COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS® 2016

Integrated Building Design Software

Steel Frame Design Manual

AIS@ ASD-1989, AISC LRFD-1993, & BS 5950-2000

Steel Frame Design Manual

AISC ASD-1989, AISC LRFD-1993 and BS 5950-2000

For ETABS® 2016

ISO ETA122815M7 Rev 0
Proudly developed in the United States of America December 2015

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Chapter I

Introduction

Overview

ETABS feature 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 steel design codes supported in program can be found in Desing > Steel Frame Design > Preferences menu.

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 the program. 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.

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

Special 1989AISC-ASD and 1993 AISC--LRFD seismic design provisions are implemented in the current version of the program. The ratio of the beam flexural capacities with respect to the column reduced flexural capacities (reduced for axial force effect) associated with the weak beam-strong column aspect of any beam/column intersection, are reported for special moment resisting frames. Capacity requirements associated with seismic framing systems that require ductility are checked.

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

Each of eleven subsequent chapters gives a detailed description of a specific code of practice as interpreted by and implemented in the program. 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 steel code (AISC 1989) as implemented in the program.
- Chapter IV gives a detailed description of the AISC LRFD steel code (AISC 1993) as implemented in the program.
- Chapter V gives a detailed description of the British code BS 5950 (BSI 2000) as implemented in the program.

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

Recommended Reading

It is recommended that the user read Chapter II "Design Algorithms" and one of eleven subsequent chapters corresponding to the code of interest to the user. Finally the user should read "Design Output" in Chapter VI for understanding and interpreting the program output related to steel design.

Chapter **II**

Design Algorithms

This chapter outlines various aspects of the steel check and design procedures that are used by the 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)**.**
- British Standards Institution's "Structural Use of Steelwork in Building", **BS 5950** (BSI 2000).

Details of the algorithms associated with each of these codes as implemented and interpreted in the program 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 AISC-LRFD code carry the prefix of "**LRFD**". Similarly,

– References to the AISC-ASD code carry the prefix of "**ASD**"

– References to the British code carry the prefix of "**BS**"

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- Effects

The program design algorithms require that the analysis results include the P- effects. The P- 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- is limited to “individual member stability”. For unbraced components, “lateral drift effects” should be considered in addition to individual member stability effect. In the program, 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 code is 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, the program assumes that the amplification is already included in the results because P- effects are considered for all but AISC-ASD code.

The users of the program should be aware that the default analysis option in the program is turned OFF for P- effect. The default number of iterations for P- analysis is 1. **The user should turn the P- analysis ON and set the maximum number of iterations for the analysis.** No P- analysis is required for the AISC-ASD code. For further reference, the user is referred to CSI Analysis Reference Manual (CSI 2005).

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 program 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.

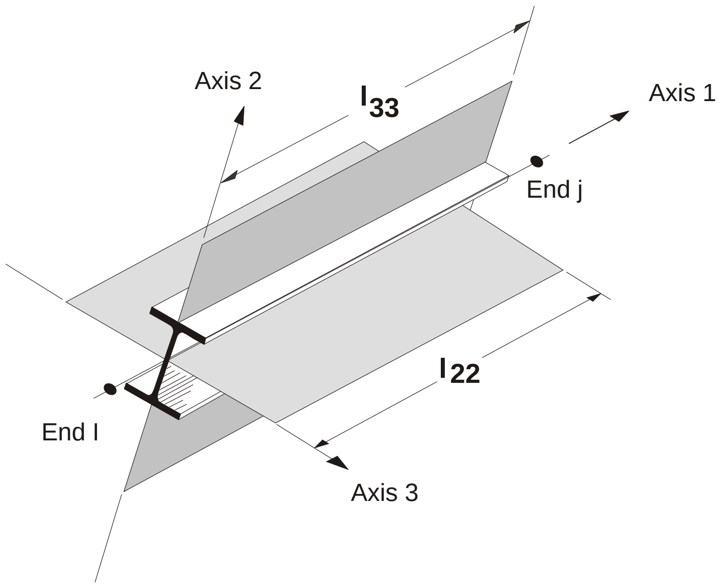


Figure II-1
Major and Minor Axes of Bending

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.

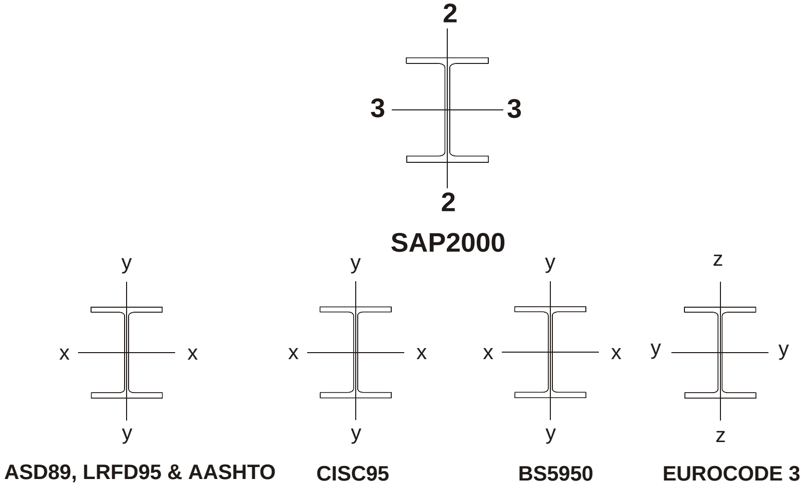


Figure II-2
Correspondence between the program Axes and Code Axes

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.

