E}{23\left(Kl/r\right)_e^{\;2}}, \qquad \text{if } \left(Kl/r\right)_e > C_c^{\,\prime}. \qquad \text{(E2-2, A-B5-12)}$$

where,

$$C_c^{\,\prime} = \sqrt{\frac{2\pi^2 E}{Q\, F_y}}\,, \text{ and} \qquad \text{(ASD E2, A-B5.2c, SAM 4)}$$

$$\left(Kl/r\right)_e = \sqrt{\frac{\pi^2 E}{F_e}}\,. \qquad \text{(ASD C-E2-2, SAM 4-4)}$$

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 SAP2000 as follows:

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

$$F_e = \left[\frac{\pi^2 E C_w}{\left(K_z l_z\right)^2} + GJ\right]\frac{1}{I_{22} + I_{33}} \qquad \text{(LRFD A-E3-5)}$$

- For T-sections and Double-angles:

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

- For Channels:

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

- For Single-angle sections with equal legs:

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

- 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):

$$(F_e - F_{e33})(F_e - F_{e22})(F_e - F_{ez}) - F_e^2 (F_e - F_{e22}) \frac{x_0^2}{r_0^2} - F_e^2 (F_e - F_{e33}) \frac{y_0^2}{r_0^2} = 0 ,$$

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),

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

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

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

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

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

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 SAP2000,

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

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

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:

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

$$F_{b33} = 0.60\, F_y \qquad \text{if } f_y > 65 \text{ ksi}, \qquad \text{(ASD F1-5)}$$

For Noncompact sections:

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

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

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

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 l_{22}/r_T ratio.

For $\dfrac{l_{22}}{r_T} \leq \sqrt{\dfrac{102{,}000\, C_b}{F_y}}$,

$$F_{b33} = 0.60 F_y \,, \qquad \text{(ASD F1-6)}$$

for $\sqrt{\dfrac{102{,}000\ C_b}{F_y}} < \dfrac{l_{22}}{r_T} \leq \sqrt{\dfrac{510{,}000\ C_b}{F_y}}$,

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

for $\dfrac{l_{22}}{r_T} > \sqrt{\dfrac{510{,}000\ C_b}{F_y}}$,

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

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

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

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,

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

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). SAP2000 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:

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

$R_{PG} = 1.0 - 0.0005 \frac{A_w}{A_f} \left[\frac{h}{t} - \frac{760}{\sqrt{F_{b33}}} \right] \leq 1.0$, (ASD G2)

$R_e = \frac{12 + \left(3\alpha - \alpha^3\right) \frac{A_w}{A_f}}{12 + 2 \frac{A_w}{A_f}} \leq 1.0$, (hybrid girders) (ASD G2)

$R_e = 1.0$, (non-hybrid girders) (ASD G2)

A_w = Area of web, in^2,

A_f = Area of compression flange, in^2,

$\alpha = \frac{0.6 F_y}{F_{b33}} \leq 1.0$ (ASD G2)

F_{b33} = Allowable bending stress assuming the section is non-compact, and

F'_{b33} = Allowable bending stress after considering web slenderness.

In the above expressions, R_e is taken as 1, because currently SAP2000 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.

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

Q_s is defined earlier.

Minor Axis of Bending

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

For Compact sections:

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

$$F_{b22} = 0.60\ F_y \qquad \text{if } f_y > 65 \text{ ksi}, \qquad \text{(ASD F2-2)}$$

For Noncompact and Slender sections:

$$F_{b22} = \left(1.075 - 0.005 \frac{b_f}{2t_f}\sqrt{F_y}\right) F_y, \quad \text{if } f_y \le 65 \text{ ksi}, \qquad \text{(ASD F2-3)}$$

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

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

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

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:

$$F_{b33} = 0.66\ F_y \qquad \text{if } f_y \le 65 \text{ ksi}, \qquad \text{(ASD F1-1)}$$

$$F_{b33} = 0.60\ F_y \qquad \text{if } f_y > 65 \text{ ksi}, \qquad \text{(ASD F1-5)}$$

For Noncompact sections:

$$F_{b33} = \left(0.79 - 0.002 \frac{b_f}{t_f}\sqrt{F_y}\right) F_y, \text{ if rolled and } f_y \le 65 \text{ ksi}, \qquad \text{(ASD F1-3)}$$

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

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

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:

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

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:

$$F'_{b33} = R_e R_{PG}\; F_{b33} \qquad \text{(ASD G2-1)}$$

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

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

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

Minor Axis of Bending

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

$$F_{b22} = 0.60\, F_y \qquad \text{(ASD F2-2)}$$

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

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

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 SAP2000.

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:

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

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

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:

$$F_{b33} = 0.60\, F_y \quad \text{(ASD F3-3)}$$

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:

$$F'_{b33} = R_e R_{PG}\, F_{b33} \quad \text{(ASD G2-1)}$$

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:

$$F_{b22} = 0.66\, F_y \quad \text{(for Compact sections)} \qquad \text{(ASD F3-1)}$$

$$F_{b22} = 0.60\, F_y \quad \text{(for Noncompact and Slender sections)} \qquad \text{(ASD F3-3)}$$

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

$$F_{b22} = 0.60\, F_y \qquad \text{(ASD F3-3)}$$

Pipe Sections

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

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

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

Round Bars

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

$$F_b = 0.75\, F_y\ . \qquad \text{(ASD F2-1)}$$

Rectangular and Square Bars

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

$$F_b = 0.75\, F_y\ . \qquad \text{(ASD F2-1)}$$

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

$$F_b = 0.60\, F_y,\ \text{And}$$

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

$$F_b = 0.75\, F_y\ . \qquad \text{(ASD F2-1)}$$

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):

$$F_{b,major} = \left[0.55 - 0.10\frac{F_{ob}}{F_y}\right]F_{ob}, \qquad \text{if} \quad F_{ob} \le F_y \qquad \text{(ASD SAM 5-3a)}$$

$$F_{b,major} = \left[0.95 - 0.50\sqrt{\frac{F_y}{F_{ob}}}\right]F_y \le 0.66F_y, \quad \text{if} \quad F_{ob} > F_y \qquad \text{(ASD SAM 5-3b)}$$

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

$$F_{ob} = C_b\,\frac{28{,}250}{l/t}, \qquad \text{(ASD SAM 5-5)}$$

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

$$F_{ob} = 143{,}100\,C_b\,\frac{I_{min}}{S_{major}\ l^2}\left[\sqrt{\beta_w^2 + 0.052(lt/r_{min})^2} + \beta_w\right], \qquad \text{(ASD SAM 5-6)}$$

where,

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

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

I_{min} = minor principal moment of inertia,

I_{max} = major principal moment of inertia,

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

r_{min} = radius of gyration for minor principal axis,

$$\beta_w = \left[\frac{1}{I_{max}} \int_A z(w^2 + z^2) dA \right] - 2z_0 \,, \qquad \text{(ASD SAM 5.3.2)}$$

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.

β_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 (ASD SAM 5.3.2). However, for conservative design in SAP2000, it is always taken as negative for unequal-leg angles.

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.

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

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

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

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

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

where,

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = slenderness reduction factor for local buckling. (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):

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

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

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

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 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 SAP2000 as:

$$F_v = 0.40\,F_y \qquad \text{(ASD F4-1, SAM 3-1)}$$

The allowable shear stress for major direction shears in I-shapes, boxes and channels is evaluated as follows:

$$F_v = 0.40\, F_y \quad , \qquad \text{if } \frac{h}{t_w} \le \frac{380}{\sqrt{F_y}}, \text{ and} \qquad \text{(ASD F4-1)}$$

$$F_v = \frac{C_v}{2.89} F_y \le 0.40\, F_y \; , \quad \text{if } \frac{380}{\sqrt{F_y}} < \frac{h}{t_w} \le 260\,. \qquad \text{(ASD F4-2)}$$

where,

$$C_v = \begin{cases} \dfrac{45{,}000\, k_v}{F_y \left(h/t_w \right)^2}, & \textit{if } \dfrac{h}{t_w} \ge 56{,}250 \dfrac{k_v}{F_y}\ , \\[2ex] \dfrac{190}{h/t_w} \sqrt{\dfrac{k_v}{F_y}}\ , & \textit{if } \dfrac{h}{t_w} < 56{,}250 \dfrac{k_v}{F_y}\ , \end{cases} \qquad \text{(ASD F4)}$$

$$k_v = \begin{cases} 4.00 + \dfrac{5.34}{\left(a/h \right)^2}, & \textit{if } \dfrac{a}{h} \le 1\ , \\[2ex] 5.34 + \dfrac{4.00}{\left(a/h \right)^2}, & \textit{if } \dfrac{a}{h} > 1\ , \end{cases} \qquad \text{(ASD F4)}$$

t_w = Thickness of the web,

a = Clear distance between transverse stiffeners, in. Currently it is taken conservatively as the length, l_{22}, of the member in SAP2000,

h = Clear distance between flanges at the section, in.

Minor Axis of Bending

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

$$F_v = 0.40\, F_y \qquad \text{(ASD F4-1, SAM 3-1)}$$

Calculation of Stress Ratios

In the calculation of the axial and bending stress capacity 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 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 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

$$\frac{f_a}{F_a} + \frac{C_{m33} f_{b33}}{\left(1 - \frac{f_a}{F'_{e33}}\right) F_{b33}} + \frac{C_{m22} f_{b22}}{\left(1 - \frac{f_a}{F'_{e22}}\right) F_{b22}}, \text{ and} \quad \text{(ASD H1-1, SAM 6.1)}$$

$$\frac{f_a}{Q\left(0.60\, F_y\right)} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \quad \text{where} \quad \text{(ASD H1-2, SAM 6.1)}$$

f_a, f_{b33}, f_{b22}, F_a, F_{b33}, and F_{b22} are defined earlier in this chapter,

C_{m33} and C_{m22} are coefficients representing distribution of moment along the member length.

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

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

$$F_e' = \frac{12\pi^2 E}{23(Kl/r)^2}. \quad \text{(ASD H1)}$$

A factor of 4/3 is applied on F_e' and $0.6F_y$ 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.

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

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

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

$$\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}}, \text{ where}$$

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

SAP2000. 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:

$$\frac{f_{v2}}{F_v}, \quad \text{and}$$

$$\frac{f_{v3}}{F_v}.$$

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).

Chapter IV

Check/Design for AISC-LRFD93

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 when the user selects the AISC-LRFD93 design code (AISC 1994). 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” 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.

A	=	Cross-sectional area, in^2
A_e	=	Effective cross-sectional area for slender sections, in^2
A_g	=	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	=	Major and minor shear areas, in^2
A_w	=	Shear area, equal dt_w per web, in^2
B_1	=	Moment magnification factor for moments not causing sidesway
B_2	=	Moment magnification factor for moments causing sidesway
C_b	=	Bending coefficient
C_m	=	Moment coefficient
C_w	=	Warping constant, in^6
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
F_{cr}	=	Critical compressive stress, ksi
F_r	=	Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi
F_y	=	Yield stress of material, ksi
G	=	Shear modulus, ksi
I_{22}	=	Minor moment of inertia, in^4
I_{33}	=	Major moment of inertia, in^4
J	=	Torsional constant for the section, in^4
K	=	Effective length factor
K_{33}, K_{22}	=	Effective length K-factors in the major and minor directions
L_b	=	Laterally unbraced length of member, in
L_p	=	Limiting laterally unbraced length for full plastic capacity, in
L_r	=	Limiting laterally unbraced length for inelastic lateral-torsional buckling, in
M_{cr}	=	Elastic buckling moment, kip-in
M_{lt}	=	Factored moments causing sidesway, kip-in
M_{nt}	=	Factored moments not causing sidesway, kip-in
M_{n33}, M_{n22}	=	Nominal bending strength in major and minor directions, kip-in
M_{ob}	=	Elastic lateral-torsional buckling moment for angle sections, kip-in
M_{r33}, M_{r22}	=	Major and minor limiting buckling moments, kip-in
M_u	=	Factored moment in member, kip-in
M_{u33}, M_{u22}	=	Factored major and minor moments in member, kip-in
P_e	=	Euler buckling load, kips
P_n	=	Nominal axial load strength, kip
P_u	=	Factored axial force in member, kips
P_y	=	$A_g F_y$, kips
Q	=	Reduction factor for slender section, = $Q_a Q_s$

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, in^3
S_{33}, S_{22}	=	Major and minor section moduli, in^3
$S_{eff,33}, S_{eff,22}$	=	Effective major and minor section moduli for slender sections, in^3
S_c	=	Section modulus for compression in an angle section, in^3
V_{n2}, V_{n3}	=	Nominal major and minor shear strengths, kips
V_{u2}, V_{u3}	=	Factored major and minor shear loads, kips
Z	=	Plastic modulus, in^3
Z_{33}, Z_{22}	=	Major and minor plastic moduli, in^3
b	=	Nominal dimension of plate in a section, in longer leg of angle sections, $b_f - 2t_w$ for welded and $b_f - 3t_w$ for rolled box sections, 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, $4 / \sqrt{h / t_w}$, $0.35 \le k_c \le 0.763$
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
β_w	=	Special section property for angles, in
λ	=	Slenderness parameter
λ_c, λ_e	=	Column slenderness parameters
λ_p	=	Limiting slenderness parameter for compact element
λ_r	=	Limiting slenderness parameter for non-compact element
λ_s	=	Limiting slenderness parameter for seismic element
$\lambda_{slender}$	=	Limiting slenderness parameter for slender element
φ_b	=	Resistance factor for bending, 0.9
φ_c	=	Resistance factor for compression, 0.85
φ_t	=	Resistance factor for tension, 0.9
φ_v	=	Resistance factor for shear, 0.9

Table IV-1
AISC-LRFD 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-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):

1.4 DL	(LRFD A4-1)
1.2 DL + 1.6 LL	(LRFD A4-2)
0.9 DL ± 1.3 WL	(LRFD A4-6)
1.2 DL ± 1.3 WL	(LRFD A4-4)
1.2 DL + 0.5 LL ± 1.3 WL	(LRFD A4-4)
0.9 DL ± 1.0 EL	(LRFD A4-6)
1.2 DL ± 1.0 EL	(LRFD A4-4)
1.2 DL + 0.5 LL ± 1.0 EL	(LRFD A4-4)

These are also the default design load combinations in SAP2000 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, SAP2000 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.

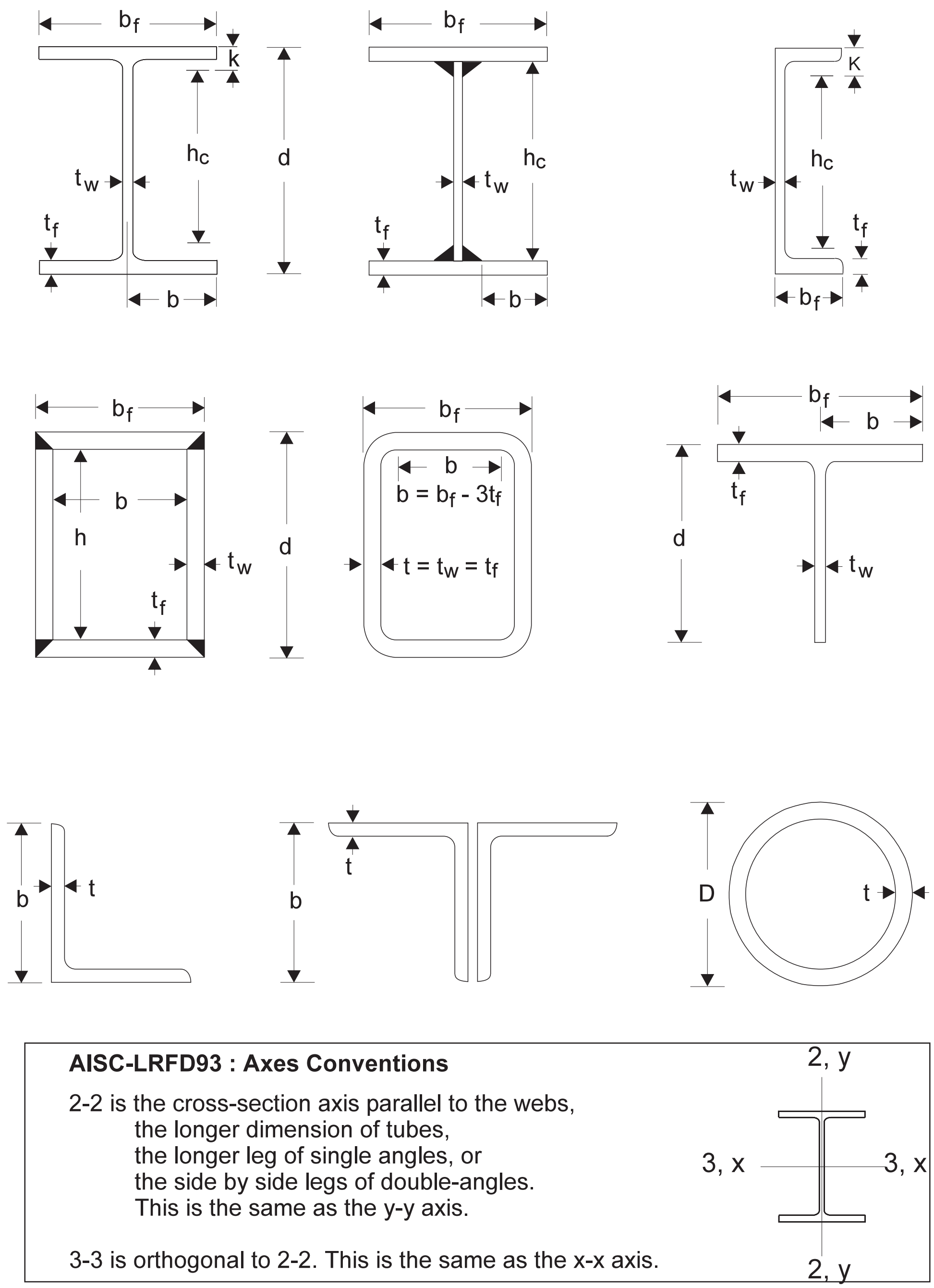


Figure IV-1

AISC-LRFD Definition of Geometric Properties

Description of Section	Check λ	COMPACT (λ_p)	NONCOMPACT λ_r	SLENDER ($\lambda_{slender}$)
I-SHAPE	$b_f / 2t_f$ (rolled)	$\leq 65 / \sqrt{F_y}$	$\leq 141 / \sqrt{F_y - 10.0}$	No limit
	$b_f / 2t_f$ (welded)	$\leq 65 / \sqrt{F_y}$	$\leq 162 / \sqrt{\frac{F_y - 16.5}{k_c}}$	No limit
	h_c / t_w	For $P_u / \varphi_b P_y \leq 0.125$, $\leq \frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75 P_u}{\varphi_b P_y}\right)$ For $P_u / \varphi_b P_y > 0.125$ $\leq \begin{cases} \frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\varphi_b P_y}\right) \\ \geq \frac{253}{\sqrt{F_y}} \end{cases}$	$\leq \frac{970}{\sqrt{F_y}}\left[1 - 0.74 \frac{P_u}{\varphi_b P_y}\right]$	$\leq \begin{cases} \frac{14000}{\sqrt{F_y\left(F_y + 16.5\right)}} \\ \leq 260 \end{cases}$
BOX	b / t_f h_c / t_w	$\leq 190 / \sqrt{F_y}$ As for I-shapes	$\leq 238 / \sqrt{F_y}$ As for I-shapes	No limit $\leq 970 / \sqrt{F_y}$
CHANNEL	b_f / t_f h_c / t_w	As for I-shapes As for I-shapes	As for I-shapes As for I-shapes	No limit As for I-shapes
T-SHAPE	$b_f / 2t_f$ d / t_w	As for I-Shapes Not applicable	As for I-Shapes $\leq 127 / \sqrt{F_y}$	No limit No limit
ANGLE	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
DOUBLE-ANGLE (Separated)	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$	No limit
PIPE	D / t	$\leq 2070 / F_y$	$\leq 8970 / F_y$	$\leq 13000 / F_y$ (Compression only) No limit for flexure
ROUND BAR	—	Assumed Compact		
RECTAN-GULAR	—	Assumed Noncompact		
GENERAL	—	Assumed Noncompact		