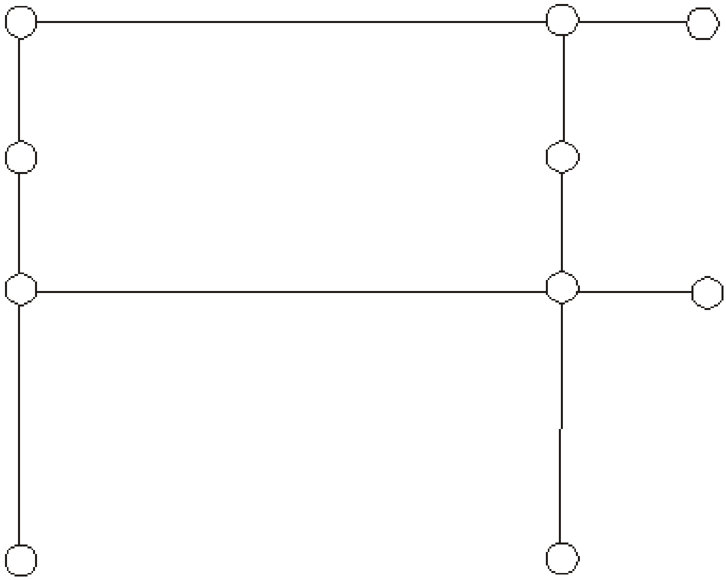


Figure II-3

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:

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

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

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   :

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

from which *K* =   / . 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 the program.

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 the program when the user selects the AISC-ASD89 design code (AISC 1989). Various notations used in this chapter are described in Table V-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}	=	$\dfrac{12\ {}^2 E}{23\left(K_{33}l_{33}/r_{33}\right)^2}$
F_{e22}	=	$\dfrac{12\ {}^2 E}{23\left(K_{22}l_{22}/r_{22}\right)^2}$
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_aQ_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 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
$_w$	=	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 the program 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. The program 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.