Table IV-2

Limiting Width-Thickness Ratios for Classification of Sections in Flexure based on AISC-LRFD

Description of Section	Width-Thickness Ratio λ	COMPACT (SEISMIC ZONE) (λ_s)	NONCOMPACT (Uniform Compression) $(M_{22} \approx M_{33} \approx 0)$ (λ_r)
I-SHAPE	$b_f / 2t_f$ (rolled)	$\leq 52 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y}$
	$b_f / 2t_f$ (welded)	$\leq 52 / \sqrt{F_y}$	$\leq 95 / \sqrt{F_y}$
	h_c / t_w	For $P_u / \varphi_b P_y \leq 0.125$, $\leq \frac{520}{\sqrt{F_y}} \left(1 - 1.54 \frac{P_u}{\varphi_b P_y} \right)$ For $P_u / \varphi_b P_y > 0.125$ $\leq \left\{ \frac{191}{\sqrt{F_y}} \left(2.33 - \frac{P_u}{\varphi_b P_y} \right) \geq \frac{253}{\sqrt{F_y}} \right\}$	$\leq 253 / \sqrt{F_y}$
BOX	b / t_f h_c / t_w	Not applicable Not applicable	$\leq 238 / \sqrt{F_y}$ $\leq 253 / \sqrt{F_y}$
CHANNEL	b_f / t_f h_c / t_w	As for I-shapes As for I-shapes	As for I-shapes As for I-shapes
T-SHAPE	$b_f / 2t_f$ d / t_w	Not applicable Not applicable	As for I-shapes $\leq 127 / \sqrt{F_y}$
ANGLE	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$
DOUBLE-ANGLE (Separated)	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$
PIPE	D / t	Not applicable	$\leq 3300 / F_y$
ROUND BAR	—	Assumed Compact	
RECTANGULAR	—	Assumed Noncompact	
GENERAL	—	Assumed Noncompact	

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

SAP2000 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 SAP2000.**

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_{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 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:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \text{, where} \qquad \text{(LRFD C1-1, SAM 6)}$$

B_1 = Moment magnification factor for non-sidesway moments,
B_2 = Moment magnification factor for sidesway moments,
M_{nt} = Factored moments not causing sidesway, and
M_{lt} = 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

$$B_1 = \frac{C_m}{\left(1 - P_u / P_e\right)} \geq 1.0 \text{, where} \qquad \text{(LRFD C1-2, SAM 6-2)}$$

P_e is the Euler buckling load ($P_e = \dfrac{A_g F_y}{\lambda^2}$, with $\lambda = \dfrac{Kl}{r\pi}\sqrt{\dfrac{F_y}{E}}$), and

C_{m33} and C_{m22} are coefficients representing distribution of moment along the member length.

$$C_m = \begin{cases} 1.00, & \text{if length is overwritten,} \\ 1.00, & \text{if tension member,} \\ 1.00, & \text{if end unrestrained,} \\ 0.6 - 0.4\dfrac{M_a}{M_b}, & \text{if no transverse loading,} \\ 0.85, & \text{if trans. load, end restrained,} \\ 1.00, & \text{if trans. load, end unrestrained,} \end{cases} \qquad \text{(LRFD C1-3)}$$

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.

SAP2000 design assumes the analysis includes P-Δ effects, therefore B_2 is taken as unity for bending in both directions. It is suggested that the P-Δ 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 $K_{22} l_{22}$ and $K_{33} l_{33}$ 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 shear stresses 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, φ, is taken as follows (LRFD A5.3):

φ_t = Resistance factor for tension, 0.9 (LRFD D1, H1, SAM 2, 6)
φ_c = Resistance factor for compression, 0.85 (LRFD E2, E3, H1)
φ_c = Resistance factor for compression in angles, 0.90 (LRFD SAM 4, 6)
φ_b = Resistance factor for bending, 0.9 (LRFD F1, H1, A-F1, A-G2, SAM 5)
φ_v = Resistance factor for shear, 0.9 (LRFD F2, A-F2, A-G3, SAM 3)

If the user specifies nominal strengths for one or more elements in the "Redefine Element Design Data" form, these values ***will override the above mentioned calculated values for those elements*** *as defined in the following subsections. The specified nominal 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_z, is used instead of r_{22} and r_{33} in computing Kl/r.

Flexural Buckling

The nominal axial compressive strength, P_n, depends on the slenderness ratio, Kl/r, and its critical value, λ_c, where

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

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}\,. \qquad \text{(LRFD E2-4, SAM 4)}$$

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:

$$P_n = A_g F_{cr}\,, \quad \text{where} \qquad \text{(LRFD E2-1)}$$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y, \quad \text{for} \quad \lambda_c \leq 1.5, \text{ and} \qquad \text{(LRFD E2-2)}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y, \quad \text{for} \quad \lambda_c > 1.5. \qquad \text{(LRFD E2-3)}$$

P_n for Slender sections is evaluated for flexural buckling as follows:

$$P_n = A_g F_{cr}\,, \quad \text{where} \qquad \text{(LRFD A-B3d, SAM 4)}$$

$$F_{cr} = Q\left(0.658^{Q\lambda_c^2}\right) F_y, \quad \text{for} \quad \lambda_c\sqrt{Q} \leq 1.5\text{, and} \qquad \text{(LRFD A-B5-15, SAM 4-1)}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y, \quad \text{for} \quad \lambda_c\sqrt{Q} > 1.5\,. \qquad \text{(LRFD A-B5-16, SAM 4-2)}$$

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

$$Q = Q_s\, Q_a\,, \quad \text{where} \qquad \text{(LRFD A-B5-17, SAM 4)}$$

Q_s = reduction factor for unstiffened slender elements, and (LRFD A-B5.3a)

Q_a = reduction factor for stiffened slender elements. (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_a factors for slender sections are calculated as the ratio of effective cross-sectional area and the gross cross-sectional area (LRFD A-B5.3c).

$$Q_a = \frac{A_e}{A_g} \qquad \text{(LRFD A-B5-14)}$$

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0\ , & \text{if } b_f/2t_f \le 95/\sqrt{F_y}\ , \\ 1.415 - 0.00437[b_f/2t_f]\sqrt{F_y}\ , & \text{if } 95/\sqrt{F_y} < b_f/2t_f < 176/\sqrt{F_y}\ , \\ 20{,}000/\{[b_f/2t_f]^2 F_y\}, & \text{if } b_f/2t_f \ge 176/\sqrt{F_y}\ . \end{cases}$ (rolled)	LRFD A-B5-5, LRFD A-B5-6
	$Q_s = \begin{cases} 1.0 & \text{if } b_f/2t_f \le 109/\sqrt{F_y/k_c}\ , \\ 1.415 - 0.00381[b_f/2t_f]\sqrt{F_y/k_c} & \text{if } 109/\sqrt{F_y/k_c} < b_f/2t_f < 200/\sqrt{F_y/k_c}\ , \\ 26{,}200\, k_c/\{[b_f/2t_f]^2 F_y\} & \text{if } b_f/2t_f \ge 200/\sqrt{F_y/k_c}\ . \end{cases}$ (welded)	LRFD A-B5-7, LRFD A-B5-8
BOX	$Q_s = 1$	LRFD A-B5.3d
CHANNEL	As for I-shapes with $b_f/2t_f$ replaced by b_f/t_f .	LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8
T-SHAPE	For flanges, as for flanges in I-shapes. For web see below. $Q_s \le \begin{cases} 1.0\ , & \text{if } d/t_w \le 127/\sqrt{F_y}\ , \\ 1.908 - 0.00715[d/t_w]\sqrt{F_y}\ , & \text{if } 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}\ , \\ 20{,}000/\{[d/t_w]^2 F_y\}, & \text{if } d/t_w \ge 176/\sqrt{F_y}\ . \end{cases}$	LRFD A-B5-5, LRFD A-B5-6, LRFD A-B5-7, LRFD A-B5-8, LRFD A-B5-9, LRFDA-B5-10
DOUBLE-ANGLE (Separated)	$Q_s = \begin{cases} 1.0\ , & \text{if } b/t \le 76/\sqrt{F_y}\ , \\ 1.340 - 0.00447[b/t]\sqrt{F_y}\ , & \text{if } 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}\ , \\ 15{,}500/\{[b/t]^2 F_y\}, & \text{if } b/t \ge 155/\sqrt{F_y}\ . \end{cases}$	LRFD A-B5-3, LRFD A-B5-4
ANGLE	$Q_s = \begin{cases} 1.0\ , & \text{if } b/t \le 0.446\sqrt{F_y/E}\ , \\ 1.34 - 0.761[b/t]\sqrt{F_y/E}\ , & \text{if } 0.446\sqrt{F_y/E} < b/t < 0.910\sqrt{F_y/E}\ , \\ 0.534/\{[b/t]^2[F_y/E]\}, & \text{if } b/t \ge 0.910\sqrt{F_y/E}\ . \end{cases}$	LRFD SAM4-3
PIPE	$Q_s = 1$	LRFD A-B5.3d
ROUND BAR	$Q_s = 1$	LRFD A-B5.3d
RECTAN-GULAR	$Q_s = 1$	LRFD A-B5.3d
GENERAL	$Q_s = 1$	LRFD A-B5.3d

Table IV-4
Reduction Factor for Unstiffened Slender Elements, Q_s

Section Type	Effective Width for Stiffened Sections	Equation Reference
I-SHAPE	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right], & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
BOX	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right], & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$) $b_e = \begin{cases} b, & \text{if } \frac{b}{t_f} \le \frac{238}{\sqrt{f}}, \\ \frac{326\,t_f}{\sqrt{f}}\left[1 - \frac{64.9}{(b/t_f)\sqrt{f}}\right], & \text{if } \frac{b}{t_f} > \frac{238}{\sqrt{f}}. \end{cases}$ (compr. or flexure, $f = F_y$)	LRFD A-B5-12 LRFD A-B5-11
CHANNEL	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \le \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}}\left[1 - \frac{57.2}{(h/t_w)\sqrt{f}}\right], & \text{if } \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
T-SHAPE	$b_e = b$	LRFD A-B5.3b
DOUBLE-ANGLE (Separated)	$b_e = b$	LRFD A-B5.3b
ANGLE	$b_e = b$	LRFD A-B5.3b
PIPE	$Q_a = \begin{cases} 1, & \text{if } \frac{D}{t} \le \frac{3{,}300}{F_y}, \\ \frac{1{,}100}{(D/t)F_y} + \frac{2}{3}, & \text{if } \frac{D}{t} > \frac{3{,}300}{F_y}. \end{cases}$ (compression only)	LRFD A-B5-13
ROUND BAR	Not applicable	—
RECTAN-GULAR	$b_e = b$	LRFD A-B5.3b
GENERAL	Not applicable	—

Table IV-5
Effective Width for Stiffened Sections

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

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

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 λ_r is given by

$$P_n = A_g F_{crft} \text{ , where} \qquad \text{(LRFD E3-1)}$$

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

$$F_{crz} = \frac{GJ}{A r_0^2} \text{ ,}$$

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

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 $\lambda_c = \frac{Kl}{\pi r_{22}} \sqrt{\frac{F_y}{E}}$.

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:

$$P_n = A_g F_{cr} \text{ , where} \qquad \text{(LRFD A-E3-1)}$$

$$F_{cr} = Q\left(0.658^{Q\lambda_e^2}\right) F_y, \quad \text{for} \quad \lambda_e\sqrt{Q} \leq 1.5, \text{ and} \qquad \text{(LRFD A-E3-2)}$$

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

In the above equations, the slenderness parameter λ_e is calculated as

$$\lambda_e = \sqrt{\frac{F_y}{F_e}}, \qquad \text{(LRFD A-E3-4)}$$

where F_e is calculated as follows:

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

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

- For T-sections and Double-angles:

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

- For Channels:

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

- For Single-angles sections with equal legs:

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

- 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):

$$(F_e - F_{e33})(F_e - F_{e22})(F_e - F_{ez}) - F_e^2(F_e - F_{e22})\frac{x_0^2}{r_0^2} - F_e^2(F_e - F_{e33})\frac{y_0^2}{r_0^2} = 0,$$

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),

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

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

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

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

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

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

K_z is the effective length factor for torsional buckling, and it is taken equal to K_{22} in SAP2000,

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. 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 Kl/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:

$$M_p = Z F_y \leq 1.5 S F_y \qquad \text{(LRFD F1-1)}$$

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):

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

(LRFD F1-1, F1-2, F1-12)

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,
$(F_y - F_r)S_{33}$ for I-shapes and channels, (LRFD F1-7)
and $F_y S_{eff,33}$ for rectangular bars and boxes, (LRFD F1-11)

M_{cr33} = Critical elastic moment,

$$\frac{C_b \pi}{L_b}\sqrt{EI_{22}GJ + \left(\frac{\pi E}{L_b}\right)^2 I_{22}C_w}$$

for I-shapes and channels, and (LRFD F1-13)

$$\frac{57\,000\; C_b \sqrt{JA}}{L_b \,/\, r_{22}}$$

for boxes and rectangular bars, (LRFD F1-14)

L_b = Laterally unbraced length, l_{22} ,

L_p = Limiting laterally unbraced length for full plastic capacity,

$$\frac{300\; r_{22}}{\sqrt{F_y}}$$

for I-shapes and channels, and (LRFD F1-4)

$$\frac{3\,750\; r_{22}}{M_{p33}}\sqrt{JA}$$

for boxes and rectangular bars, (LRFD F1-5)

L_r = Limiting laterally unbraced length for inelastic lateral-torsional buckling,

$$\frac{r_{22}X_1}{F_y - F_r}\left\{1 + \left[1 + X_2(F_y - F_r)^2\right]^{1/2}\right\}^{1/2}$$

for I-shapes and channels, and (LRFD F1-6)

$$\frac{57\,000\; r_{22}\sqrt{JA}}{M_{r33}}$$

for boxes and rectangular bars, (LRFD F1-10)

$$X_1 = \frac{\pi}{S_{33}}\sqrt{\frac{EGJA}{2}}, \qquad \text{(LRFD F1-8)}$$

$$X_2 = 4\frac{C_w}{I_{22}}\left(\frac{S_{33}}{GJ}\right)^2, \qquad \text{(LRFD F1-9)}$$

$$C_b = \frac{12.5\, M_{max}}{2.5\, M_{max} + 3\, M_A + 4\, M_B + 3\, M_C}, \text{ and} \qquad \text{(LRFD F1-3)}$$

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 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.

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

$$M_{n22} = M_{p22} = Z_{22} F_y \leq 1.5 S_{22} F_y \qquad \text{(LRFD F1)}$$

For pipes and circular bars bent about any axis,

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

T-sections and Double Angles

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

$$M_{n33} = \frac{\pi \sqrt{E I_{22} G J}}{L_b} \left[B + \sqrt{1 + B^2} \right] \text{, where} \qquad \text{(LRFD F1-15)}$$

$$M_{n33} \leq 1.5 F_y S_{33} \text{, for positive moment, stem in tension} \qquad \text{(LRFD F1.2c)}$$

$$M_{n33} \leq F_y S_{33} , \quad \text{for negative moment, stem in compression} \quad \text{(LRFD F1.2c)}$$

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

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,

$$M_{n22} = S_{22} F_y .$$

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):

$$M_{n,major} = \left[0.92 - 0.17 \frac{M_{ob}}{M_{y,major}} \right] M_{ob} \leq 1.25 M_{y,major}, \quad if \ M_{ob} \leq M_{y,major},$$

$$M_{n,major} = \left[1.58 - 0.83 \sqrt{\frac{M_{y,major}}{M_{ob}}} \right] M_{y,major} \leq 1.25 M_{y,major}, \ if \ M_{ob} > M_{y,major},$$

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

$$M_{ob} = C_b \frac{0.46 E b^2 t^2}{l}, \qquad \text{(LRFD SAM 5-5)}$$

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

$$M_{ob} = 4.9 E C_b \frac{I_{min}}{l^2} \left[\sqrt{\beta_w^2 + 0.052 (lt/r_{min})^2} + \beta_w \right], \qquad \text{(LRFD SAM 5-6)}$$

where,

$t = \min \left(t_w, t_f \right)$,

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

I_{min} = minor principal axis moment of inertia,

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

r_{min} = radius of gyration for minor principal axis,

$$\beta_w = \left[\frac{1}{I_{max}} \int_A z (w^2 + z^2) dA \right] - 2 z_0, \qquad \text{(LRFD SAM 5.3.2)}$$

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.

β_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 SAP2000, 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,

$$M_{n33} = \begin{cases} M_{p33}, & \text{if} \quad \lambda \le \lambda_p, \\ M_{p33} - (M_{p33} - M_{r33})\left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right), & \text{if} \quad \lambda_p < \lambda \le \lambda_r, \\ M_{cr33} \le M_{p33}, & \text{if} \quad \lambda > \lambda_r. \end{cases} \quad \text{(A-F1-3)}$$

and for minor direction bending,

$$M_{n22} = \begin{cases} M_{p22}, & \text{if} \quad \lambda \le \lambda_p, \\ M_{p22} - (M_{p22} - M_{r22})\left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right), & \text{if} \quad \lambda_p < \lambda \le \lambda_r, \\ M_{cr22} \le M_{p22}, & \text{if} \quad \lambda > \lambda_r. \end{cases} \quad \text{(A-F1-3)}$$

where,

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

M_{r33}	=	Major limiting buckling moment,
M_{r22}	=	Minor limiting buckling moment,
M_{cr33}	=	Major buckling moment,
M_{cr22}	=	Minor buckling moment,
λ	=	Controlling slenderness parameter,
λ_p	=	Largest value of λ for which $M_n = M_p$, and
λ_r	=	Largest value of λ for which buckling is inelastic.

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

I Shapes, Channels

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

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

$$\lambda_p = \frac{65}{\sqrt{F_y}}, \qquad \text{(LRFD B5.1, Table A-F1.1)}$$

$$\lambda_r = \begin{cases} \dfrac{141}{\sqrt{F_y - F_r}}, & \text{For rolled shape,} \\ \dfrac{162}{\sqrt{(F_y - F_r)/k_c}}, & \text{For welded shape,} \end{cases} \qquad \text{(LRFD Table A-F1.1)}$$

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

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

$$M_{cr33} = \begin{cases} \dfrac{20{,}000}{\lambda^2} S_{33}, & \text{For rolled shape,} \\ \dfrac{26{,}200 k_c}{\lambda^2} S_{33}, & \text{For welded shape,} \end{cases} \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{cr22} = \begin{cases} \dfrac{20{,}000}{\lambda^2} S_{22}, & \text{For rolled shape,} \\ \dfrac{26{,}200 k_c}{\lambda^2} S_{22}, & \text{For welded shape,} \end{cases} \qquad \text{(LRFD Table A-F1.1)}$$

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

Boxes

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

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

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

$$M_{r33} = (F_y - F_r)S_{eff,33}, \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{r22} = (F_y - F_r)S_{eff,22}, \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{cr33} = F_y\, S_{eff,33}\left(S_{eff,33} / S_{33}\right), \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{cr22} = F_y\, S_{eff,22}, \qquad \text{(LRFD Table A-F1.1)}$$

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

$S_{eff,33}$ = effective major section modulus considering slenderness, and

$S_{eff,22}$ = effective minor section modulus considering slenderness.

T-sections and Double Angles

No local buckling is considered for T sections and Double angles in SAP2000. 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):

$$M_n = \begin{cases} 1.25 F_y S_c \,, & if \quad \dfrac{b}{t} \le 0.382\sqrt{\dfrac{E}{F_y}}, \\ F_y S_c \left[1.25 - 1.49 \left(\dfrac{b/t}{0.382\sqrt{\dfrac{E}{F_y}}} - 1 \right) \right], & if \quad 0.382\sqrt{\dfrac{E}{F_y}} < \dfrac{b}{t} \le 0.446\sqrt{\dfrac{E}{F_y}}, \\ Q F_y S_c \,, & if \quad \dfrac{b}{t} > 0.446\sqrt{\dfrac{E}{F_y}}, \end{cases}$$

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

$$\lambda = \frac{D}{t}, \qquad \text{(LRFD Table A-F1.1)}$$

$$\lambda_p = \frac{2{,}070}{F_y}, \qquad \text{(LRFD Table A-F1.1)}$$

$$\lambda_r = \frac{8{,}970}{F_y} \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{r33} = M_{r22} = \left(\frac{600}{D/t} + F_y \right) S \,, \qquad \text{(LRFD Table A-F1.1)}$$

$$M_{cr33} = M_{cr22} = \left(\frac{9{,}570}{D/t} \right) S \,, \qquad \text{(LRFD Table A-F1.1)}$$

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 SAP2000 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 SAP2000 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):

$$M_{n33} = \begin{cases} M_{p33}\,, & if \quad \lambda \le \lambda_p\,, \\ M_{p33} - (M_{p33} - M_{r33})\left(\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right), & if \quad \lambda_p < \lambda \le \lambda_r\,, \\ S_{33} R_{PG} R_e F_{cr}\,, & if \quad \lambda > \lambda_r\,, \end{cases} \quad \text{(A-F1,A-G1)}$$

where,

M_{n33}	=	Nominal major bending strength,
M_{p33}	=	Major plastic moment, $Z_{33} F_y \le 1.5\, S_{33} F_y$, (LRFD F1.1)
M_{r33}	=	Major limiting buckling moment, $R_e\, S_{33} F_y$, (LRFD Table A-F1.1)
λ	=	Web slenderness parameter,
λ_p	=	Largest value of λ for which $M_n = M_p$,
λ_r	=	Largest value of λ 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:

$$\lambda = \frac{h_c}{t_w}\,,$$

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

$$\lambda_r = \frac{970}{\sqrt{F_y}}\left(1 - 0.74\frac{P_u}{\varphi_b P_y}\right).$$

The parameters R_{PG}, R_e, and F_{cr} for slender web sections are calculated in SAP2000 as follows:

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

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

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

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

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

In the above expressions, R_e is taken as 1, because currently SAP2000 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 η in SAP2000 as follows:

$$F_{cr} = \begin{cases} F_y, & \text{if} \quad \eta \le \eta_p, \\ C_b F_y \left[1 - \frac{1}{2} \frac{\eta - \eta_p}{\eta_r - \eta_p} \right] \le F_y, & \text{if} \quad \eta_p < \eta \le \eta_r, \\ \frac{C_{PG}}{\eta^2}, & \text{if} \quad \eta > \eta_r, \end{cases} \quad \text{(LRFD A-G2-4, 5, 6)}$$

The parameters η, η_p, η_r, and C_{PG} for lateral-torsional buckling for slender web I, Channel and Box sections are given below:

$$\eta = \frac{L_b}{r_T}, \quad \text{(LRFD A-G2-7)}$$

$$\eta_p = \frac{300}{\sqrt{F_y}}, \quad \text{(LRFD A-G2-8)}$$

$$\eta_r = \frac{756}{\sqrt{F_y}}, \quad \text{(LRFD A-G2-9)}$$

$$C_{PG} = 286{,}000\ C_b, \text{ and} \quad \text{(LRFD A-G2-10)}$$

r_T = radius of gyration of the compression flange plus one-third of the compression portion of the web, and it is taken as $b_f \big/ \sqrt{12}$ in SAP2000.

C_b = a factor which depends on span moment. It is calculated using the equation given in page 62.

The parameters η, η_p, η_r, and C_{PG} for flange local buckling for slender web I, Channel and Box sections are given below:

$$\eta = \frac{b}{t}, \quad \text{(LRFD A-G2-11)}$$

$$\eta_p = \frac{65}{\sqrt{F_y}}, \quad \text{(LRFD A-G2-12)}$$

$$\eta_r = \frac{230}{\sqrt{F_y / k_c}}, \quad \text{(LRFD A-G2-13)}$$

$$C_{PG} = 26{,}200\, k_c, \text{ and} \quad \text{(LRFD A-G2-14)}$$

$$C_b = 1. \quad \text{(LRFD A-G2-15)}$$

T-sections and Double Angles

No local buckling is considered for T-sections and Double-angles in SAP2000. 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 SAP2000.

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 SAP2000.

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 SAP2000 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 $\dfrac{h}{t_w} \le \dfrac{418}{\sqrt{F_y}}$,

$$V_{n2} = 0.6 F_y A_w, \qquad \text{(LRFD F2-1)}$$

for $\dfrac{418}{\sqrt{F_y}} < \dfrac{h}{t_w} \le \dfrac{523}{\sqrt{F_y}}$,

$$V_{n2} = 0.6\, F_y\, A_w \frac{418}{\sqrt{F_y}} \Big/ \frac{h}{t_w}, \text{ and} \qquad \text{(LRFD F2-2)}$$

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

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

The nominal shear strength for all other sections is taken as:

$$V_{n2} = 0.6\, F_y A_{v2}.$$

Minor Axis of Bending

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

$$V_{n3} = 0.6 F_y A_{v3}$$

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 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.

Axial and Bending Stresses

The interaction ratio is determined based on the ratio $\frac{P_u}{\varphi P_n}$. If P_u is tensile, P_n is the nominal axial tensile strength and $\varphi = \varphi_t = 0.9$; and if P_u is compressive, P_n 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 $\frac{P_u}{\varphi P_n} \geq 0.2$, the capacity ratio is given as

$$\frac{P_u}{\varphi P_n} + \frac{8}{9}\left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}}\right). \quad \text{(LRFD H1-1a, SAM 6-1a)}$$

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

$$\frac{P_u}{2\varphi P_n} + \left(\frac{M_{u33}}{\varphi_b M_{n33}} + \frac{M_{u22}}{\varphi_b M_{n22}}\right). \quad \text{(LRFD H1-1b, SAM 6-1a)}$$

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 SAP2000. 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:

$$\frac{V_{u2}}{\varphi_v V_{n2}}, \text{ and}$$

$$\frac{V_{u3}}{\varphi_v V_{n3}},$$

where $\varphi_v = 0.9$.

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 AASHTO 1997

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 when the user selects the AASHTO design code (AASHTO 1997). Various notations used in this chapter are described in Table V-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 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.

The design and check are limited to noncomposite, nonhybrid and unstiffened sections. Composite, hybrid and stiffened sections should be investigated by the users independently of SAP2000.

A	=	Cross-sectional area, in^2
A_g	=	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	=	Major and minor shear areas, in^2
A_w	=	Shear area, equal dt_w per web, in^2
C_b	=	Bending coefficient
C_m	=	Moment coefficient
C_w	=	Warping constant, in^6
D	=	Outside diameter of pipes, in
D_c	=	Depth of web in compression, in
D_{cp}	=	Depth of web in compression under plastic moment, in
E	=	Modulus of elasticity, ksi
F_{cr}	=	Critical compressive stress, ksi
F_r	=	Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi
F_y	=	Yield stress of material, ksi
G	=	Shear modulus, ksi
I_{22}	=	Minor moment of inertia, in^4
I_{33}	=	Major moment of inertia, in^4
J	=	Torsional constant for the section, in^4
K	=	Effective length factor
K_{33}, K_{22}	=	Effective length K-factors in the major and minor directions
L_b	=	Laterally unbraced length of member, in
L_p	=	Limiting laterally unbraced length for full plastic capacity, in
L_r	=	Limiting laterally unbraced length for inelastic lateral-torsional buckling, in
M_{cr}	=	Elastic buckling moment, kip-in
M_b	=	Factored moments not causing sidesway, kip-in
M_s	=	Factored moments causing sidesway, kip-in
M_{n33}, M_{n22}	=	Nominal bending strength in major and minor directions, kip-in
M_{p33}, M_{p22}	=	Major and minor plastic moments, kip-in
M_{r33}, M_{r22}	=	Major and minor limiting buckling moments, kip-in
M_u	=	Factored moment in member, kip-in
M_{u33}, M_{u22}	=	Factored major and minor moments in member, kip-in
P_e	=	Euler buckling load, kips
P_n	=	Nominal axial load strength, kip
P_u	=	Factored axial force in member, kips

Table V-1
AASHTO-LRFD Notations

S	=	Section modulus, in^3
S_{33}, S_{22}	=	Major and minor section moduli, in^3
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, in^3
Z_{33}, Z_{22}	=	Major and minor plastic moduli, in^3
b	=	Nominal dimension of longer leg of angles, in $b_f - 2t_w$ for welded and $b_f - 3t_w$ for rolled BOX (TS) sections
b_f	=	Flange width, in
d	=	Overall depth of member, 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, $\frac{4}{\sqrt{h / t_w}}$, $0.35 \le k_c \le 0.763$
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
r_z	=	Minimum Radius of gyration for angles, in
t	=	Thickness, in
t_f	=	Flange thickness, in
t_w	=	Thickness of web, in
δ_b	=	Moment magnification factor for moments not causing sidesway
δ_s	=	Moment magnification factor for moments causing sidesway
λ	=	Slenderness parameter
λ_c	=	Column slenderness parameter
λ_p	=	Limiting slenderness parameter for compact element
λ_r	=	Limiting slenderness parameter for non-compact element
φ	=	Resistance factor
φ_f	=	Resistance factor for bending, 0.9
φ_c	=	Resistance factor for compression, 0.85
φ_y	=	Resistance factor for tension, 0.9
φ_v	=	Resistance factor for shear, 0.9

Table V-1
AASHTO-LRFD Notations (continued)

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 prescribed load cases for which the structure needs to be checked.

There are six types of dead loads: dead load of structural components and nonstructural attachments (DC), downdrag (DD), dead load of wearing surface and utilities (DW), horizontal earth pressure load (EH), vertical earth pressure load (EV), earth surcharge load (ES). Each type of dead load case requires a separate load factor (AASHTO 3.4.1).

There are six types of live loads: vehicular live load (LL), vehicular dynamic load allowance (IM), vehicular centrifugal force (CE), vehicular braking force (BR), pedestrian live load (PL), and live load surcharge (LS). All these live load cases require the same factor and do not need to be treated separately (AASHTO 3.4.1).

If the structure is subjected to structural dead load (DL), live load (LL), wind load (WL), and earthquake loads (EL), and considering that wind and earthquake forces are reversible, the following default load combinations have been considered for Strength and Extreme Event limit states (AASHTO 3.4.1).

1.50 DL	(Strength-IV)
1.25 DL + 1.75 LL	(Strength-I)
0.90 DL $\pm$ 1.4 WL	(Strength-III)
1.25 DL $\pm$ 1.4 WL	(Strength-III)
1.25 DL + 1.35 LL $\pm$ 0.40 WL	(Strength-V)
0.90 DL $\pm$ 1.0 EL	(Extreme-I)
1.25 DL + 0.5 LL $\pm$ 1.0 EL	(Extreme-I)

These are also the default design load combinations in SAP2000 whenever the AASHTO LRFD 1997 code is used. There are more different types of loads specified in the code than are considered in the current implementation of the default load combinations. However, the user has full control of the definition of loads and load combinations. The user is expected to define the other load combinations as necessary.

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 AASHTO code, SAP2000 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 (AASHTO C4.5.3.2.1) of 1.25 DL plus 1.35 LL (See 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, or Slender. SAP2000 classifies individual members according to the width/thickness ratio quantities given in Table V-2 (AASHTO 6). The definitions of the section properties required in these tables are given in Figure V-1. **If the limits for non-compact criteria are not met, the section is classified as Slender. Currently SAP2000 does not check stresses for Slender sections**.

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 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:

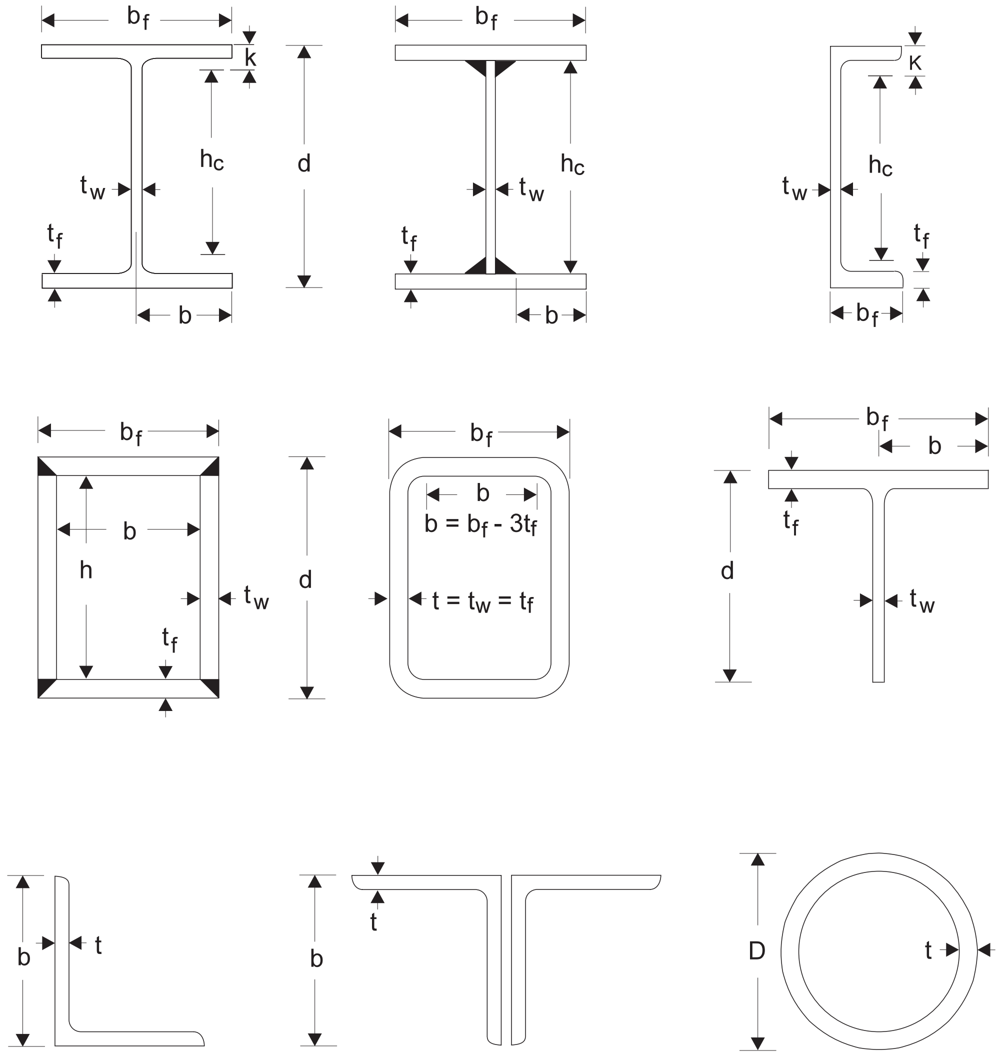
$$M_u = \delta_b M_b + \delta_s M_s \text{, where} \qquad \text{(AASHTO 4.5.3.2.2b)}$$

δ_b = Moment magnification factor for moments in braced mode,
δ_s = Moment magnification factor for moments in sidesway mode,
M_b = Factored moments not causing sidesway, and
M_s = Factored moments causing sidesway.