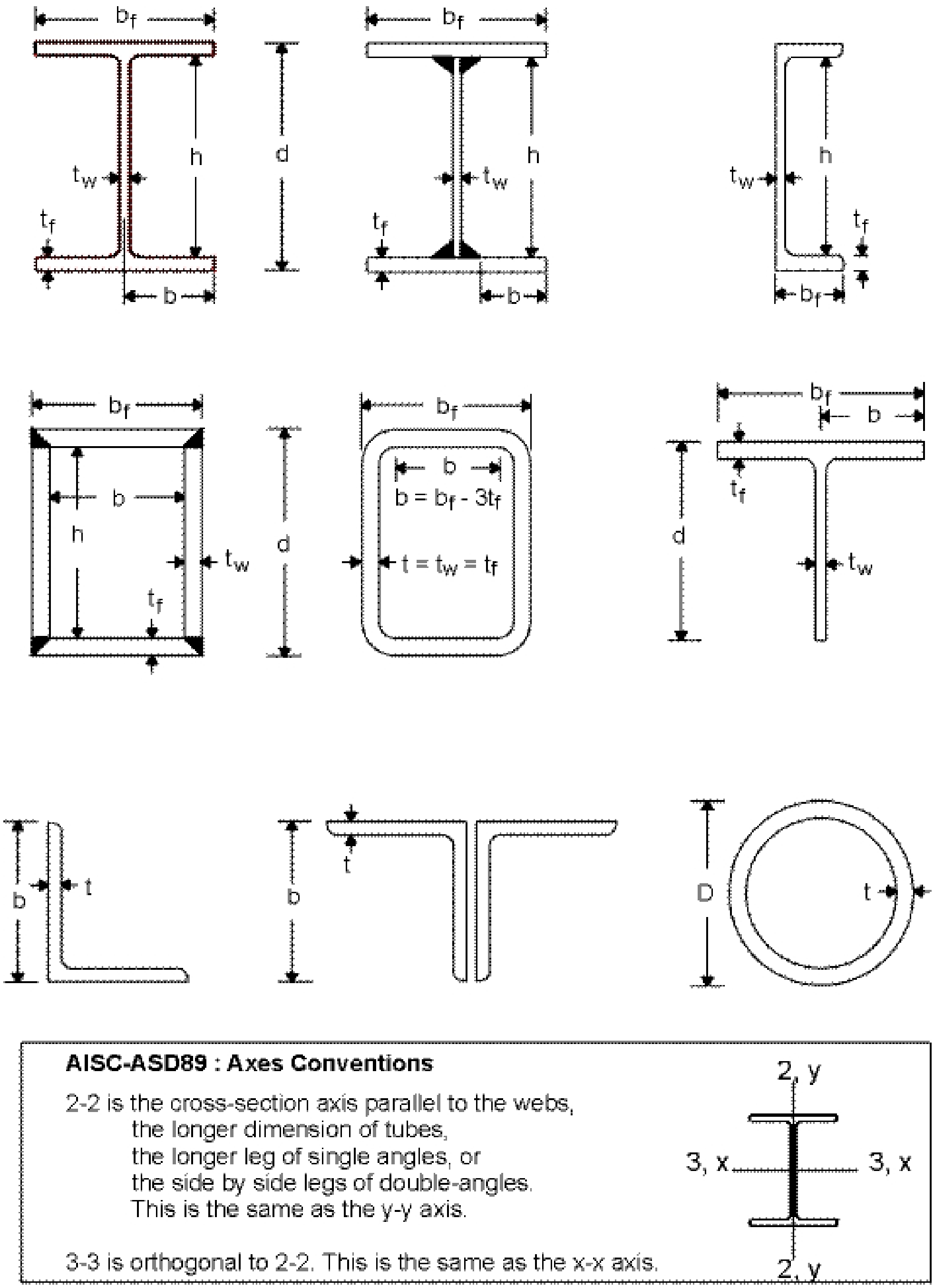


Figure III-1

AISC-ASD Definition of Geometric Properties

Section Description	Ratio Checked	Compact Section	Noncompact Section	Slender Section
I-SHAPE	$b_f / 2t_f$ (rolled)	$65 / \sqrt{F_y}$	$95 / \sqrt{F_y}$	No limit
	$b_f / 2t_f$ (welded)	$65 / \sqrt{F_y}$	$95 / \sqrt{F_y / k_c}$	No limit
	d / t_w	For f_a / F_y 0.16 $\frac{640}{\sqrt{F_y}}(1 \quad 3.74 \frac{f_a}{F_y})$, For $f_a / F_y > 0.16$ $257 / \sqrt{F_y}$.	No limit	No limit
	h / t_w	No limit	If compression only, $253 / \sqrt{F_y}$ otherwise $760 / \sqrt{F_b}$	$\frac{14000}{\sqrt{F_y(F_y + 16.5)}}$ 260
BOX	b / t_f	$190 / \sqrt{F_y}$	$238 / \sqrt{F_y}$	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	$t_w \quad t_f / 2$, $d_w \quad 6b_f$	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 0.25, t_f / t_w 3.0 If rolled b_f / d_w 0.5, t_f / t_w 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	$b_f / 2t_f$	$65 / \sqrt{F_y}$	$95 / \sqrt{F_y}$	No limit
	d / t_w	Not applicable	$127 / \sqrt{F_y}$	No limit
	Other	No limit	No limit	If welded b_f/d_w 0.5, t_f/t_w 1.25 If rolled b_f/d_w 0.5, t_f/t_w 1.10
DOUBLE ANGLES	b / t	Not applicable	$76 / \sqrt{F_y}$	No limit
ANGLE	b / t	Not applicable	$76 / \sqrt{F_y}$	No limit
PIPE	D / t	$3{,}300 / F_y$	$3{,}300 / F_y$	$13{,}000 / F_y$ (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.

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

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 program "Overwrites 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 $0.60\,F_y$.

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

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

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

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

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:

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

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

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:

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

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

where,

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

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:

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

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:

$$A_e = A_g \qquad \left(b \quad b_e\right) t$$

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):

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

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0 & \text{if} \quad b_f/2t_f \quad 95/\sqrt{F_y/k_c}, \\ 1.293 \quad 0.00309[b_f/2t_f]\sqrt{F_y/k_c} & \text{if} \quad 95/\sqrt{F_y/k_c} < b_f/2t_f < 195/\sqrt{F_y/k_c}, \\ 26{,}200\, k_c / \{[b_f/2t_f]^2 F_y\} & \text{if} \quad b_f/2t_f \quad 195/\sqrt{F_y/k_c}. \end{cases}$	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. $Q_s \quad \begin{cases} 1.0\,, & \text{if} \quad d/t_w \quad 127/\sqrt{F_y}, \\ 1.908 \quad 0.00715[d/t_w]\sqrt{F_y}, & \text{if} \quad 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}, \\ 20{,}000/\{[d/t_w]^2 F_y\}, & \text{if} \quad d/t_w \quad 176/\sqrt{F_y}. \end{cases}$	ASD A-B5-3, ASD A-B5-4, ASD A-B5-5, ASD A-B5-6
DOUBLE-ANGLE	$Q_s = \begin{cases} 1.0\,, & \text{if} \quad b/t \quad 76/\sqrt{F_y}, \\ 1.340 \quad 0.00447[b/t]\sqrt{F_y}, & \text{if} \quad 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500/\{[b/t]^2 F_y\}, & \text{if} \quad b/t \quad 155/\sqrt{F_y}. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
ANGLE	$Q_s = \begin{cases} 1.0\,, & \text{if} \quad b/t \quad 76/\sqrt{F_y}, \\ 1.340 \quad 0.00447[b/t]\sqrt{F_y}, & \text{if} \quad 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500/\{[b/t]^2 F_y\}, & \text{if} \quad b/t \quad 155/\sqrt{F_y}. \end{cases}$	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	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{195.74}{\sqrt{f}}, \\ \frac{253\,t_w}{\sqrt{f}} \ 1 \ \frac{44.3}{(h/t_w)\sqrt{f}}\ , & if \ \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	ASD A-B5-8
BOX	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{195.74}{\sqrt{f}}, \\ \frac{253\,t_w}{\sqrt{f}} \ 1 \ \frac{44.3}{(h/t_w)\sqrt{f}}\ , & if \ \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	ASD A-B5-8
	$b_e = \begin{cases} b, & if \ \frac{b}{t_f} \ \frac{183.74}{\sqrt{f}}, \\ \frac{253\,t_f}{\sqrt{f}} \ 1 \ \frac{50.3}{(h/t_f)\sqrt{f}}\ , & if \ \frac{b}{t} > \frac{183.74}{\sqrt{f}}. \end{cases}$ (compr., flexure, $f = 0.6F_y$)	ASD A-B5-7
CHANNEL	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{195.74}{\sqrt{f}}, \\ \frac{253\,t_w}{\sqrt{f}} \ 1 \ \frac{44.3}{(h/t_w)\sqrt{f}}\ , & if \ \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	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	