Description of Section	Check λ	Compact (λ_p)	Noncompact λ_r
I-SHAPE	$b_f / 2t_f$	$\leq 0.382\sqrt{\frac{E}{F_y}}$	$\leq 1.38\sqrt{\frac{E}{F_y\sqrt{\frac{2D_c}{t_w}}}}$
	$2D_{cp}/t_w$	$\leq 3.76\sqrt{\frac{E}{F_y}}$	$\leq 6.77\sqrt{\frac{E}{F_y}}$
	L_b	$\leq \left[0.124 - 0.0759\frac{M_u}{M_p}\right]\frac{r_{22}E}{F_y}$	$\leq 1.76\, r_t\sqrt{\frac{E}{F_y}}$
BOX	—	Assumed noncompact	
CHANNEL	b_f / t_f	$\leq 65 / \sqrt{F_y}$	$\leq 141 / \sqrt{F_y - 10.0}$
	h_c / t_w	For $P_u / \varphi_f P_y \leq 0.125$, $\leq \frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P_u}{\varphi_f P_y}\right)$ For $P_u / \varphi_f P_y > 0.125$ $\leq \left\{\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\varphi_f P_y}\right) \geq \frac{253}{\sqrt{F_y}}\right\}$	$\leq \frac{970}{\sqrt{F_y}}$
T-SHAPE	$b_f / 2t_f$ d / t_w	As for Channels Not applicable	As for Channels $\leq 127 / \sqrt{F_y}$
ANGLE	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$
DOUBLE-ANGLE (Sep.)	b / t	Not applicable	$\leq 76 / \sqrt{F_y}$
PIPE	D / t	$\leq 2\sqrt{E/F_y}$	$\leq 8.8\sqrt{E/F_y}$
ROUND BAR	—	Assumed compact	
RECTAN-GULAR	—	Assumed Compact	
GENERAL	—	Assumed Noncompact	

Table V-2

Limiting Width-Thickness Ratio for Flexure
Classification of Sections According to AASHTO



AASHTO-LRFD97 : 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.

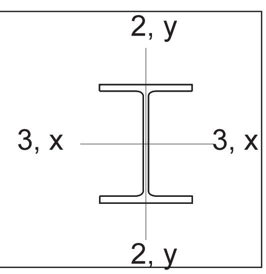


Figure V-1
AASHTO Definition of Geometric Properties

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

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\varphi_c P_e}} \geq 1.0, \text{ where} \qquad \text{(AASHTO 4.5.3.2.2b)}$$

P_e is the Euler buckling load,

$$P_e = \frac{\pi^2 EI}{(Kl_u)^2}, \qquad \text{(AASHTO 4.5.3.2.2b)}$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4, \text{ where} \qquad \text{(AASHTO 4.5.3.2.2b)}$$

M_a / M_b is the ratio of the smaller to the larger nonsway moments at the ends of the member, M_a / M_b being positive for single curvature bending and negative for double curvature bending. For compression members with transverse load on the member, C_m is assumed as 1.0. When M_b is zero, C_m is taken as 1.0. The program defaults C_m 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 C_m for any member.

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

SAP2000 design assumes the analysis includes P-Δ effects, therefore δ_s is taken as unity for bending in both directions. It is suggested that the P-Δ analysis be done at the factored load level of 1.25 DL plus 1.35 LL (AASHTO C4.5.3.2.1). See also White and Hajjar (1991). 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 δ_b and δ_s for any member.

Calculation of Nominal Strengths

The nominal strengths in compression, tension, bending, and shear are computed for Compact and Non-compact sections according to the following subsections. The strength reduction factor, φ, is taken as follows (AASHTO 6.5.4.2):

φ_f = Resistance factor for bending, 1.0 (AASHTO 6.5.4.2, 6.10.2)
φ_v = Resistance factor for shear, 1.0 (AASHTO 6.5.4.2, 6.10.2)
φ_y = Resistance factor for tension, 0.95 (AASHTO 6.5.4.2, 6.8.2)
φ_c = Resistance factor for compression, 0.9 (AASHTO 6.5.4.2, 6.9.2)

For 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.

The AASHTO design in SAP2000 is limited to noncomposite, nonhybrid and unstiffened sections. The user must separately investigate this reduction if such sections are used.

If the user specifies nominal strengths for one or more elements in the "Redefine Element Design Data", these values ***will override all the above mentioned calculated values for those elements*** *as defined in the following subsections.*

Compression Capacity

The nominal axial compressive strength, P_n, depends on the slenderness ratio, $\frac{Kl}{r}$, and its critical value, λ_c. $\frac{Kl}{r}$ is the larger of $\frac{K_{33}\, l_{33}}{r_{33}}$ and $\frac{K_{22}\, l_{22}}{r_{22}}$, and

$$\lambda_c = \left(\frac{Kl}{r\pi}\right)^2 \frac{F_y}{E} . \qquad \text{(AASHTO 6.9.4.1)}$$

P_n is evaluated for flexural buckling as follows:

$$P_n = \left(0.66^{\lambda_c}\right) F_y A_g, \quad \text{for} \quad \lambda_c \leq 2.25 \text{, and} \qquad \text{(AASHTO 6.9.4.1)}$$

$$P_n = \left[\frac{0.88}{\lambda_c}\right] F_y A_g, \quad \text{for} \quad \lambda_c > 2.25 . \qquad \text{(AASHTO 6.9.4.1)}$$

For single angles r_z is used in place of r_{22} and r_{33}. For members in compression, if $\frac{Kl}{r}$ is greater than 120, a message to that effect is printed (AASHTO 6.9.3).

In computing the column compression capacity, the sections are assumed to satisfy the slenderness requirements given below:

$$\frac{b}{t} \le k\sqrt{\frac{E}{F_y}}\,, \qquad \text{(AASHTO 6.9.4.2)}$$

where the constant k ranges between 0.56 and 1.86 depending on the supports of the outstanding elements of the sections (AASHTO Table 6.9.4.2-1). If this slenderness criteria is not satisfied, it is suggested that AISC-LRFD (1986) code should be used (AASHTO C6.9.4.1). The users are specifically expected to consult AISC-LRFD for this situation, because the current version of SAP2000 does not consider this slenderness criteria.

Tension Capacity

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

$$P_n = A_g F_y \qquad \text{(AASHTO 6.8.2.1)}$$

It should be noted that no net section checks are made. For members in tension, if l/r is greater than 140, a message to that effect is printed (AASHTO 6.8.4).

Flexure Capacity

The nominal bending strength depends on the following criteria: the geometric shape of the cross-section, the axis of bending, the compactness of the section, and a slenderness parameter for lateral-torsional buckling. The nominal bending strength is the minimum value obtained from yielding, lateral-torsional buckling, flange local buckling, and web local buckling.

The nominal moment capacity about the minor axis is always taken to be the plastic moment capacity about the minor axis unless as specified below.

$$M_{n22} = M_{p22} = Z_{22} F_y \,.$$

However, the moment capacity about the major axis is determined depending on the shapes as follows.

General Section

General Sections are considered to be noncompact and their nominal moment capacity about the major axis is given by

$$M_n = S\,F_y \,.$$

I-Section

For compact I sections the moment capacity about the major axis is given as:

$$M_n = Z F_y \qquad \text{(AASHTO 6.10.6.2, 6.10.5.2.3a, 6.10.5.1.3)}$$

For noncompact I sections the moment capacity about the major axis is given as:

$$M_n = R_h R_b S F_y \,, \qquad \text{(AASHTO 6.10.6.3.1, 6.10.5.3.2a, 6.10.5.3.1)}$$

where R_h is the *hybrid factor*,

$$R_h = 1.0 \,, \text{ for nonhybrid sections, and} \qquad \text{(AASHTO 6.10.5.4.1a)}$$

R_b is the *load shedding factor*, and for nonhybrid sections,

$$R_b = \begin{cases} 1.0 \,, & \text{if } \dfrac{2D_c}{t_w} \le \lambda_b \sqrt{\dfrac{E}{F_y}} \,, \\ 1 - \dfrac{a_r}{1200 + 300 a_r} \left(\dfrac{2D_c}{t_w} - \lambda_b \sqrt{\dfrac{E}{f_c}} \right), & \text{if } \dfrac{2D_c}{t_w} > \lambda_b \sqrt{\dfrac{E}{F_y}} \,, \end{cases}$$

(6.10.5.4.2a)

where

$$a_r = \frac{2D_c t_w}{b_f t_f} \,, \text{ and} \qquad \text{(AASHTO 6.10.5.4.2a)}$$

$$\lambda_b = 5.76 \,. \qquad \text{(AASHTO 6.10.5.4.2a)}$$

For slender unstiffened I sections, when the unbraced length of the compression flange, L_b, exceeds the criteria for noncompactness $\left(L_b > 1.76 \, r_t \sqrt{E / F_y} \right)$ (AASHTO 6.10.5.3.3d), and the web slenderness and the compression flange slenderness criteria for noncompact sections are satisfied (AASHTO 6.10.5.3.2b, 6.10.5.3.3c), the moment capacity about the major axis is given as follows (AASHTO 6.10.6.4.1):

If $\frac{2D_c}{t_w} \leq \lambda_b \sqrt{\frac{E}{F_y}}$, then

$$M_n = 3.14\ EC_b R_h \left(\frac{I_{22}}{L_b}\right) \sqrt{0.772 \left(\frac{J}{I_{22}}\right) + 9.87 \left(\frac{d}{L_b}\right)^2} \leq R_h M_y \text{ , (6.10.6.4.1)}$$

if $\frac{2D_c}{t_w} > \lambda_b \sqrt{\frac{E}{F_y}}$ and $L_p < L_b \leq L_r$, then

$$M_n = C_b R_b R_h M_y \left[1.0 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq R_b R_h M_y \text{ , and} \qquad \text{(6.10.6.4.1)}$$

if $\frac{2D_c}{t_w} > \lambda_b \sqrt{\frac{E}{F_y}}$ and $L_b > L_r$, then

$$M_n = C_b R_b R_h \frac{M_y}{2} \left(\frac{L_r}{L_b}\right)^2 \leq R_b R_h M_y \text{ ,} \qquad \text{(AASHTO 6.10.6.4.1)}$$

where,

$$J = \frac{d\, t_w^3}{3} + \sum \frac{b_f t_f^3}{3} \text{ ,} \qquad \text{(AASHTO 6.10.6.4.1)}$$

$$L_p = 1.76\ r_t \sqrt{\frac{E}{F_y}} \text{ ,} \qquad \text{(AASHTO 6.10.6.4.1)}$$

$$L_r = \sqrt{19.71 \frac{I_y d}{S_{33}} \frac{E}{F_y}} \text{ ,} \qquad \text{(AASHTO 6.10.6.4.1)}$$

$$\lambda_b = 5.76 \text{ , and} \qquad \text{(AASHTO 6.10.6.4.1)}$$

$$C_b = 1.75 - 1.05(M_a / M_b) + 0.3(M_a / M_b)^2 \leq 2.3 \text{ .} \qquad \text{(AASHTO 6.10.5.5.2)}$$

C_b is the *moment gradient correction factor*, M_a / M_b is the ratio of the smaller to the larger moments at the ends of the member, M_a / M_b being positive for single curvature bending and negative for double curvature bending. When M_b is zero, C_b is taken as 1.0. The program also defaults C_b 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 C_b for any member.

r_t is the minimum radius of gyration taken about the vertical axis of the compression flange plus one-third of the web in compression (AASHTO 6.10.5.3.3d).

For slender unstiffened I sections, when the compression flange exceeds the criteria for noncompactness, i.e. $b_f/2t_f > 1.38\sqrt{E/\left(f_c\sqrt{2D_c/t_w}\right)}$, (AASHTO 6.10.5.3.3c), but $b_f/2t_f \le 2.52\sqrt{E/\left(f_c\sqrt{2D_{cp}/t_w}\right)}$ and the compression flange bracing and the web slenderness requirements are satisfied for noncompact sections (AASHTO 6.10.5.3.3d, 6.10.5.3.2b), the moment capacity about the major axis is given as follows (AASHTO 6.10.5.6.2):

$$M_n = \left[1.0 - \left(1.0 - \frac{0.7}{M_p/M_y}\right)\left(\frac{Q_p - Q_{fl}}{Q_p - 0.7}\right)\right] M_p \le M_p\,, \qquad (6.10.5.6.2)$$

where,

$Q_p = 3.0$, and (AASHTO 6.10.5.6.2)

$$Q_{fl} = \begin{cases} \dfrac{30.5}{\sqrt{\dfrac{2D_{cp}}{t_w}}}, & \text{if } \dfrac{b_f}{2t_f} \le 0.382\sqrt{\dfrac{E}{F_y}}, \\[2ex] \dfrac{4.45}{\left(\dfrac{b_f}{2t_f}\right)^2 \sqrt{\dfrac{2D_{cp}}{t_w}}}\dfrac{E}{F_y}, & \text{if } \dfrac{b_f}{2t_f} > 0.382\sqrt{\dfrac{E}{F_y}}. \end{cases} \qquad \text{(AASHTO 6.10.5.6.2)}$$

Box Section

Noncomposite Box Sections are considered to be noncompact and their nominal moment capacity about the major axis is given as follows:

$$M_n = \left[1 - 0.064\frac{F_y S\, l_{22}}{AE}\sqrt{\frac{2\left(d_w/t_w + b_f/t_f\right)}{I_{22}}}\right] SF_y \le M_p \qquad (6.12.2.2.2)$$

Pipe Section

For compact Pipe sections ($D/t < 2\sqrt{E/F_y}$) the moment capacity about the major axis is given as:

$$M_n = Z F_y \qquad \text{(AASHTO 6.12.2.2.3)}$$

For noncompact Pipe sections ($2\sqrt{E/F_y} < D/t \le 8.8\sqrt{E/F_y}$) the moment capacity about the major axis is given as:

$$M_n = S F_y \qquad \text{(AASHTO 6.12.2.2.3)}$$

Circular Bar

Solid Circular Bars are not subjected to lateral-torsional buckling. They are considered to be compact and their nominal moment capacity about the major axis is given by

$$M_n = Z F_y .$$

Rectangular and Channel Sections

The nominal moment capacity of Rectangular and Channel Sections about the major axis is computed according to AISC-LRFD 1986 based on yielding and Lateral-Torsional-Buckling limit states as follows (AASHTO 6.12.2.2.4a):

For channels and rectangular bars bent about the major axis, if $L_b \le L_p$

$$M_{n33} = M_{p33} ,$$

if $L_p < L_b \le L_r$

$$M_{n33} = C_b \left[M_{p33} - (M_{p33} - M_{r33}) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_{p33} , \qquad \text{(LRFD F1-3)}$$

and if $L_b > L_r$,

$$M_{n33} = M_{cr33} \le C_b M_{r33} \le M_{p33} , \qquad \text{(LRFD F1-12)}$$

where

M_{n33} = Nominal major bending strength,
M_{p33} = Major plastic moment, $Z_{33} F_y \le 1.5 S_{33} F_y$,
M_{r33} = Major limiting buckling moment,
$(F_y - F_r) S_{33}$ for channels, (LRFD F1-7)

and $F_y S_{33}$ for rectangular bars, (LRFD F1-11)

M_{cr33} = Critical elastic moment,

$$\frac{C_b \pi}{L_b}\sqrt{EI_{22}GJ + \left(\frac{\pi E}{L_b}\right)^2 I_{22} C_w} \quad \text{for channels, and (LRFD F1-13)}$$

$$\frac{57000\, C_b \sqrt{JA}}{L_b / r_{22}} \quad \text{for rectangular bars, (LRFD F1)}$$

L_b = Laterally unbraced length, l_{22} ,

L_p = Limiting laterally unbraced length for full plastic capacity,

$$\frac{300\ r_{22}}{\sqrt{F_y}} \quad \text{for channels, and (LRFD F1-4)}$$

$$\frac{3750\ r_{22}}{M_{p33}}\sqrt{JA} \quad \text{for rectangular bars, (LRFD F1-5)}$$

L_r = Limiting laterally unbraced length for inelastic lateral-torsional buckling,

$$\frac{r_{22} X_1}{F_y - F_r}\left\{1 + \sqrt{1 + X_2 (F_y - F_r)^2}\right\}^{½} \text{for channels, (LRFD F1-6)}$$

$$\frac{57000\ r_{22}\sqrt{JA}}{M_{r33}} \quad \text{for rectangular sections, (LRFD F1-10)}$$

$$X_1 = \frac{\pi}{S_{33}}\sqrt{\frac{EGJA}{2}}, \quad \text{(LRFD F1-8)}$$

$$X_2 = 4\frac{C_w}{I_{22}}\left(\frac{S_{33}}{GJ}\right)^2, \quad \text{(LRFD F1-9)}$$

$$C_b = 1.75 - 1.05(M_a / M_b) + 0.3(M_a / M_b)^2 \leq 2.3. \quad \text{(AASHTO 6.10.5.5.2)}$$

For non-compact channels, the nominal bending strengths are not taken greater than that given by the formulas below for the various local buckling modes possible for these sections. The nominal flexural strength M_n for the limit state of flange and web local buckling is:

For major direction bending

$$M_{n33} = M_{p33} - (M_{p33} - M_{r33})\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right), \quad \text{(LRFD A-F1-3)}$$

and for minor direction bending

$$M_{n22} = M_{p22} - (M_{p22} - M_{r22})\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right), \qquad \text{(LRFD A-F1-3)}$$

where,

M_{r33} = Major limiting buckling moment, (LRFD Table A-F1.1)
$(F_y - F_r)S_{33}$ for flange buckling of channels, and
$F_y S_{33}$ for web buckling of channels,

M_{r22} = Minor limiting buckling moment, (LRFD Table A-F1.1)
$F_y S_{22}$ or flange buckling of channels,

λ = Controlling slenderness parameter,

λ_p = Largest value of λ for which $M_n = M_p$, and

λ_r = Largest value of λ for which buckling is inelastic.

T-Sections and Double Angles

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

$$M_{n33} = C_b \frac{\pi\sqrt{EI_{22}GJ}}{L_b}\left[B + \sqrt{1 + B^2}\right] \leq F_y S_{33}, \text{ where} \qquad \text{(LRFD F1-15)}$$

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

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).

Single Angles

For single angles the nominal major and minor direction bending strengths are assumed as,

$$M_n = S F_y.$$

Shear Capacities

Major Axis of Bending

The nominal shear strength, V_{n2}, for major direction shears in I-shapes, boxes and channels is evaluated assuming unstiffened girders as follows (AASHTO 6.10.7):

For $\frac{d}{t_w} \leq 2.46\sqrt{\frac{E}{F_y}}$,

$$V_{n2} = 0.58 F_y A_w, \qquad \text{(AASHTO 6.10.7.2)}$$

for $2.46\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 3.07\sqrt{\frac{E}{F_y}}$,

$$V_{n2} = 1.48\, t_w^2 \sqrt{EF_y}\,, \text{ and} \qquad \text{(AASHTO 6.10.7.2)}$$

for $\frac{d}{t_w} > 3.07\sqrt{\frac{E}{F_y}}$,

$$V_{n2} = 4.55 \frac{t_w^3 E}{d}. \qquad \text{(AASHTO 6.10.7.2)}$$

The nominal shear strength for all other sections is taken as:

$$V_{n2} = 0.58 F_y A_{v2}.$$

Minor Axis of Bending

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

$$V_{n3} = 0.58 F_y A_{v3}$$

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 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.

Axial and Bending Stresses

The interaction ratio is determined based on the ratio $\frac{P_u}{\varphi P_n}$. If P_u is tensile, P_n is the nominal axial tensile strength and $\varphi = \varphi_t = 0.95$; and if P_u is compressive, P_n is the nominal axial compressive strength and $\varphi = \varphi_c = 0.90$. In addition, the resistance factor for bending, $\varphi_f = 1.0$.

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

$$\frac{P_u}{2\varphi P_n} + \left(\frac{M_{u33}}{\varphi_f M_{n33}} + \frac{M_{u22}}{\varphi_f M_{n22}} \right). \qquad \text{(AASHTO 6.8.2.3, 6.9.2.2)}$$

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

$$\frac{P_u}{\varphi P_n} + \frac{8}{9} \left(\frac{M_{u33}}{\varphi_f M_{n33}} + \frac{M_{u22}}{\varphi_f M_{n22}} \right). \qquad \text{(AASHTO 6.8.2.3, 6.9.2.2)}$$

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.

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 produced as follows:

$$\frac{V_{u2}}{\varphi_v V_{n2}}, \text{ and}$$

$$\frac{V_{u3}}{\varphi_v V_{n3}}.$$

Chapter VI

Check/Design for CISC94

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 when the user selects the CAN/CSA-S16.1-94 design code (CISC 1995). Various notations used in this chapter are described in Table VI-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.

A	=	Cross-sectional area, mm^2
A_g	=	Gross cross-sectional area, mm^2
A_{v2}, A_{v3}	=	Major and minor shear areas, mm^2
A_w	=	Shear area, mm^2
C_e	=	Euler buckling strength, N
C_f	=	Factored compressive axial load, N
C_r	=	Factored compressive axial strength, N
C_w	=	Warping constant, mm^6
C_y	=	Compressive axial load at yield stress, $A_g F_y$, N
D	=	Outside diameter of pipes, mm
E	=	Modulus of elasticity, MPa
F_y	=	Specified minimum yield stress, MPa
G	=	Shear modulus, MPa
I_{33}, I_{22}	=	Major and minor moment of inertia, mm^4
J	=	Torsional constant for the section, mm^4
K	=	Effective length factor
K_{33}, K_{22}	=	Effective length K-factors in the major and minor directions (assumed as 1.0 unless overwritten by user)
L	=	Laterally unbraced length of member, mm
M_{f33}, M_{f22}	=	Factored major and minor bending loads, N-mm
M_{p33}, M_{p22}	=	Major and minor plastic moments, N-mm
M_{r33}, M_{r22}	=	Factored major and minor bending strengths, N-mm
M_u	=	Critical elastic moment, N-mm
M_{y33}, M_{y22}	=	Major and minor yield moments, N-mm
S_{33}, S_{22}	=	Major and minor section moduli, mm^3
T_f	=	Factored tensile axial load, N
T_r	=	Factored tensile axial strength, N
U_1	=	Moment magnification factor to account for deformation of member between ends
U_2	=	Moment magnification factor (on sidesway moments) to account for P-Δ
V_{f2}, V_{f3}	=	Factored major and minor shear loads, N
V_{r2}, V_{r3}	=	Factored major and minor shear strengths, N
Z_{33}, Z_{22}	=	Major and minor plastic moduli, mm^3

Table VI-1
CISC 94 Notations

b	=	Nominal dimension of longer leg of angles $(b_f - 2t_w)$ for welded $(b_f - 3t_f)$ for rolled box sections, mm
b_f	=	Flange width, mm
d	=	Overall depth of member, mm
h	=	Clear distance between flanges , taken as $(d - 2t_f)$, mm
k	=	Web plate buckling coefficient, assumed as 5.34 (no stiffeners)
k	=	Distance from outer face of flange to web toe of fillet , mm
l	=	Unbraced length of member, mm
l_{33}, l_{22}	=	Major and minor direction unbraced member lengths, mm
r	=	Radius of gyration, mm
r_{33}, r_{22}	=	Radii of gyration in the major and minor directions, mm
r_z	=	Minimum Radius of gyration for angles, mm
t	=	Thickness, mm
t_f	=	Flange thickness, mm
t_w	=	Web thickness, mm
λ	=	Slenderness parameter
φ	=	Resistance factor, taken as 0.9
ω_1	=	Moment Coefficient
ω_{13}, ω_{12}	=	Major and minor direction moment coefficients
ω_2	=	Bending coefficient

Table VI-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, SAP2000 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 VI-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 SAP2000 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} and V_{f3} 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 SAP2000 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:

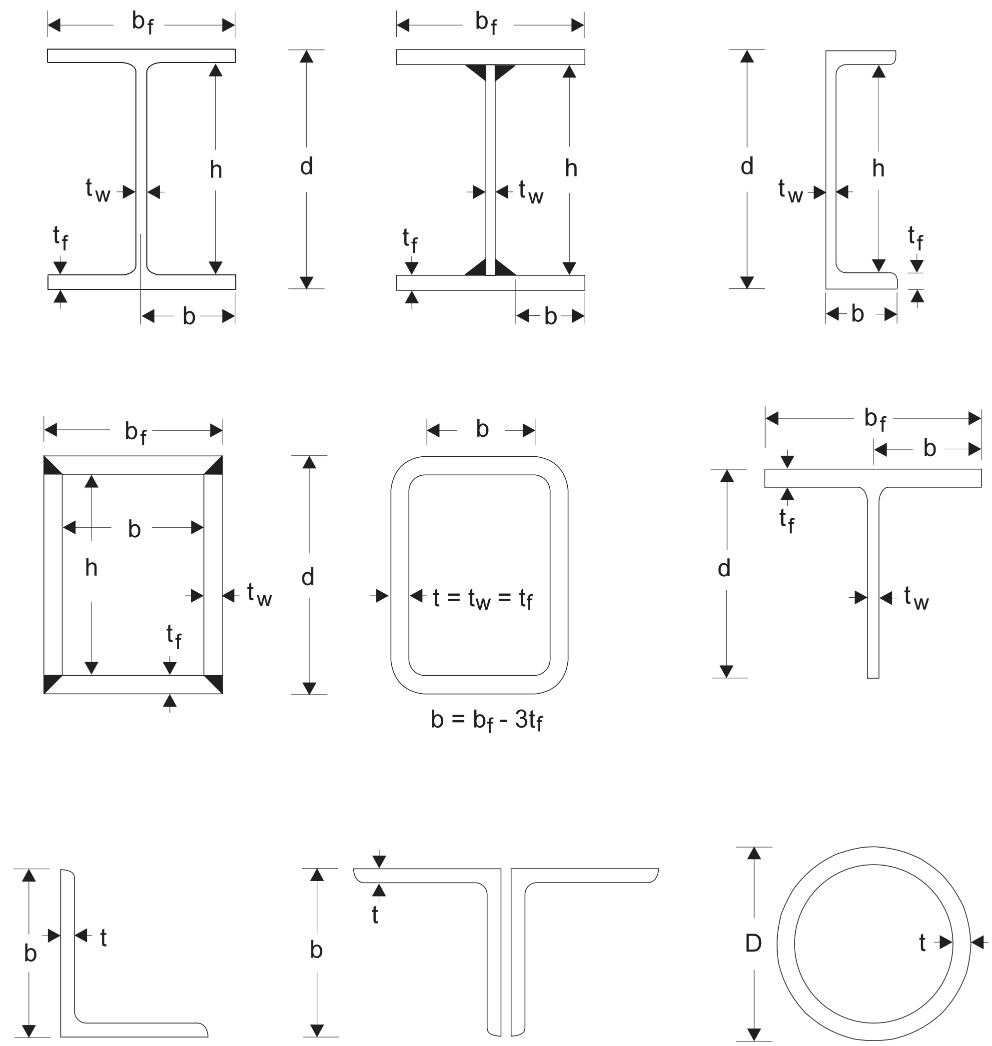
$$M_f = M_{fg} + U_2 M_{ft}, \text{ where} \qquad \text{(CISC 8.6.1)}$$

U_2 = Moment magnification factor for sidesway moments,
M_{fg} = Factored moments not causing translation, and
M_{ft} = Factored moments causing sidesway.

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Noncompact)
I-SHAPE	$b_f / 2t_f$	$\le 145 / \sqrt{F_y}$	$\le 170 / \sqrt{F_y}$	$\le 200 / \sqrt{F_y}$
	h / t_w	$\le \frac{1100}{\sqrt{F_y}}\left(1 - 0.39\frac{C_f}{C_y}\right)$	$\le \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_f}{C_y}\right)$	$\le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{C_y}\right)$
BOX	b / t_f	$\le 420 / \sqrt{F_y}$ (rolled) $\le 525 / \sqrt{F_y}$ (welded)	$\le 525 / \sqrt{F_y}$	$\le 670 / \sqrt{F_y}$
	h / t_w	As for I-shapes	As for I-shapes	As for I-shapes
CHANNEL	b_f / t_f h / t_w	Not applicable Not applicable	Not applicable Not applicable	$\le 200 / \sqrt{F_y}$ As for I-shapes
T-SHAPE	$b_f / 2t_f$ d / t_w	Not applicable Not applicable	Not applicable Not applicable	$\le 200 / \sqrt{F_y}$ $\le 340 / \sqrt{F_y}$
DOUBLE ANGLE	b / t	Not applicable	Not applicable	$\le 200 / \sqrt{F_y}$
ANGLE	b / t	Not applicable	Not applicable	$\le 200 / \sqrt{F_y}$
PIPE (Flexure)	D / t	$\le 13000 / F_y$	$\le 18000 / F_y$	$\le 66000 / F_y$
PIPE (Axial)	D / t	—	—	$\le 23000 / F_y$
ROUND BAR	—	Assumed Class 2		
RECTAN-GULAR	—	Assumed Class 2		
GENERAL	—	Assumed Class 3		

Table VI-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.

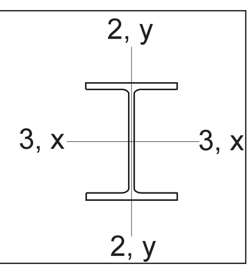


Figure VI-1

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 SAP2000. The strength reduction factor, φ, 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 nominal strengths for one or more elements in the "Redefine Element Design Data", these values ***will override all the above mentioned calculated values for those elements*** *as defined in the following subsections.*

Compression Strength

The factored axial compressive strength value, C_r, for Class 1, 2, or 3 sections depends on a factor, λ, which eventually depends on the slenderness ratio, Kl/r, which is the larger of $\left(K_{33}\ l_{33}\right)/r_{33}$ and $\left(K_{22}\ l_{22}\right)/r_{22}$, and is defined as

$$\lambda = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} .$$

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):

$$C_r = \varphi\, AF_y\left(1+\lambda^{2n}\right)^{-\frac{1}{n}}, \text{ where} \qquad \text{(CISC 13.3.1)}$$

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). SAP2000 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 SAP2000 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 $\varphi A_g F_y$ (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).

$$T_r = \varphi A_g F_y \qquad \text{(CISC 13.2)}$$

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, $M_r = \varphi ZF_y$, and (CISC 13.5)

For Class 3, $M_r = \varphi SF_y$. (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 $M_u > 0.67\, M_{p33}$,

$$M_{r3} = 1.15\varphi\, M_{p33}\left(1\text{-}0.28\,\frac{M_{p33}}{M_u}\right) \leq \varphi\, M_{p33} \text{ , and} \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = \varphi\, M_u \text{ , where} \qquad \text{(CISC 13.6)}$$

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

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

L = Laterally unbraced length, l_{22} ,
C_w = Warping constant assumed as 0.0 for boxes, pipes, rectangular and circular bars, and

$$\omega_2 = 1.75 + 1.05\left(\frac{M_a}{M_b}\right) + 0.30\left(\frac{M_a}{M_b}\right)^2 \leq 2.5\,. \qquad \text{(CISC 13.6)}$$

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 positive for double curvature bending and negative for single curvature bending. If any moment within the segment is greater than M_b, ω_2 is taken as 1.0. The program defaults ω_2 to 1.0 if the unbraced length, l of the member is overwritten by the user (i.e. it is not equal to the length of the member). ω_2 should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. The user can overwrite the value of ω_2 for any member by specifying it.

For Class 3 sections of I-shapes, channels, boxes bent about the major axis,

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

$$M_{r33} = 1.15\varphi\ M_{y33}\left(1 - 0.28\ \frac{M_{y33}}{M_u}\right) \leq \varphi\ M_{y33}, \text{ and} \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = \varphi M_u, \quad \text{where} \qquad \text{(CISC 13.6)}$$

M_{r33} and M_u are as defined earlier for Class 1 and 2 sections and M_{y33} is the major yield moment, $S_{33}F_y$.

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 $M_u > 0.67\ M_{p33}$,

$$M_{r33} = 1.15\varphi\ M_{p33}\left(1\text{-}0.28\ \frac{M_{p33}}{M_u}\right) \leq \phi\ M_{p33}\ , \text{ and} \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = \varphi\ M_u\ . \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = 1.15\, \varphi\, M_{p33} \left(1\text{-}0.28 \frac{M_{p33}}{M_u} \right) \leq \varphi\, M_{p33} \text{, and} \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = \varphi\, M_u\, . \qquad \text{(CISC 13.6)}$$

Channel Sections

Major Axis of Bending

For Class 3 channel sections bent about their major axis,

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

$$M_{r33} = 1.15 \varphi\, M_{y33} \left(1 - 0.28 \frac{M_{y33}}{M_u} \right) \leq \varphi\, M_{y33} \text{, and} \qquad \text{(CISC 13.6)}$$

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

$$M_{r33} = \varphi\, M_u\, .$$

Minor Axis of Bending

For Class 3 channel sections bent about their minor axis,

$$M_{r22} = M_{y22} = S_{22} F_y\, .$$

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),

$$M_{r33} = \varphi \frac{\omega_2 \pi \sqrt{E I_{22} G J}}{L} \left[B + \sqrt{1 + B^2} \right] \leq \phi\, F_y S_{33} \text{, where}$$

$$B = \pm\; 2.3 (d/L) \sqrt{I_{22} \,/\, J}\, .$$

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,

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

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,

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

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

Shear Strengths

The factored shear strength, V_{r2}, for major direction shears in I-shapes, boxes and channels is evaluated as follows (CISC 13.4.1.1):

- For $\frac{h}{t_w} \leq 439\sqrt{\frac{k_v}{F_y}}$,

$$V_{r2} = \varphi A_w \left\{0.66 F_y\right\} . \qquad \text{(CISC 13.4.1.1)}$$

- For $439\sqrt{\frac{k_v}{F_y}} < \frac{h}{t_w} \leq 502\sqrt{\frac{k_v}{F_y}}$,

$$V_{r2} = \varphi A_w \left\{290 \frac{\sqrt{k_v F_y}}{h / t_w}\right\} . \qquad \text{(CISC 13.4.1.1)}$$

- For $502\sqrt{\frac{k_v}{F_y}} < \frac{h}{t_w} \leq 621\sqrt{\frac{k_v}{F_y}}$,

$$V_{r2} = \varphi A_w \left\{F_{cri} + F_t\right\} , \text{ where} \qquad \text{(CISC 13.4.1.1)}$$

$$F_{cri} = 290 \frac{\sqrt{k_v F_y}}{h / t_w}, \text{ and}$$

$$F_t = \left(0.5 F_y - 0.866 F_{cri}\right) \left\{ \frac{1}{\sqrt{1 + (a/h)^2}} \right\}.$$

Assuming no stiffener is used, the value of F_t is taken as zero.

- For $\frac{h}{t_w} > 621 \sqrt{\frac{k_v}{F_y}}$,

$$V_{r2} = \varphi A_w \left\{ F_{cre} + F_t \right\}, \text{ where} \qquad \text{(CISC 13.4.1.1)}$$

$$F_{cre} = \frac{180000\, k_v}{(h/t_w)^2}.$$

In the above equations, k_v is the shear buckling coefficient, and it is defined as:

$$k_v = 4 + \frac{5.34}{(a/h)^2}, \qquad a/h < 1$$

$$k_v = 5.34 + \frac{4}{(a/h)^2}, \qquad a/h \geq 1$$

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

$$V_{r2} = 0.66\, \varphi\, F_y A_{v3}. \qquad \text{(CISC 13.4.2)}$$

The factored shear strength for major and minor direction shears for all other sections is assumed as (CISC 13.4.2):

$$V_{r2} = 0.66\, \varphi\, F_y A_{v2}, \text{ and} \qquad \text{(CISC 13.4.2)}$$

$$V_{r3} = 0.66\, \varphi\, F_y A_{v3}. \qquad \text{(CISC 13.4.2)}$$

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:

$$\frac{C_f}{C_r} + \frac{U_{13} M_{f33}}{M_{r33}} + \frac{U_{12} M_{f22}}{M_{r22}}, \text{ for all but Class 1 I-shaped sections} \quad (13.8.1)$$

$$\frac{C_f}{C_r} + 0.85 \frac{U_{13} M_{f33}}{M_{r33}} + 0.6 \frac{U_{12} M_{f22}}{M_{r22}}, \text{ for Class 1 I-shaped sections} \quad (13.8.2)$$

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 $\lambda = 0$.

    $C_r = \varphi A F_y$ (CISC 13.3.1)

  - The M_{r33} and M_{r22} are calculated assuming that the member is laterally fully supported ($l_{22} = 0$ and $l_{33} = 0$) irrespective of its actual lateral bracing length (CISC 13.5), and
  - U_{12} and U_{13} are taken as 1.