Table III-4

Effective Width for Stiffened Sections

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

where,

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

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

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 the program as follows:

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

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

- For T-sections and Double-angles:

$$F_e = \frac{F_{e22} + F_{ez}}{2H} \div \quad 1 \quad \sqrt{1 \quad \frac{4\, F_{e22} F_{ez} H}{(F_{e22} + F_{ez})^2}} \qquad \text{(LRFD A-E3-6)}$$

- For Channels:

$$F_e = \frac{F_{e33} + F_{ez}}{2H} \div \quad 1 \quad \sqrt{1 \quad \frac{4\, F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \qquad \text{(LRFD A-E3-6)}$$

- For Single-angle sections with equal legs:

$$F_e = \frac{F_{e33} + F_{ez}}{2H} \div \quad 1 \quad \sqrt{1 \quad \frac{4\, F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \qquad \text{(ASD SAM C-C4-1)}$$

- 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):

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

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,

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

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

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

$$F_{ez} = \quad \frac{{}^{2}EC_w}{\left(K_z l_z\right)^2} + GJ \quad \frac{1}{Ar_0^2}, \qquad \text{(LRFD A-E3-12)}$$

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 the program,

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

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

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:

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

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

For Noncompact sections:

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

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

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

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 $\frac{l_{22}}{r_T} \quad \sqrt{\frac{102{,}000\, C_b}{F_y}}$,

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

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

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

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

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

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

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

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,

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

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). The program 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:

$$F_{b33} = R_{PG} R_e F_{b33} \text{, where} \qquad \text{(ASD G2-1)}$$

$$R_{PG} = 1.0 \quad 0.0005 \frac{A_w}{A_f} \quad \frac{h}{t} \quad \frac{760}{\sqrt{F_{b33}}} \quad 1.0 \text{,} \qquad \text{(ASD G2)}$$

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

$$R_e = 1.0 \text{,} \qquad \text{(non-hybrid girders)} \qquad \text{(ASD G2)}$$

A_w = Area of web, in^2,

A_f = Area of compression flange, in^2,

$$= \frac{0.6 F_y}{F_{b33}} \quad 1.0 \qquad \text{(ASD G2)}$$

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 the program 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.

$$F_{b33} \quad Q_s \left(0.6 F_y\right) \text{, where} \qquad \text{(ASD A-B5.2a, A-B5.2d)}$$

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:

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

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

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

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:

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

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:

$$F_{b33} \quad Q_s\left(0.6 F_y\right) \qquad \text{(ASD A-B5.2a, A-B5.2d)}$$

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,

$$F_b = 0.60\, Q_s\, F_y\ .$$

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

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

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 the program.

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:

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

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

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:

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

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:

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

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):

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

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

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

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

where,

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

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

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,

$$_w = \frac{1}{I_{max}} \; {}_A z(w^2 + z^2)dA \qquad 2z_0 \, , \qquad \text{(ASD SAM 5.3.2)}$$

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.

$_w$ 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 the program, 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.

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

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

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

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

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

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):

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

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

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

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 the program 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:

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

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

where,

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

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

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 the program,

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

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

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

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.

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

For sway frame $C_m = 0.85$, for nonsway frame without transverse load $C_m = 0.6 \quad 0.4 M_a / M_b$, 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

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

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

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

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

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

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

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

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 the program. 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 the program when the user selects the AISC-LRFD93 design code (AISC 1994). Various notations used in this chapter are described in Table VI-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, $b_f \quad 2t_w$ for welded and $b_f \quad 3t_w$ 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 $d \quad 2k$ for rolled sections, and $d \quad 2t_f$ for welded sections
k	=	Distance from outer face of flange to web toe of fillet, in
k_c	=	Parameter used for section classification, $4 / \sqrt{h / t_w}$, $0.35 \quad k_c \quad 0.763$
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
${}_{w}$	=	Special section property for angles, in
	=	Slenderness parameter
${}_{c}, {}_{e}$	=	Column slenderness parameters
${}_{p}$	=	Limiting slenderness parameter for compact element
${}_{r}$	=	Limiting slenderness parameter for non-compact element
${}_{s}$	=	Limiting slenderness parameter for seismic element
${}_{slender}$	=	Limiting slenderness parameter for slender element
${}_{b}$	=	Resistance factor for bending, 0.9
${}_{c}$	=	Resistance factor for compression, 0.85
${}_{t}$	=	Resistance factor for tension, 0.9
${}_{v}$	=	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)