    $U_{13} = U_{12} = 1.0$. (CISC 13.8.1, 13.8.2)

  If the capacities (C_r, M_{r22} and M_{r33}) 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 $\frac{K_{22}\, l_{22}}{r_{22}}$ and $\frac{K_{33}\, l_{33}}{r_{33}}$ as described in an earlier section (CISC 13.3.1) .
  - M_{r33} and M_{r22} are calculated assuming that the member is laterally fully supported ($l_{22} = 0$ and $l_{33} = 0$) irrespective of its actual lateral bracing length (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_{r22}, and M_{r33}) 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 $\frac{K_{22}\, l_{22}}{r_{22}}$ (CISC 13.3.1),
  - M_{r33} and M_{r22} 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,

$U_{13} \geq 1$ is enforced. (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:

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

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,

$$U_1 = \frac{\omega_1}{1 - C_f / C_e}, \qquad \text{(CISC 13.8.3)}$$

$$C_e = \frac{\pi^2 E I}{L^2},$$

$$\omega_1 = 0.6 - 0.4 \, M_a / M_b \geq 0.4, \text{ and}$$

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. ω_1 is assumed as 1.0 for beams with transverse load and when M_b is zero.

The program defaults ω_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 ω_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

$$\left[\frac{T_f}{T_r}\right] + \left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right], \qquad \text{(CISC 13.9)}$$

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

In the second case the ratio is calculated as

$$\left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right] - \left[\frac{T_f\ Z_{33}}{M_{r33}\ A}\right] \quad \text{(for Class 1 and 2), or} \qquad \text{(CISC 13.9)}$$

$$\left[\frac{M_{f33}}{M_{r33}} + \frac{M_{f22}}{M_{r22}}\right] - \left[\frac{T_f\ S_{33}}{M_{r33}\ A}\right] \quad \text{(for Class 3).} \qquad \text{(CISC 13.9)}$$

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

For circular 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:

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

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

Chapter VII

Check/Design for BS 5950

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 when the user selects the BS 5950 design code (BSI 1990). 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.

A	=	Cross-sectional area, mm^2
A_g	=	Gross cross-sectional area, mm^2
A_{v2}, A_{v3}	=	Major and minor shear areas, mm^2
B	=	Breadth, mm
D	=	Depth of section, mm or outside diameter of pipes, mm
E	=	Modulus of elasticity, MPa
F_c	=	Axial compression, N
F_t	=	Axial tension, N
F_{v2}, F_{v3}	=	Major and minor shear loads, N
G	=	Shear modulus, MPa
H	=	Warping constant, mm^6
I_{33}	=	Major moment of inertia, mm^4
I_{22}	=	Minor moment of inertia, mm^4
J	=	Torsional constant for the section, mm^4
K	=	Effective length factor
K_{33}, K_{22}	=	Major and minor effective length factors
M	=	Applied moment, N-mm
M_{33}	=	Applied moment about major axis, N-mm
M_{22}	=	Applied moment about minor axis, N-mm
M_{a33}	=	Major maximum bending moment, N-mm
M_{a22}	=	Minor maximum bending moment, N-mm
M_b	=	Buckling resistance moment, N-mm
M_c	=	Moment capacity, N-mm
M_{c33}	=	Major moment capacity, N-mm
M_{c22}	=	Minor moment capacity, N-mm
M_E	=	Elastic critical moment, N-mm
P_c	=	Compression resistance, N
P_{c33}, P_{c22}	=	Major and minor compression resistance, N
P_t	=	Tension capacity, N
P_{v2}, P_{v3}	=	Major and minor shear capacities, N
S_{33}, S_{22}	=	Major and minor plastic section moduli, mm^3
T	=	Thickness of flange or leg, mm
Y_s	=	Specified yield strength, MPa
Z_{33}, Z_{22}	=	Major and minor elastic section moduli, mm^3

Table VII-1
BS 5950 Notations

a	=	Robertson constant
b	=	Outstand width, mm
d	=	Depth of web, mm
h	=	Story height, mm
k	=	Distance from outer face of flange to web toe of fillet , mm
l	=	Unbraced length of member, mm
l_{33}, l_{22}	=	Major and minor direction unbraced member lengths, mm
l_{e33}, l_{e22}	=	Major and minor effective lengths, mm ($K_{33} l_{33}$, $K_{22} l_{22}$)
m	=	Equivalent uniform moment factor
n	=	Slenderness correction factor
q_e	=	Elastic critical shear strength of web panel, MPa
q_{cr}	=	Critical shear strength of web panel, MPa
r_{33}, r_{22}	=	Major and minor radii of gyration, mm
r_z	=	Minimum radius of gyration for angles, mm
t	=	Thickness, mm
t_f	=	Flange thickness, mm
t_w	=	Thickness of web, mm
u	=	Buckling parameter
v	=	Slenderness factor
β	=	Ratio of smaller to larger end moments
ε	=	Constant $\left(\frac{275}{\rho_y}\right)^{1/2}$
λ	=	Slenderness parameter
λ_o	=	Limiting slenderness
λ_{LT}	=	Equivalent slenderness
λ_{Lo}	=	Limiting equivalent slenderness
η	=	Perry factor
η_{LT}	=	Perry coefficient
ρ_c	=	Compressive strength, MPa
ρ_E	=	Euler strength, MPa
ρ_y	=	Yield strength, MPa
ψ	=	Monosymmetry index

Table VII-1

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 $\pm$ 1.4 WL
1.4 DL $\pm$ 1.4 WL
1.2 DL + 1.2 LL $\pm$ 1.2 WL (BS 2.4.1.1)

1.0 DL $\pm$ 1.4 EL
1.4 DL $\pm$ 1.4 EL
1.2 DL + 1.2 LL $\pm$ 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 SAP2000 and define the appropriate design combinations.

When using the BS 5950 code, SAP2000 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. SAP2000 checks the sections according to Table VII-2 (BS 3.5.2). The parameters R, γ_c and ε 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 ρ_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:

  $$R = \frac{P}{A_g \rho_y}$$

- α is given as $2\gamma_c / d$, where γ_c 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.

  $$\alpha = \frac{\gamma_c}{d/2}$$

  $$\gamma_c = \begin{cases} \left(\frac{D}{2} - T\right) - \frac{P}{2\rho_y t}, & \text{for I and Channel section} \\ \left(\frac{D}{2} - T\right) - \frac{P}{4\rho_y t}, & \text{for Box and Double Channel section} \end{cases}$$

  In calculating γ_c, compression is taken as negative and tension is taken as positive.

- ε 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 SAP2000 does not check stresses for Slender sections.**

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Semi-compact)
I-SHAPE	b/T (Rolled)	$\leq 8.5\,\varepsilon$	$\leq 9.5\,\varepsilon$	$\leq 15\,\varepsilon$
	b/T (welded)	$\leq 7.5\,\varepsilon$	$\leq 8.5\,\varepsilon$	$\leq 13\,\varepsilon$
	d/t webs ($\alpha < 2$)	$\leq \frac{79\,\varepsilon}{0.4 + 0.6\,\alpha}$	$\leq \frac{98\,\varepsilon}{\alpha}$	For $R > 0$: $\leq \frac{120\,\varepsilon}{1+1.5R}$ and $\leq \left(\frac{41}{R} - 13\right)\varepsilon$ (welded) $\leq \frac{120\,\varepsilon}{1+1.5R}$ and $\leq \left(\frac{41}{R} - 2\right)\varepsilon$ (rolled) For $R = 0$: $\leq 120\,\varepsilon$, and For $R < 0$: $\leq \frac{120\,\varepsilon}{(1+R)^2}$ and $\leq 250\,\varepsilon$.
	d/t webs ($\alpha \geq 2$) (rolled)	$\leq 39\,\varepsilon$	$\leq 39\,\varepsilon$	$\leq 39\,\varepsilon$
	d/t webs ($\alpha \geq 2$) (welded)	$\leq 28\,\varepsilon$	$\leq 28\,\varepsilon$	$\leq 28\,\varepsilon$
BOX	b/T (Rolled)	$\leq 26\,\varepsilon$	$\leq 32\,\varepsilon$	$\leq 39\,\varepsilon$
	b/T (welded)	$\leq 23\,\varepsilon$	$\leq 25\,\varepsilon$	$\leq 28\,\varepsilon$
	d/t	As for I-shapes	As for I-shapes	As for I-shapes
CHANNEL	b/T d/t	As for I-shapes	As for I-shapes	As for I-shapes
T-SHAPE	b/T d/t	$\leq 8.5\,\varepsilon$ $\leq 8.5\,\varepsilon$	$\leq 9.5\,\varepsilon$ $\leq 9.5\,\varepsilon$	$\leq 19\,\varepsilon$ $\leq 19\,\varepsilon$
DOUBLE ANGLE (separated)	d/t $(b+d)/t$	$\leq 8.5\,\varepsilon$ $\leq 23\,\varepsilon$	$\leq 9.5\,\varepsilon$ $\leq 23\,\varepsilon$	$\leq 15\,\varepsilon$ $\leq 23\,\varepsilon$

Table VII-2
Limiting Width-Thickness Ratios for
Classification of Sections based on BS 5950

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Semi-compact)
ANGLE	b/t $(b+d)/t$	$\leq 8.5\,\varepsilon$ $\leq 23\,\varepsilon$	$\leq 9.5\,\varepsilon$ $\leq 23\,\varepsilon$	$\leq 15\,\varepsilon$ $\leq 23\,\varepsilon$
PIPE	D/t	$\leq 40\,\varepsilon^2$	$\leq 57\,\varepsilon^2$	$\leq 80\,\varepsilon^2$
SOLID CIRCLE	—	Assumed Compact		
SOLID RECTANGLE	—	Assumed Compact		
GENERAL	—	Assumed Semi-compact		

Table VII-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 or F_c, M_{33}, M_{22}, F_{v2}, and F_{v3} 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.

$$M = M_g + \left\{ \frac{1}{1 - 200\,\varphi_{s,max}} \right\} M_s, \text{ where} \qquad \text{(BS 5.6.3)}$$

$\varphi_{s,max}$ = Maximum story-drift divided by the story-height,
M_g = Factored moments not causing translation, and
M_s = Factored moments causing sidesway.

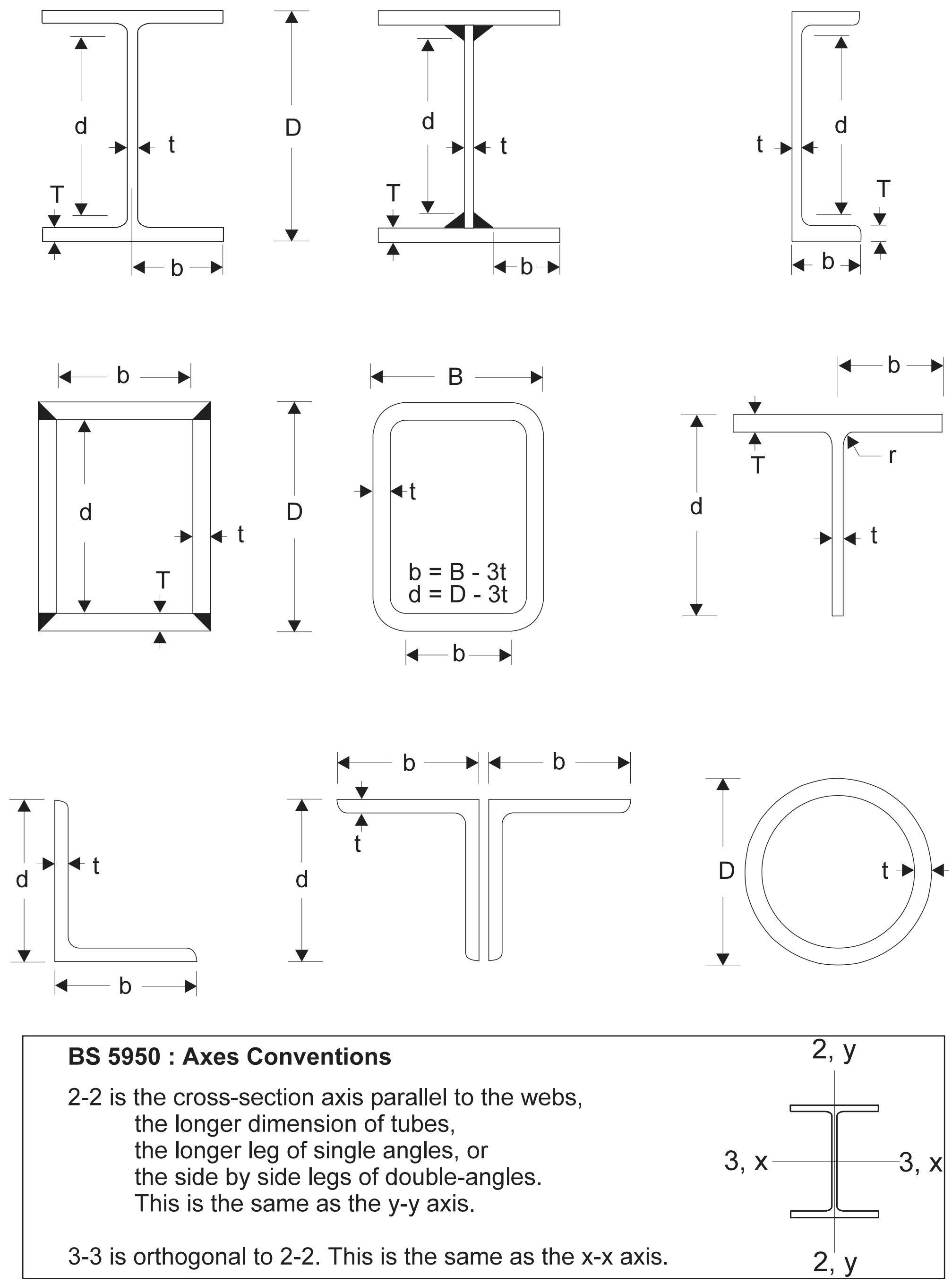


Figure VII-1

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. SAP2000 design assumes a P-Δ analysis has been done and, therefore, $\varphi_{s,max}$ 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 nominal strengths in compression, tension, bending, and shear are computed for Class 1, 2, and 3 sections according to the following subsections. By default, SAP2000 takes the design strength, ρ_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).

$$\rho_y = 1.0 Y_s \qquad \text{(BS 3.1.1)}$$

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 nominal strengths for one or more elements in the "Redefine Element Design Data", these values ***will override all above the mentioned calculated values for those elements*** *as defined in the following subsections.*

Compression Resistance

The compression resistance for plastic, compact, or semi-compact sections is evaluated as follows:

$$P_c = A_g \, \rho_c \, , \qquad \text{(BS 4.7.4)}$$

where ρ_c is the compressive strength given by

$$\rho_c = \frac{\rho_E \, \rho_y}{\varphi + (\varphi^2 - \rho_E \, \rho_y)^{1/2}} \text{, where} \qquad \text{(BS C.1)}$$

$$\varphi = \frac{\rho_y + (\eta + 1)\rho_E}{2} \, , \qquad \text{(BS C.1)}$$

Description of Section	Thickness (mm)	Axis of Bending	
		Major	Minor
I-SHAPE (rolled)	any	2.0	3.5
H-SHAPE (rolled)	≤ 40 > 40	3.5 5.5	5.5 8.0
I-SHAPE (welded)	≤ 40 > 40	3.5 3.5	5.5 8.0
BOX or Pipe (Rolled)	any	2.0	2.0
BOX (welded)	≤ 40 > 40	3.5 5.5	3.5 5.5
CHANNEL, T-SHAPE, ANGLE	any	5.5	5.5
RECTANGULAR or **CIRCLE**	≤ 40 > 40	3.5 5.5	3.5 5.5
GENERAL	any	5.5	5.5

Table VII-3
Robertson Constant in BS 5950

ρ_E = Euler strength, $\pi^2 E / \lambda^2$,

η = Perry factor, $0.001\, a\,(\lambda - \lambda_0) \geq 0$, (BS C.2)

a = Robertson constant from Table VII-3, (BS C2, BS Table 25)

λ_0 = Limiting slenderness, $0.2\left(\frac{\pi^2 E}{\rho_y}\right)^{1/2}$, and (BS C.2)

λ = the slenderness ratio in either the major, $\lambda_{33} = l_{e33} / r_{33}$, or in the minor, $\lambda_{22} = l_{e22} / r_{22}$ direction (BS 4.7.3.1). The larger of the two values is used in the above equations to calculate P_c.

For single angles r_z is used instead of r_{33} and r_{22}. For members in compression, if λ 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

$$P_t = A_g \rho_y \, . \qquad \text{(BS 4.6.1)}$$

It should be noted that no net section checks are made. For main members in tension, the slenderness, λ, should not be greater than 250 (BS 4.7.3.2). If λ 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_{c33} and M_{c22} 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

$$M_c = \rho_y S \leq 1.2 \rho_y Z \, , \qquad \text{for} \quad F_v \leq 0.6 P_v \, , \qquad \text{(BS 4.2.5)}$$

$$M_c = \rho_y (S - S_v \rho_1) \leq 1.2 \rho_y Z \, , \qquad \text{for} \quad F_v > 0.6 P_v \, , \qquad \text{(BS 4.2.6)}$$

where

S = Plastic modulus of the gross section about the relevant axis,

Z = 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,

ρ_1 = $\dfrac{2.5\,F_v}{P_v} - 1.5$.

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 SAP2000 as

$$M_c = \rho_y S \le 1.2\,\rho_y Z. \qquad \text{(BS 4.2.5)}$$

Semi-compact Sections

Reduction of moment capacity due to coexistent shear does not apply for semi-compact sections.

$$M_c = \rho_y Z \qquad \text{(BS 4.2.5)}$$

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

$$M_b = \frac{\rho_y\, S_{33}\, M_E}{\varphi_B + (\varphi_B^{\,2} - \rho_y\, S_{33} M_E)^{1/2}}, \text{ where} \qquad \text{(BS B2.1)}$$

$$\varphi_B = \frac{\rho_y S_{33} + (\eta_{LT} + 1) M_E}{2},$$

$M_E =$ The elastic critical moment, $\dfrac{S_{33}\pi^2 E}{\lambda_{LT}^{\;2}}$, and (BS B2.3)

$\eta_{LT} =$ The Perry coefficient.

The Perry coefficient, η_{LT}, for rolled and welded sections is taken as follows:

For rolled sections

$$\eta_{LT} = \alpha_b \left\{\lambda_{LT} - \lambda_{L0}\right\} \geq 0, \text{ and} \qquad \text{(BS B2.3)}$$

for welded sections

$$\eta_{LT} = 2\alpha_b \lambda_{L0} \geq 0, \text{ with } \alpha_b (\lambda_{LT} - \lambda_{L0}) \leq \eta_{LT} \leq 2\alpha_b (\lambda_{LT} - \lambda_{L0}). \text{(BS B2.2)}$$

In the above definition of η_{LT}, λ_{L0} and λ_{LT} are the limiting equivalent slenderness and the equivalent slenderness, respectively, and α_b is a constant. α_b is taken as 0.007 (BS 2.3). For flanged members symmetrical about at least one axis and uniform throughout their length, λ_{L0} is defined as follows:

$$\lambda_{L0} = 0.4\sqrt{\frac{\pi^2 E}{\rho_y}}, \qquad \text{(BS B2.4)}$$

For I, Channel, Double-Channel, and T sections λ_{LT} is defined as

$$\lambda_{LT} = n u v \lambda, \qquad \text{(BS B2.5)}$$

and for Box sections λ_{LT} is defined as

$$\lambda_{LT} = 2.25\, n\left(\phi_b \lambda\right)^{1/2}, \text{ where} \qquad \text{(BS B2.5)}$$

- λ 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).

  $$n = \frac{1}{\sqrt{C_b}} \leq 1.0, \text{ where}$$

$$C_b = \frac{12.5\, M_{max}}{2.5\, M_{max} + 3\, M_A + 4\, M_B + 3\, M_C}\text{, and}$$

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. The program also defaults C_b 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). 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 can overwrite the value of C_b 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,

$$u = \left(\frac{4 S_{33}^2 \gamma}{A^2 (D - T)^2} \right)^{1/4}\text{, for I, Channel, and Double-Channel,} \quad \text{(BS B2.5b)}$$

$$u = \left(\frac{I_{22} S_{33}^2 \gamma}{A^2 H} \right)^{1/4}\text{, for T section, where} \quad \text{(BS B2.5b)}$$

$$\gamma = \left(1 - \frac{I_{22}}{I_{33}} \right). \quad \text{(BS B2.5b)}$$

- v is the slenderness factor. For I, Channel, Double-Channel, and T sections, it is given by the following formula.

$$v = \frac{1}{\left[\left\{ 4N(N-1) + \frac{1}{20} \left[\frac{\lambda}{x} \right]^2 + \psi^2 \right\}^{1/2} + \psi \right]^{1/2}}\text{, where} \quad \text{(BS B2.5d)}$$

$$N = \begin{cases} 0.5, & \text{for I, Channel, Double-Channel sections,} \\ 1.0, & \text{for T sections with flange in compression,} \\ 0.0, & \text{for T sections with flange in tension, and} \end{cases} \quad \text{(BS B2.5d)}$$

$$\psi = \begin{cases} 0.0, & \text{for I, Channel, Double-Channel sections,} \\ 0.8, & \text{for T sections with flange in compression, and} \\ -1.0, & \text{for T sections with flange in tension.} \end{cases} \quad \text{(BS B2.5d)}$$

- ϕ_b is the buckling index for box section factor. It is given by the following formula. (BS B2.6.1).

$$\phi_b = \left(\frac{S_{33}^{2}\gamma'}{A^2 J} \right)^{1/2} \text{, where} \qquad \text{(BS B2.6.1)}$$

$$\gamma' = \left(1 - \frac{I_{22}}{I_{33}} \right) \left(1 - \frac{J}{2.6 I_{33}} \right). \qquad \text{(BS B2.6.1)}$$

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:

$$P_{v2} = 0.6\, \rho_y\, A_{v2}\text{, and} \qquad \text{(BS 4.2.3)}$$

$$P_{v3} = 0.6\, \rho_y\, A_{v3}\,. \qquad \text{(BS 4.2.3)}$$

The shear areas A_{v3} and A_{v2} are given in Table VII-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.

Description of Section	Condition	Axis of Bending	
		Major	Minor
I-SHAPE	Rolled Welded	tD td	$0.9\left(4bT\right)$ $0.9\left(4bT\right)$
CHANNEL	Rolled Welded	tD td	$0.9\left(2bT\right)$ $0.9\left(2bT\right)$
DOUBLE CHANNEL	Rolled Welded	$2.0\ tD$ $2.0\ td$	$2.0 * 0.9\left(2bT\right)$ $2.0 * 0.9\left(2bT\right)$
BOX	—	$\frac{D}{D+B}A$	$\frac{B}{D+B}A$
T-SHAPE	Rolled Welded	td $t\left(d-T\right)$	$0.9\left(2bT\right)$ $0.9\left(2bT\right)$
DOUBLE ANGLE	—	$2td$	$2bt$
ANGLE	—	td	bt
RECTANGULAR	—	$0.9\ A$	$0.9\ A$
CIRCLE	—	$0.9\ A$	$0.9\ A$
PIPE	—	$0.6\ A$	$0.6\ A$
GENERAL	—	$0.9\ A$	$0.9\ A$

Table VII-4

Shear Area in BS 5950

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.

$$\frac{F_t}{A_g \rho_y} + \frac{M_{33}}{M_{c33}} + \frac{M_{22}}{M_{c22}} \qquad \text{(BS 4.8.2)}$$

Under Axial Compression

Similarly, the same simplified approach is used for axial compression.

$$\frac{F_c}{A_g \rho_y} + \frac{M_{33}}{M_{c33}} + \frac{M_{22}}{M_{c22}} \qquad \text{(BS 4.8.3.2)}$$

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:

$$\frac{F_c}{A_g \rho_c} + \frac{m_{33} M_{33}}{M_b} + \frac{m_{22} M_{22}}{\rho_y Z_{22}} \qquad \text{(BS 4.8.3.3.1)}$$

The equivalent uniform moment factor, m, for members of uniform section and with flanges, not loaded between adjacent lateral restraints, is defined as

$$m = 0.57 + 0.33\beta + 0.10\beta^2 \geq 0.43 . \qquad \text{(BS Table 18)}$$

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. β 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:

$$\frac{F_{v2}}{P_{v2}}, \text{ and}$$

$$\frac{F_{v3}}{P_{v3}}.$$

Chapter VIII

Check/Design for EUROCODE 3

This chapter describes the details of the structural steel design and stress check algorithms that are used by SAP2000 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 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 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	=	Gross cross-sectional area, mm^2
A_{v2}, A_{v3}	=	Areas for shear in the 2- and 3-directions, mm^2
C_1	=	Bending coefficient
E	=	Modulus of elasticity, MPa
G	=	Shear modulus, MPa
I_t	=	Torsion constant, mm^4
I_w	=	Warping constant, mm^6
I_{33}	=	Major moment of inertia, mm^4
I_{22}	=	Minor moment of inertia, mm^4
K	=	Effective length factor
L	=	Length, span, mm
K_{33}, K_{22}	=	Major and minor effective length factors
$M_{b.Rd}$	=	Design buckling resistance moment, N-mm
M_{cr}	=	Elastic critical moment for lateral-torsional buckling, N-mm
$M_{g.Sd}$	=	Design moments not causing sidesway , N-mm
$M_{s.Sd}$	=	Design moments causing sidesway, N-mm
$M_{V.Sd}$	=	Design moment resistance after considering shear, N-mm
$M_{33.Sd}$	=	Design value of moment about the major axis, N-mm
$M_{22.Sd}$	=	Design value of moment about the minor axis, N-mm
$M_{33.Rd}$	=	Design moment resistance about the major axis, N-mm
$M_{22.Rd}$	=	Design moment resistance about the minor axis, N-mm
$N_{b.Rd}$	=	Design buckling resistance of a compression member, N
$N_{b33.Rd}$	=	Design buckling resistance of a compression member about the major axis, N
$N_{b22.Rd}$	=	Design buckling resistance of a compression member about the minor axis, N
$N_{c.Sd}$	=	Design value of compressive force, N
$N_{c.Rd}$	=	Design compression resistance, N
$N_{t.Sd}$	=	Design value of tensile force, N
$N_{t.Rd}$	=	Design tension resistance, N
$N_{pl.Rd}$	=	Design plastic shear resistance, N
$V_{2.Sd}$	=	Design value of shear force in the major direction, N
$V_{3.Sd}$	=	Design value of shear force in the minor direction, N
$V_{2.Rd}$	=	Design shear resistance in the major direction, N

Table VIII-1
Eurocode 3 Notations

$V_{3.Rd}$	=	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_z	=	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
α	=	Ratio used in classification of sections
γ_{M0}, γ_{M1}	=	Material partial safety factors
ε	=	$\left[\frac{235}{f_y}\right]^{1/2}$ (f_y in MPa)
ρ	=	Reduction factor
τ_{ba}	=	Post-critical shear strength, MPa
χ_{33}, χ_{22}	=	Reduction factors for buckling about the 3-3 and 2-2 axes
χ_{LT}	=	Reduction factor for lateral-torsional buckling
ψ	=	Ratio of smaller to larger end moment of unbraced segment
ψ_s	=	Amplification factor for sway moments

Table VIII-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):

1.35 DL
1.35 DL + 1.50 LL (EC3 2.3.3)

1.35 DL ± 1.50 WL
1.00 DL ± 1.50 WL
1.35 DL + 1.35 LL ± 1.35 WL (EC3 2.3.3)

1.00 DL ± 1.00 EL
1.00 DL + 1.5*0.3 LL ± 1.0 EL (EC3 2.3.3)

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, SAP2000 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).

SAP2000 conservatively classifies the compression elements according to Table VIII-2 and Table VIII-3. Table VIII-2 is used when the section is under the influence of axial compression force only or combined axial compression force and bending. Table VIII-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 VIII-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 SAP2000 does not check stresses for Class 4 sections.

One of the major factors in determining the limiting width-thickness ratio is ε. This parameter is used to reflect the influence of yield stress on the section classification.

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad \text{(EC3 5.3.2)}$$

In classifying I, Box, Channel, Double-Channel, and T sections, two other factors (α, ψ) are defined as follows:

Section	Element	Ratio Checked	Class 1	Class 2	Class 3
I-SHAPE	web	d/t_w	If $\alpha > 0.5$, $\frac{396\,\varepsilon}{(13\alpha - 1)}$, else if $\alpha \geq 0.5$, $\frac{36\,\varepsilon}{\alpha}$.	If $\alpha > 0.5$, $\frac{456\,\varepsilon}{(13\alpha - 1)}$, else if $\alpha \geq 0.5$, $\frac{41.5\,\varepsilon}{\alpha}$.	If $\psi > -1$, $\frac{42\,\varepsilon}{0.67 + 0.33\psi}$, else if $\psi \geq -1$, $\frac{62\,\varepsilon(1-\psi)}{\sqrt{-\psi}}$
	flange	c/t_f (rolled)	$10\,\varepsilon$	$11\,\varepsilon$	$15\,\varepsilon$
		c/t_f (welded)	$9\,\varepsilon$	$10\,\varepsilon$	$14\,\varepsilon$
BOX	web	d/t_w	Same as I-Shape	Same as I-Shape	Same as I-Shape
	flange	$(b - 3t_f)/t_f$ (rolled)	$42\,\varepsilon$	$42\,\varepsilon$	$42\,\varepsilon$
		b/t_f (welded)	$42\,\varepsilon$	$42\,\varepsilon$	$42\,\varepsilon$
CHANNEL	web	d/t_w	Same as I-Shape	Same as I-Shape	Same as I-Shape
	flange	b/t_f	$10\,\varepsilon$	$11\,\varepsilon$	$15\,\varepsilon$
T-SHAPE	web	d/t_w	$33\,\varepsilon$	$38\,\varepsilon$	$42\,\varepsilon$
	flange	$b/2t_f$ (rolled)	$10\,\varepsilon$	$11\,\varepsilon$	$15\,\varepsilon$
		$b/2t_f$ (welded)	$9\,\varepsilon$	$10\,\varepsilon$	$14\,\varepsilon$
DOUBLE ANGLES	—	h/t $(b+h)/[2\max(t,b)]$	Not applicable	Not applicable	15ε 11.5ε
ANGLE	—	h/t $(b+h)/[2\max(t,b)]$	Not applicable	Not applicable	15ε 11.5ε
PIPE	—	d/t	$50\varepsilon^2$	$70\varepsilon^2$	$90\varepsilon^2$
ROUND BAR	—	None	Assumed Class 1		
RECTANGLE	—	None	Assumed Class 2		

Table VIII-2

Limiting Width-Thickness Ratios for
Classification of Sections based on Eurocode 3 (Compression and Bending)

Section	Element	Ratio Checked	Class 1	Class 2	Class 3
I-SHAPE	web	d/t_w	72 ε	83 ε	124 ε
	flange	c/t_f (rolled)	10 ε	11 ε	15 ε
		c/t_f (welded)	9 ε	10 ε	14 ε
BOX	web	d/t_w	72 ε	83 ε	124 ε
	flange	$(b-3t_f)/t_f$ (rolled)	33 ε	38 ε	42 ε
		b/t_f (welded)	33 ε	38 ε	42 ε
CHANNEL	web	d/t_w (Major axis)	72 ε	83 ε	124 ε
		d/t_w (Minor axis)	33 ε	38 ε	42 ε
	flange	b/t_f	10 ε	11 ε	15 ε
T-SHAPE	web	d/t_w	33 ε	38 ε	42 ε
	flange	$b/2t_f$ (rolled)	10 ε	11 ε	15 ε
		$b/2t_f$ (welded)	9 ε	10 ε	14 ε
DOUBLE ANGLES	—	h/t $(b+h)/[2\max(t,b)]$	Not applicable	Not applicable	15.0 ε 11.5 ε
ANGLE	—	h/t $(b+h)/[2\max(t,b)]$	Not applicable	Not applicable	15.0ε 11.5ε
PIPE	—	d/t	$50\varepsilon^2$	$70\varepsilon^2$	$90\varepsilon^2$
ROUND BAR	—	None	Assumed Class 1		
RECTANGLE	—	None	Assumed Class 2		
GENERAL	—	None	Assumed Class 3		

Table VIII-3
Limiting Width-Thickness Ratios for
Classification of Sections based on Eurocode 3 (Bending Only)

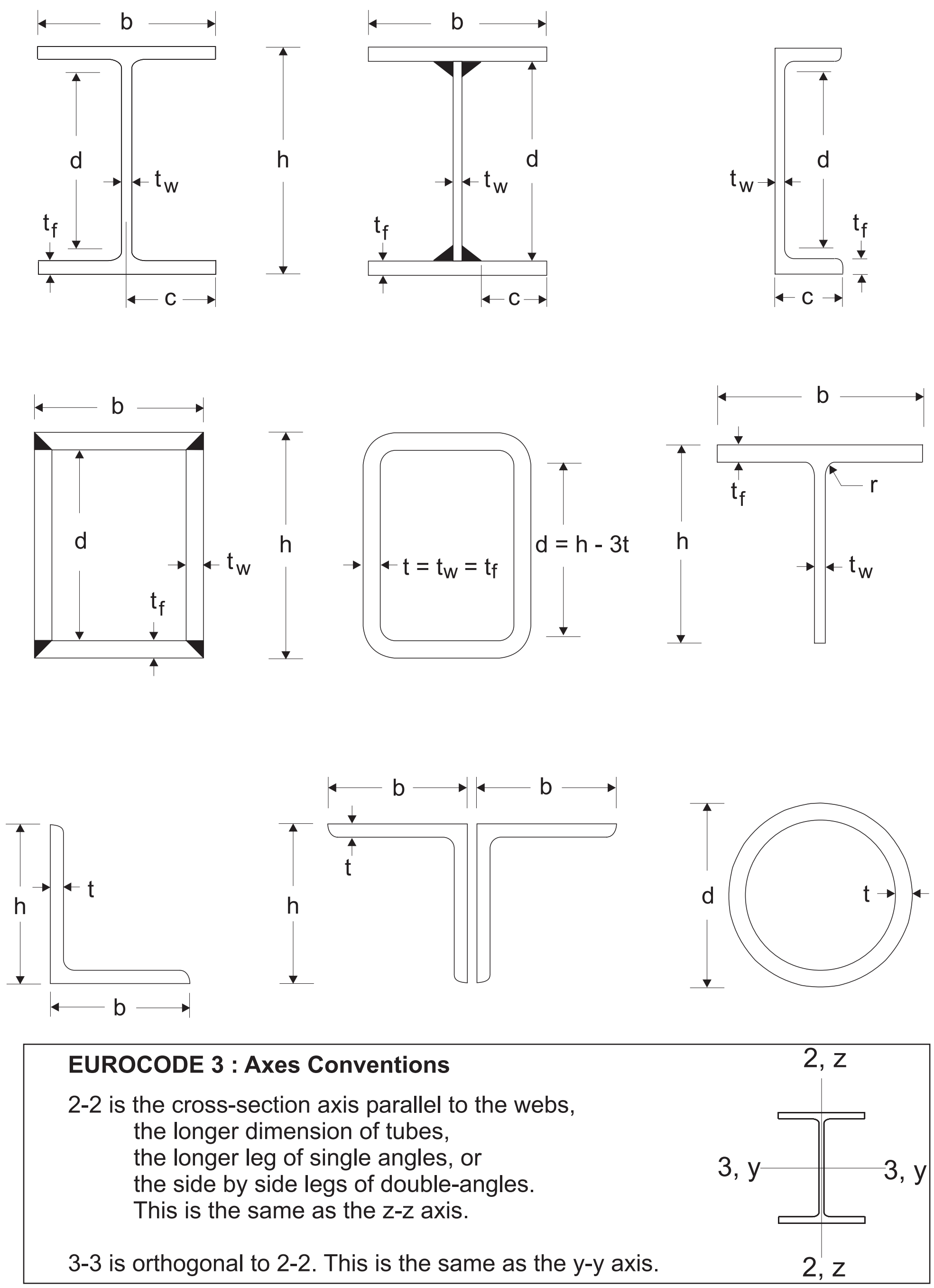


Figure VIII-1
Eurocode 3 Definition of Geometric Properties

$$\alpha = \begin{cases} \frac{1}{2} - \frac{1}{2}\frac{N_{c,Sd}}{ht_w f_f}, & \text{for I, Channel, and T sections,} \\ \frac{1}{2} - \frac{1}{2}\frac{N_{c,Sd}}{2ht_w f_f}, & \text{for Box and Double-Channel sections, and} \end{cases}$$

$$\psi = -\left(1 + 2\frac{N_{c,Sd}}{Af_y}\right),$$

$$0 < \alpha \leq 1.0,$$

$$-3.0 < \psi \leq 1.0.$$

In the above expression, $N_{c,Sd}$ is taken as positive for tension and negative for compression. α equals 0.0 for full tension, 0.5 for pure bending and 1.0 for full compression. ψ 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 $N_{t.Sd}$ 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”.

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.