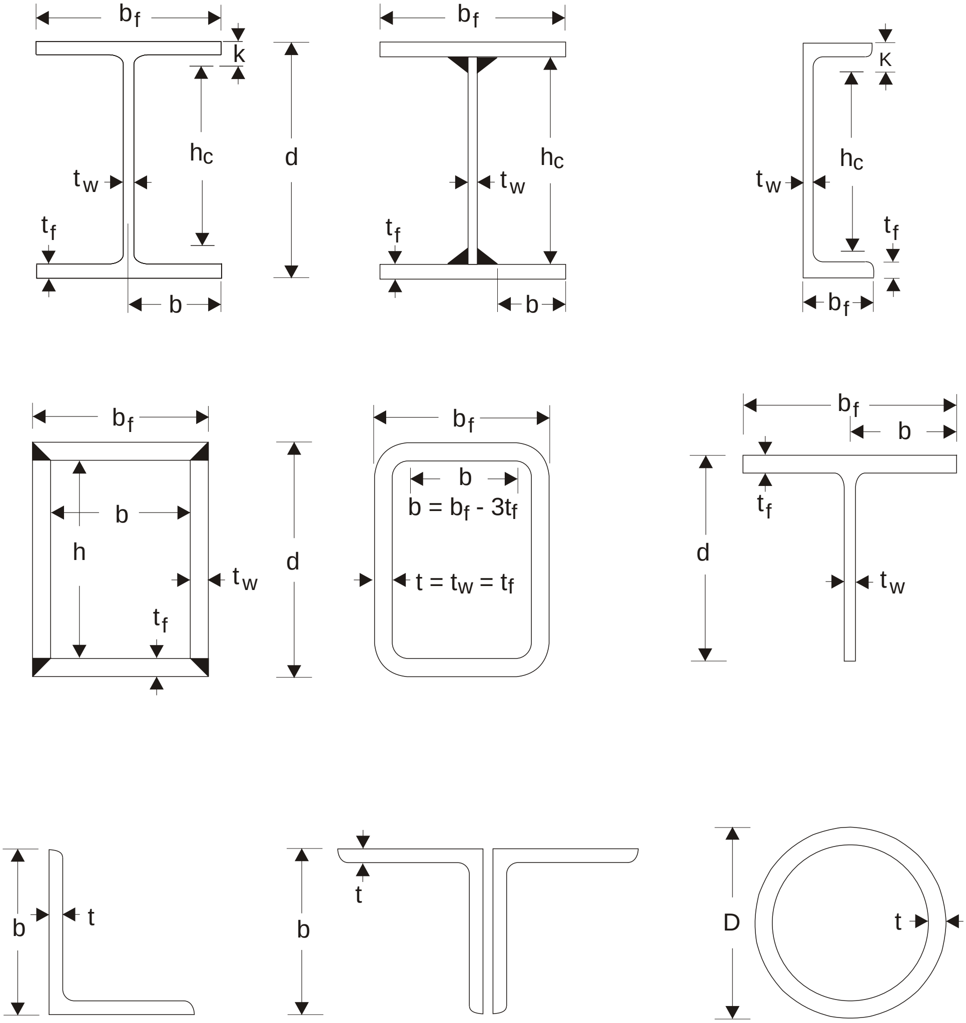
These are also the default design load combinations in the program whenever the AISC-LRFD93 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

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, the program design assumes that a P- analysis has been performed so that moment magnification factors for moments causing sidesway can be taken as unity. It is recommended that the P- 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. The



AISC-LRFD93 : Axes Conventions

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

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

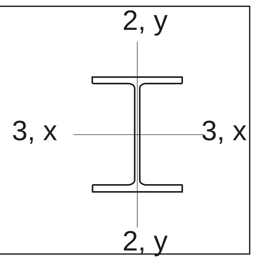


Figure IV-1

AISC-LRFD Definition of Geometric Properties

Description of Section	Check ( )	COMPACT ($_p$)	NONCOMPACT ($_r$)	SLENDER ($_{slender}$)
I-SHAPE	$b_f / 2t_f$ (rolled)	$65 / \sqrt{F_y}$	$141 / \sqrt{F_y - 10.0}$	No limit
	$b_f / 2t_f$ (welded)	$65 / \sqrt{F_y}$	$162 / \sqrt{\frac{F_y - 16.5}{k_c}}$	No limit
	h_c / t_w	For $P_u /\ {}_bP_y$ 0.125, $\frac{640}{\sqrt{F_y}}\ 1 - \frac{2.75P_u}{{}_bP_y}$ For $P_u /\ {}_bP_y > 0.125$ $\frac{191}{\sqrt{F_y}}\ 2.33 - \frac{P_u}{{}_bP_y}$ $\frac{253}{\sqrt{F_y}}$	$\frac{970}{\sqrt{F_y}}\ 1 - 0.74\ \frac{P_u}{{}_bP_y}$	$\frac{14000}{\sqrt{F_y(F_y + 16.5)}}$ 260
BOX	b / t_f h_c / t_w	$190 / \sqrt{F_y}$ As for I-shapes	$238 / \sqrt{F_y}$ As for I-shapes	No limit $970 / \sqrt{F_y}$
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 $127 / \sqrt{F_y}$	No limit No limit
ANGLE	b / t	Not applicable	$76 / \sqrt{F_y}$	No limit
DOUBLE-ANGLE (Separated)	b / t	Not applicable	$76 / \sqrt{F_y}$	No limit
PIPE	D / t	$2070 / F_y$	$8970 / F_y$	$13000 / F_y$ (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 ( )	COMPACT (SEISMIC ZONE) ($_s$)	NONCOMPACT (Uniform Compression) (M_{22} M_{33} 0) ($_r$)
I-SHAPE	$b_f / 2t_f$ (rolled)	$52 / \sqrt{F_y}$	$95 / \sqrt{F_y}$
	$b_f / 2t_f$ (welded)	$52 / \sqrt{F_y}$	$95 / \sqrt{F_y}$
	h_c / t_w	For $P_u / \ _bP_y$ 0.125, $\frac{520}{\sqrt{F_y}}$ 1 - 1.54 $\frac{P_u}{_bP_y}$ ÷ For $P_u / \ _bP_y > 0.125$ $\frac{191}{\sqrt{F_y}}$ 2.33 - $\frac{P_u}{_bP_y}$ ÷ $\frac{253}{\sqrt{F_y}}$	$253 / \sqrt{F_y}$
BOX	b / t_f h_c / t_w	Not applicable Not applicable	$238 / \sqrt{F_y}$ $253 / \sqrt{F_y}$
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 $127 / \sqrt{F_y}$
ANGLE	b / t	Not applicable	$76 / \sqrt{F_y}$
DOUBLE-ANGLE (Separated)	b / t	Not applicable	$76 / \sqrt{F_y}$
PIPE	D / t	Not applicable	$3300 / F_y$
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

program 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

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

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

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

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

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.

The program design assumes the analysis includes P- effects, therefore B_2 is taken as unity for bending in both directions. It is suggested that the P- 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, , is taken as follows (LRFD A5.3):

$_t$ = Resistance factor for tension, 0.9 (LRFD D1, H1, SAM 2, 6)
$_c$ = Resistance factor for compression, 0.85 (LRFD E2, E3, H1)
$_c$ = Resistance factor for compression in angles, 0.90 (LRFD SAM 4, 6)
$_b$ = Resistance factor for bending, 0.9 (LRFD F1, H1, A-F1, A-G2, SAM 5)
$_v$ = 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 "Overwrites" 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, $_c$, where

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

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

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:

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

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

$$F_{cr} = \frac{0.877}{{}^{2}_{c}} F_y , \quad \text{for} \quad {}_{c} > 1.5. \qquad \text{(LRFD E2-3)}$$

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

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

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

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

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	$Q_s = \begin{array}{lll} 1.0\ , & if & b_f/2t_f \quad 95/\sqrt{F_y}\ , \\ 1.415 \quad 0.00437[b_f/2t_f]\sqrt{F_y}\ , & if & 95/\sqrt{F_y} < b_f/2t_f < 176/\sqrt{F_y}\ , \\ 20{,}000/\{[b_f/2t_f]^2 F_y\}, & if & b_f/2t_f \quad 176/\sqrt{F_y}\ . \end{array}$ (rolled)	LRFD A-B5-5, LRFD A-B5-6
	$Q_s = \begin{array}{lll} 1.0 & if & b_f/2t_f \quad 109/\sqrt{F_y/k_c}\ , \\ 1.415 \quad 0.00381[b_f/2t_f]\sqrt{F_y/k_c} & if & 109/\sqrt{F_y/k_c} < b_f/2t_f < 200/\sqrt{F_y/k_c}\ , \\ 26{,}200\, k_c/\{[b_f/2t_f]^2 F_y\} & if & b_f/2t_f \quad 200/\sqrt{F_y/k_c}\ . \end{array}$ (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. $Q_s \quad \begin{array}{lll} 1.0\ , & if & d/t_w \quad 127/\sqrt{F_y}\ , \\ 1.908 \quad 0.00715[d/t_w]\sqrt{F_y}\ , & if & 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}\ , \\ 20{,}000/\{[d/t_w]^2 F_y\}, & if & d/t_w \quad 176/\sqrt{F_y}\ . \end{array}$	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)	$Q_s = \begin{array}{lll} 1.0\ , & if & b/t \quad 76/\sqrt{F_y}\ , \\ 1.340 \quad 0.00447[b/t]\sqrt{F_y}\ , & if & 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}\ , \\ 15{,}500/\{[b/t]^2 F_y\}, & if & b/t \quad 155/\sqrt{F_y}\ . \end{array}$	LRFD A-B5-3, LRFD A-B5-4
ANGLE	$Q_s = \begin{array}{lll} 1.0\ , & if & b/t \quad 0.446\sqrt{F_y/E}\ , \\ 1.34 \quad 0.761[b/t]\sqrt{F_y/E}\ , & if & 0.446\sqrt{F_y/E} < b/t < 0.910\sqrt{F_y/E}\ , \\ 0.534/\{[b/t]^2[F_y/E]\}, & if & b/t \quad 0.910\sqrt{F_y/E}\ . \end{array}$	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	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}} \ 1 \ \frac{57.2}{(h/t_w)\sqrt{f}}, & if \ \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
BOX	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}} \ 1 \ \frac{57.2}{(h/t_w)\sqrt{f}}, & if \ \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	LRFD A-B5-12
	$b_e = \begin{cases} b, & if \ \frac{b}{t_f} \ \frac{238}{\sqrt{f}}, \\ \frac{326\,t_f}{\sqrt{f}} \ 1 \ \frac{64.9}{(b/t_f)\sqrt{f}}, & if \ \frac{b}{t_f} > \frac{238}{\sqrt{f}}. \end{cases}$ (compr. or flexure, $f = F_y$)	LRFD A-B5-11
CHANNEL	$h_e = \begin{cases} h, & if \ \frac{h}{t_w} \ \frac{253}{\sqrt{f}}, \\ \frac{326\,t_w}{\sqrt{f}} \ 1 \ \frac{57.2}{(h/t_w)\sqrt{f}}, & if \ \frac{h}{t_w} > \frac{253}{\sqrt{f}}. \end{cases}$ (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	$Q_a = \begin{cases} 1, & if \ \frac{D}{t} \ \frac{3{,}300}{F_y}, \\ \frac{1{,}100}{(D/t)F_y} + \frac{2}{3}, & if \ \frac{D}{t} > \frac{3{,}300}{F_y}. \end{cases}$ (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:

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

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 $_r$ is given by

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

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

$$F_{crz} = \frac{GJ}{Ar_0^2}\,,$$

$$H = 1 \quad \frac{x_0^2 + y_0^2}{r_0^2} \div,$$

$r_0 =$ 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 $_c = \frac{Kl}{r_{22}} \sqrt{\frac{F_y}{E}}$.