SAP2000 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. ψ_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 ψ_s for both major and minor direction bending in which case M_{Sd} in a particular direction is taken as:

$$M_{Sd} = M_{g.Sd} + \psi_s M_{s.Sd}, \text{ where} \qquad \text{(EC3 5.2.6.2)}$$

$M_{g.Sd}$ = Design moments not causing translation, and
$M_{s.Sd}$ = 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 nominal 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:

$$\gamma_{M0} = 1.1, \text{ and} \qquad \text{(EC3 5.1.1)}$$

$$\gamma_{M1} = 1.1. \qquad \text{(EC3 5.1.1)}$$

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 nominal capabilities for one or more elements in the "Redefine Element Design Data", these values are **will override all the above mentioned calculated values for those elements** as defined in the following subsections.*

Tension Capacity

The design tension resistance for all classes of sections is evaluated in SAP2000 as follows:

$$N_{t.Rd} = A f_y / \gamma_{M0} \qquad \text{(EC3 5.4.3)}$$

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 ($N_{b.Rd}$).

$$N_{c.Rd} = \min(N_{pl.Rd}, N_{b.Rd}) \qquad \text{(EC3 5.4.4)}$$

The plastic resistance of Class 1, Class 2, and Class 3 sections is given by

$$N_{pl.Rd} = A f_y / \gamma_{M0} \, . \qquad \text{(EC3 5.4.4)}$$

The design buckling resistance of a compression member is taken as

$$N_{b.Rd} = \chi_{min} \, \beta_A \, A f_y / \gamma_{M1} \, , \quad \text{where} \qquad \text{(EC3 5.5.1)}$$

$\beta_A = 1$, for Class 1, 2 or 3 cross-sections.

χ 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.

$$\chi = \frac{1}{\varphi + [\varphi^2 - \bar{\lambda}^2]^{1/2}} \leq 1 \, , \quad \text{in which} \qquad \text{(EC3 5.5.1.2)}$$

$$\varphi = 0.5 \, [1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] \, ,$$

Section	Limits	α (major axis)	α (minor axis)
I-SHAPE (rolled) $h/b > 1.2$	$t_f \leq 40$ mm	0.21	0.34
	$t_f > 40$ mm	0.34	0.49
I-SHAPE (rolled) $h/b \leq 1.2$	$t_f \leq 100$ mm	0.34	0.49
	$t_f > 100$ mm	0.76	0.76
I-SHAPE (welded)	$t_f \leq 40$ mm	0.34	0.49
	$t_f > 40$ mm	0.49	0.76
BOX	Rolled	0.21	0.21
	welded	0.34	0.34
CHANNEL	any	0.49	0.49
T-SHAPE	any	0.49	0.49
DOUBLE ANGLES	any	0.49	0.49
ANGLE	any	0.49	0.49
PIPE	any	0.21	0.21
ROUND BAR	any	0.49	0.49
RECTANGLE	any	0.49	0.49
GENERAL	any	0.49	0.49

Table VIII-4
The α factor for different sections and different axes of buckling

$$\bar{\lambda} = \left\{ \frac{\lambda}{\lambda_1} \right\} [\, \beta_A \,]^{0.5} \,,$$

$\lambda = \dfrac{K_{33}\ l_{33}}{i_{33}}$ or $\dfrac{K_{22}\ l_{22}}{i_{22}}$. The two values of λ give χ_3 and χ_2. χ_{min} is the lesser of the two.

$K = \dfrac{l}{L} \leq 1$. K is conservatively taken as 1 in SAP2000 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,

$\lambda_1 = \pi \left[\dfrac{E}{f_y} \right]^{1/2}$, and

α is an imperfection factor and is obtained from Table VIII-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). SAP2000 does not currently considers 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

$$V_{Rd} = V_{pl.Rd} = \frac{A_v f_y}{\sqrt{3}} \Big/ \gamma_{M0} \,, \qquad \text{(EC3 5.4.6)}$$

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

$$V_{Rd} = V_{ba.Rd} = d\, t_w\, \tau_{ba} \,/\, \gamma_{M1}\,, \qquad (\text{for } \frac{d}{t_w} > 69\,\varepsilon) \qquad \text{(EC3 5.6.3)}$$

where, τ_{ba} is the simple post-critical shear strength which is determined as follows:

$$\tau_{ba} = \frac{f_{yw}}{\sqrt{3}}\,, \qquad \text{for} \quad \bar{\lambda}_w \leq 0.8\,, \qquad \text{(EC3 5.6.3)}$$

$$\tau_{ba} = \left[1 - 0.625\,(\,\bar{\lambda}_w - 0.8\,)\right]\frac{f_{yw}}{\sqrt{3}}\,, \qquad \text{for} \quad 0.8 < \bar{\lambda}_w < 1.2\,, \text{ and} \qquad \text{(EC3 5.6.3)}$$

$$\tau_{ba} = \left[0.9 \,/\, \bar{\lambda}_w\right]\frac{f_{yw}}{\sqrt{3}}\,, \qquad \text{for} \quad \bar{\lambda}_w \geq 1.2\,. \qquad \text{(EC3 5.6.3)}$$

in which $\bar{\lambda}_w$ is the web slenderness ratio,

$$\bar{\lambda}_w = \frac{d \,/\, t_w}{37.4\,\varepsilon\sqrt{k_\tau}}\,, \text{ and} \qquad \text{(EC3 5.6.3)}$$

k_τ is the buckling factor for shear. For webs with transverse stiffeners at the supports but no intermediate transverse stiffeners,

$$k_\tau = 5.34\,. \qquad \text{(EC3 5.6.3)}$$

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 $V_{Sd} \leq 0.5 V_{pl.Rd}$

- For Class 1 and Class 2 Sections

$$M_{c.Rd} = M_{pl.Rd} = W_{pl}\, f_y \,/\, \gamma_{M0}\,. \qquad \text{(EC3 5.4.5.2)}$$

- For Class 3 Sections

$$M_{c.Rd} = M_{el.Rd} = W_{el}\, f_y \,/\, \gamma_{M0}\ . \qquad \text{(EC3 5.4.5.2)}$$

If $V_{Sd} > 0.5 V_{pl.Rd}$

- For I, Box, and Channel sections bending about the 3-3 axis the moment capacities considering the effects of shear force are computed as

$$M_{V.Rd} = \left[W_{pl} - \frac{\rho A_v^2}{4 t_w} \right] \frac{f_y}{\gamma_{M0}} \leq M_{c.Rd}\ , \text{ where} \qquad \text{(EC3 5.4.7)}$$

$$\rho = \left[\frac{2\, V_{Sd}}{V_{pl.Rd}} - 1 \right]^2 .$$

- 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,

$$M_{b.Rd} = \chi_{LT}\, \beta_w\, W_{pl.33}\, f_y \,/\, \gamma_{M1}\ , \text{ where} \qquad \text{(EC3 5.5.2)}$$

$\beta_w = 1$, for Class 1 and Class 2 sections,

$\beta_w = \dfrac{W_{el.33}}{W_{pl.33}}$, for Class 3 sections,

$$\chi_{LT} = \frac{1}{\varphi_{LT} + [\varphi_{LT}^2 - \bar{\lambda}_{LT}^2]^{1/2}} \leq 1, \text{ in which}$$

$$\varphi_{LT} = 0.5 \left[1 + \alpha_{LT} (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2 \right], \text{ where}$$

$\alpha_{LT} = 0.21$, for rolled sections,

$\alpha_{LT} = 0.49$, for welded sections, and

$$\bar{\lambda}_{LT} = \left[\frac{\beta_w \, W_{pl.33} \, f_y}{M_{cr}} \right]^{0.5}, \text{ where}$$

$$M_{cr} = C_1 \frac{\pi^2 \, E I_{22}}{L^2} \left[\frac{I_w}{I_{22}} + \frac{L^2 \, G I_t}{\pi^2 \, E I_{22}} \right]^{0.5}, \qquad \text{(EC3 F1.1)}$$

I_t = The torsion constant,

I_w = The warping constant,

L = Laterally unbraced length for buckling about the minor axis. It is taken as l_{22},

$C_1 = 1.88 - 1.40\,\psi + 0.52\,\psi^2 \le 2.7$, and

ψ = The ratio of smaller to larger end moment of unbraced segment, $\frac{M_a}{M_b}$. ψ varies between -1 and 1 ($-1 \le \psi \le 1$). 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 $\bar{\lambda}_{LT} \le 0.4$, 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,

$$M_{b.Rd} = W_{el,33} \, f_y \,/\, \gamma_{M1} \,,$$

and the lateral-torsional buckling resistance of Rectangle, Circle and Pipe sections is evaluated as,

$$M_{b.Rd} = W_{pl,33} \, f_y \,/\, \gamma_{M1} \,.$$

Currently SAP2000 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.

$$V_{Sd} \leq 0.5 V_{pl.Rd}, \text{ and} \qquad \text{(EC3 5.4.9)}$$

$$V_{Sd} \leq 0.5 V_{ba.Rd}, \qquad \text{(EC3 5.4.9)}$$

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

$$\frac{N_{c.Sd}}{N_{pl.Rd}} + \frac{M_{33.Sd}}{M_{pl.33.Rd}} + \frac{M_{22.Sd}}{M_{pl.22.Rd}}. \qquad \text{(EC3 5.4.8.1)}$$

For Class 3 sections, the capacity ratio is conservatively taken as

$$\frac{N_{c.Sd}}{Af_{yd}} + \frac{M_{33.Sd}}{W_{el.33} f_{yd}} + \frac{M_{22.Sd}}{W_{el.22} f_{yd}}, \text{ where} \qquad \text{(EC3 5.4.8.1)}$$

$$f_{yd} = \frac{f_y}{\gamma_{M0}}.$$

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, $V_{ba.Rd}$, the shear is considered to be high, i.e. the shear is high if

$$V_{Sd} > 0.5\, V_{pl.Rd}\ , \quad \text{or} \qquad \text{(EC3 5.4.9)}$$

$$V_{Sd} > 0.5\, V_{ba.Rd}\ . \qquad \text{(EC3 5.4.9)}$$

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

$$\frac{N_{c.Sd}}{N_{pl.Rd}} + \frac{M_{33.Sd}}{M_{V.33.Rd}} + \frac{M_{22.Sd}}{M_{V.22.Rd}}, \text{ where} \qquad \text{(EC3 5.4.8.1)}$$

$M_{V.33.Rd}$ and $M_{V.22.Rd}$ are the design moment resistances about the major and the minor axes, respectively, considering the effect of high shear (see page 142).

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

$$\frac{N_{c.Sd}}{N_{b.min.Rd}} + \frac{k_{33}\, M_{33.Sd}}{\eta M_{c.33.Rd}} + \frac{k_{22}\, M_{22.Sd}}{\eta M_{c.22.Rd}}, \text{ where} \qquad \text{(EC3 5.5.4)}$$

$$N_{b.min.Rd} = \min\left\{N_{b.33.Rd},\ \ N_{b.22.Rd}\right\},$$

$$\eta = \frac{\gamma_{M0}}{\gamma_{M1}},$$

$$k_{33} = 1 - \frac{\mu_{33}\, N_{c.Sd}}{\chi_{33}\, A f_y} \le 1.5,$$

$$k_{22} = 1 - \frac{\mu_{22}\, N_{c.Sd}}{\chi_{22}\, A f_y} \le 1.5,$$

$$\mu_{33} = \bar{\lambda}_{33} (2\beta_{M.33} - 4) + \left[\frac{W_{pl.33} - W_{el.33}}{W_{el.33}} \right] \leq 0.9, \text{ (Class 1 and Class 2),}$$

$$\mu_{22} = \bar{\lambda}_{22} (2\beta_{M.22} - 4) + \left[\frac{W_{pl.22} - W_{el.22}}{W_{el.22}} \right] \leq 0.9, \text{ (Class 1 and Class 2),}$$

$$\mu_{33} = \bar{\lambda}_{33} (2\beta_{M.33} - 4) \leq 0.9, \quad \text{(for Class 3 sections),}$$

$$\mu_{22} = \bar{\lambda}_{22} (2\beta_{M.22} - 4) \leq 0.9, \quad \text{(for Class 3 sections),}$$

$\beta_{M.33}$ = 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

$$\beta_M = (1.8 - 0.7\psi) + \frac{M_Q}{\Delta M} (0.7\psi - 0.5), \text{ and}$$

M_Q = Absolute maximum moment due to lateral load only assuming simple support at the ends,

ψ = Absolute value of the ratio of smaller to larger end moment. ψ varies between -1 and 1 ($-1 \leq \psi \leq 1$). A negative value implies double curvature.

ΔM = Absolute maximum value of moment for moment diagram without change of sign, and

ΔM = 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

$$\frac{N_{c.Sd}}{N_{b.22.Rd}} + \frac{k_{LT}\, M_{33.Sd}}{M_{b.Rd}} + \frac{k_{22}\, M_{22.Sd}}{\eta M_{c.22.Rd}}, \text{ where} \qquad \text{(EC3 5.5.4)}$$

k_{22} and η are as defined in the previous subsection "Bending, Compression, and Flexural Buckling",

$$k_{LT} = 1 - \frac{\mu_{LT} N_{c.Sd}}{\chi_{22} A f_y} \leq 1, \text{ where}$$

$$\mu_{LT} = 0.15 \bar{\lambda}_{22} \beta_{M.LT} - 0.15 \leq 0.9, \text{ and}$$

$\beta_{M.LT}$ = 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.

$$V_{Sd} \leq 0.5 V_{pl.Rd}, \text{ and} \quad \text{(EC3 5.4.9)}$$

$$V_{Sd} \leq 0.5 V_{ba.Rd}, \quad \text{(EC3 5.4.9)}$$

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

$$\frac{N_{t.Sd}}{N_{t.Rd}} + \frac{M_{33.Sd}}{M_{pl.33.Rd}} + \frac{M_{22.Sd}}{M_{pl.22.Rd}}. \quad \text{(EC3 5.4.8.1)}$$

For Class 3 sections, the capacity ratio is conservatively taken as

$$\frac{N_{t.Sd}}{A f_{yd}} + \frac{M_{33.Sd}}{W_{el.33} f_{yd}} + \frac{M_{22.Sd}}{W_{el.22} f_{yd}}. \quad \text{(EC3 5.4.8.1)}$$

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

$$V_{Sd} > 0.5 V_{pl.Rd}, \quad \text{or} \quad \text{(EC3 5.4.9)}$$

$$V_{Sd} > 0.5 V_{ba.Rd}. \quad \text{(EC3 5.4.9)}$$

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

$$\frac{N_{t.Sd}}{N_{t.Rd}} + \frac{M_{33.Sd}}{M_{V.33.Rd}} + \frac{M_{22.Sd}}{M_{V.22.Rd}} . \qquad \text{(EC3 5.4.8.1)}$$

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:

$$M_{eff.33.Sd} = M_{33.Sd} - \psi_{vec} \frac{N_{t.Sd} W_{com.33}}{A} \text{, where} \qquad \text{(EC3 5.5.3)}$$

$\psi_{vec} = 0.8$ (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, $N_{t.Sd}$, major axis bending, $M_{33.Sd}$, and minor axis bending, $M_{22.Sd}$, the capacity ratio is taken as

$$\frac{N_{t.Sd}}{N_{t.Rd}} + \frac{k_{LT}\, M_{33.Sd}}{M_{b.Rd}} + \frac{k_{22}\, M_{22.Sd}}{\eta\, M_{c.22.Rd}} - \psi_{vec} k_{LT} \frac{N_{t.Sd} W_{com.33}}{A\, M_{b.Rd}} , \qquad \text{(EC3 5.5.4)}$$

where k_{LT}, k_{22} and η 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:

$$\frac{V_{2.Sd}}{V_{2.Rd}} \text{ and } \frac{V_{3.Sd}}{V_{3.Rd}} .$$

Chapter IX

Design Output

Overview

SAP2000 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. 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 details of the calculation from the designer's point of view. 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 information is specific to the chosen steel design codes which are available in the program and is only described where required. 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 Output

The graphical output can be produced either as color screen display or in gray-scaled printed form. 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,
- K-factors for major and minor direction of buckling,
- Unbraced Length Ratios,
- C_m-factors,
- C_b-factors,
- Live Load Reduction Factors,
- δ_s-factors,
- δ_b-factors,
- design type,
- allowable stresses in axial, bending, and 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 **Display Design Info...** from the **Design** menu. 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 SAP2000 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...** from the **View** menu.

The graphical display in an active window can be printed in gray scaled black and white from the SAP2000 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 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:

- Load Combination Multipliers
  - Combination name,
  - Load types, and
  - Load factors.
- Steel Stress Check Element Information (code dependent)
  - Frame ID,
  - Design Section ID,
  - K-factors for major and minor direction of buckling,
  - Unbraced Length Ratios,
  - C_m-factors,
  - C_b-factors, and
  - Live Load Reduction Factors.

- Steel Moment Magnification Factors (code dependent)
  - Frame ID,
  - Section ID,
  - Framing Type,
  - δ_b-factors, and
  - δ_s-factors.

The output design information includes the following:

- Steel Stress Check Output (code dependent)
  - Frame ID,
  - Section location,
  - Controlling load combination ID for P-M interaction,
  - Tension or compression indication,
  - Axial and bending interaction ratio,
  - Controlling load combination ID for major and minor shear forces, and
  - Shear stress ratios.

The tabular output can be accessed by selecting **Print Design Tables...** from the **File** menu. This will pop up a dialog box. Then the user can specify the design quantities for which the results are to be tabulated. 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 chose a file from. Then clicking the **OK** button will direct the tabular output to the requested stream — the file or the printer.

Member Specific Information

The member specific design information shows the details of the calculation from the designer's point of view. It provides an access to the geometry and material data, other input data, design section dimensions, design and allowable stresses, reinforcement details, 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.

- Frame ID,
- 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 **ReDesign** and **Details** buttons in the dialog box. Additional information that is available by clicking on the **ReDesign** button is as follows:

- Design Factors (code dependent)
  - Effective length factors, K, for major and minor direction of buckling,
  - Unbraced Length Ratios,
  - C_m-factors,
  - C_b-factors,
  - Live Load Reduction Factors,
  - δ_s-factors, and
  - δ_b-factors.
- Element Section ID
- Element Framing Type
- Overwriting allowable stresses

Additional information that is available by clicking on the **Details** button is given below.

- Frame, Section, Station, and Load Combination IDs,
- Section geometric information and graphical representation,
- Material properties of steel,
- Moment factors,
- Design and allowable stresses for axial force and biaxial moments, and
- Design and allowable stresses for shear.

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Index