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:

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

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

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

In the above equations, the slenderness parameter $_e$ is calculated as

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

where F_e is calculated as follows:

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

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

- For T-sections and Double-angles:

$$F_e = \frac{F_{e22} + F_{ez}}{2H} \div\ 1 \quad \sqrt{1 \quad \frac{4\, F_{e22} F_{ez} H}{(F_{e22} + F_{ez})^2}} \qquad \text{(LRFD A-E3-6)}$$

- For Channels:

$$F_e = \frac{F_{e33} + F_{ez}}{2H} \div\ 1 \quad \sqrt{1 \quad \frac{4\, F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \qquad \text{(LRFD A-E3-6)}$$

- For Single-angles sections with equal legs:

$$F_e = \frac{F_{e33} + F_{ez}}{2H} \div\ 1 \quad \sqrt{1 \quad \frac{4\, F_{e33} F_{ez} H}{(F_{e33} + F_{ez})^2}} \qquad \text{(LRFD A-E3-6)}$$

- 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):

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

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,

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

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

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

$$F_{ez} = \frac{{}^2EC_w}{\left(K_z l_z\right)^2} + GJ \quad \frac{1}{Ar_0^2}, \qquad \text{(LRFD A-E3-12)}$$

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 the program,

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.

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

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):

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

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

where,

M_{n33} = Nominal major bending strength,

M_{p33} = Major plastic moment, $Z_{33} F_y \quad 1.5 S_{33} F_y$, (LRFD F1.1)

M_{r33} = Major limiting buckling moment,

$(F_y \quad F_r) S_{33}$ 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,

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

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)

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

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

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

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

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

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

M_{max}, M_A, M_B, and M_C are absolute values of maximum moment, ¼ point, center of span and ¾ 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:

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

For pipes and circular bars bent about any axis,

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

T-sections and Double Angles

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

$$M_{n33} = \frac{\sqrt{EI_{22}GJ}}{L_b} \quad B + \sqrt{1 + B^2} \quad \text{, where} \qquad \text{(LRFD F1-15)}$$

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

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

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

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):

$$M_{n,major} = 0.92 \quad 0.17 \frac{M_{ob}}{M_{y,major}} \quad M_{ob} \quad 1.25 M_{y,major}, \quad if\, M_{ob} \quad M_{y,major},$$

$$M_{n,major} = 1.58 \quad 0.83 \sqrt{\frac{M_{y,major}}{M_{ob}}} \quad M_{y,major} \quad 1.25 M_{y,major}, \; if\, M_{ob} > M_{y,major},$$

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

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

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,

$${}_w = \frac{1}{I_{max}} \;{}_A z(w^2 + z^2)\,dA \quad 2z_0, \qquad \text{(LRFD SAM 5.3.2)}$$

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.

$_w$ 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 the program, 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,

$$M_{n33} = \begin{cases} M_{p33}, & \textit{if} \quad {}_p, \\ M_{p33} \quad (M_{p33} \quad M_{r33}) \dfrac{\quad {}_p}{{}_r \quad {}_p} \div, & \textit{if} \quad {}_p < \quad {}_r, \\ M_{cr33} \quad M_{p33}, & \textit{if} \quad > \quad {}_r. \end{cases} \qquad \text{(A-F1-3)}$$

and for minor direction bending,

$$M_{n22} = \begin{cases} M_{p22}, & \textit{if} \quad {}_p, \\ M_{p22} \quad (M_{p22} \quad M_{r22}) \dfrac{\quad {}_p}{{}_r \quad {}_p} \div, & \textit{if} \quad {}_p < \quad {}_r, \\ M_{cr22} \quad M_{p22}, & \textit{if} \quad > \quad {}_r. \end{cases} \qquad \text{(A-F1-3)}$$

where,

M_{n33} = Nominal major bending strength,
M_{n22} = Nominal minor bending strength,
M_{p33} = Major plastic moment, $Z_{33}F_y \quad 1.5\, S_{33}F_y$,
M_{p22} = Minor plastic moment, $Z_{22}F_y \quad 1.5\, S_{22}F_y$,
M_{r33} = Major limiting buckling moment,
M_{r22} = Minor limiting buckling moment,
M_{cr33} = Major buckling moment,
M_{cr22} = Minor buckling moment,
= Controlling slenderness parameter,
$_p$ = Largest value of for which $M_n = M_p$, and
$_r$ = Largest value of for which buckling is inelastic.

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

I Shapes, Channels

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

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

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

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

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

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

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

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

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

Boxes

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

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

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

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

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

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

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

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

$S_{eff,33}$ = 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 the program. 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):

$$M_n = \begin{array}{lll} 1.25 F_y S_c , & if & \frac{b}{t} \quad 0.382\sqrt{\frac{\mathrm{E}}{F_y}}, \\ F_y S_c \quad 1.25 \quad 1.49 \dfrac{b/t}{0.382\sqrt{\frac{\mathrm{E}}{F_y}}} \quad 1 , & if & 0.382\sqrt{\frac{\mathrm{E}}{F_y}} < \frac{b}{t} \quad 0.446\sqrt{\frac{\mathrm{E}}{F_y}}, \\ \mathrm{Q} F_y S_c , & if & \frac{b}{t} > 0.446\sqrt{\frac{\mathrm{E}}{F_y}}, \end{array}$$

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

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

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

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

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

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

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 the program 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 the program 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):

$$M_{n33} = \begin{cases} M_{p33}, & \text{if } \quad {}_{p}, \\ M_{p33} \quad (M_{p33} \quad M_{r33}) \dfrac{\quad {}_{p}}{{}_{r} \quad {}_{p}} \div, & \text{if } \quad {}_{p} < \quad {}_{r}, \text{(A-F1,A-G1)} \\ S_{33} R_{PG} R_e F_{cr}, & \text{if } \quad > \quad {}_{r}, \end{cases}$$

where,

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

$_r$ = Largest value of 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:

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

$$_p = \begin{cases} \dfrac{640}{\sqrt{F_y}} \quad 1 - 2.75 \dfrac{P_u}{_b P_y} \div, & \text{for} \quad \dfrac{P_u}{_b P_y} \quad 0.125, \\ \dfrac{191}{\sqrt{F_y}} \quad 2.33 - \dfrac{P_u}{_b P_y} \div \quad \dfrac{253}{\sqrt{F_y}}, & \text{for} \quad \dfrac{P_u}{_b P_y} > 0.125, \text{ and} \end{cases}$$

$$_r = \frac{970}{\sqrt{F_y}} \quad 1 - 0.74 \frac{P_u}{_b P_y} \div.$$

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

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

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

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

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

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

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

$$F_{cr} = \begin{cases} F_y, & if \quad {}_p, \\ C_b F_y \quad 1 \quad \frac{1}{2} \frac{\quad {}_p}{{}_r \quad {}_p} \quad F_y, & if \quad {}_p < \quad {}_r, \\ \frac{C_{PG}}{2}, & if \quad > {}_r, \end{cases} \qquad \text{(LRFD A-G2-4, 5, 6)}$$

The parameters , $_p$, $_r$, and C_{PG} for lateral-torsional buckling for slender web I, Channel and Box sections are given below:

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

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

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

$C_{PG} = 286{,}000\ C_b$, and (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 the program.

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

The parameters , $_p$, $_r$, and C_{PG} for flange local buckling for slender web I, Channel and Box sections are given below:

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

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

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

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

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

T-sections and Double Angles

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

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 the program.

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 the program 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 $\dfrac{h}{t_w} \qquad \dfrac{418}{\sqrt{F_y}}$,

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

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

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

for $\dfrac{523}{\sqrt{F_y}} < \dfrac{h}{t_w} \qquad 260$,

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

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 $\frac{P_u}{P_n}$. If P_u is tensile, P_n is the nominal axial tensile strength and $=\ {}_t = 0.9$; and if P_u is compressive, P_n is the nominal axial compressive strength and $=\ {}_c = 0.85$, except for angle sections $=\ {}_c = 0.90$ (LRFD SAM 6). In addition, the resistance factor for bending, ${}_b = 0.9$.

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

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

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

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

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 the program. 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:

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

$$\frac{V_{u3}}{{}_{v}V_{n3}} ,$$

where ${}_{v} = 0.9$.

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 BS 5950-2000

This chapter describes the details of the structural steel design and stress check algorithms that are used by the program when the user selects the BS 5950-00 design code (BSI 2000). 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 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 V-1

BS 5950-2000 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
	=	Ratio of smaller to larger end moments
	=	Constant $\left(\frac{275}{\ _y}\right)^{½}$
	=	Slenderness parameter
$_o$	=	Limiting slenderness
$_{LT}$	=	Equivalent slenderness
$_{Lo}$	=	Limiting equivalent slenderness
	=	Perry factor
$_{LT}$	=	Perry coefficient
$_c$	=	Compressive strength, MPa
$_E$	=	Euler strength, MPa
$_y$	=	Yield strength, MPa
	=	Monosymmetry index

Table V-1

BS 5950-2000 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 5950-00 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 5950-00 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 and 2.3.2.4). 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 5.1.2.4). It is recommended that the user should define additional load cases for considering the notional load in the program and define the appropriate design combinations.

When using the BS 5950-00 code, the program design assumes that a P- 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- 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. Program checks the sections according to Table V-2 (BS 3.5.2). The parameters r_1, r_2 and along with the slenderness ratios are the major factors in classification of section.

- r_1 and r_2 is the ratio of mean longitudinal stress in the web to $\ _y$ in a section. This implies that for a section in pure bending r_1 and r_2 are zero. In calculating r_1 and r_2, compression is taken as positive and tension is taken as negative. For I, H, RHS and welded box sections with equal flanges, r_1 and r_2 are calculated as follows:

$$r_1 = \frac{P}{dt\ \ _y} \qquad where \qquad 1 < r_1 \quad 1$$

$$r_2 = \frac{P}{A_g\ \ _y}$$

- is defined as follows:

$$= \left(\frac{275}{\ _y}\right)^{1/2}$$

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 program does not check stresses for Slender sections.**

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.

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Semi-compact)
I-SHAPE	b/T (Rolled)	9	10	15
	b/T (welded)	8	9	13
	b/T (Compression due to bending)	28	32	40
	b/T (Axial compression)	Not Applicable	Not applicable	40
	d/t (Neutral axis at mid-depth)	80	100	120
	d/t (r_1 is negative)	$\frac{80}{1+r_1}$ 40	$\frac{100}{1+r_1}$ 40	$\frac{120}{1+2r_2}$ 40
	d/t (r_2 is positive)	$\frac{80}{1+r_1}$ 40	$\frac{100}{1+1.5r_1}$ 40	$\frac{120}{1+2r_2}$ 40
	d/t (Axial compression)	Not applicable	Not applicable	$\frac{120}{1+2r_2}$ 40
BOX	b/T (Flange compression due to bending)	28 80 d/t	32 80 d/t	40
	b/T (Flange axial compression)	Not applicable	Not applicable	40
	d/t (Web neutral axis at mid-depth)	64	80	120

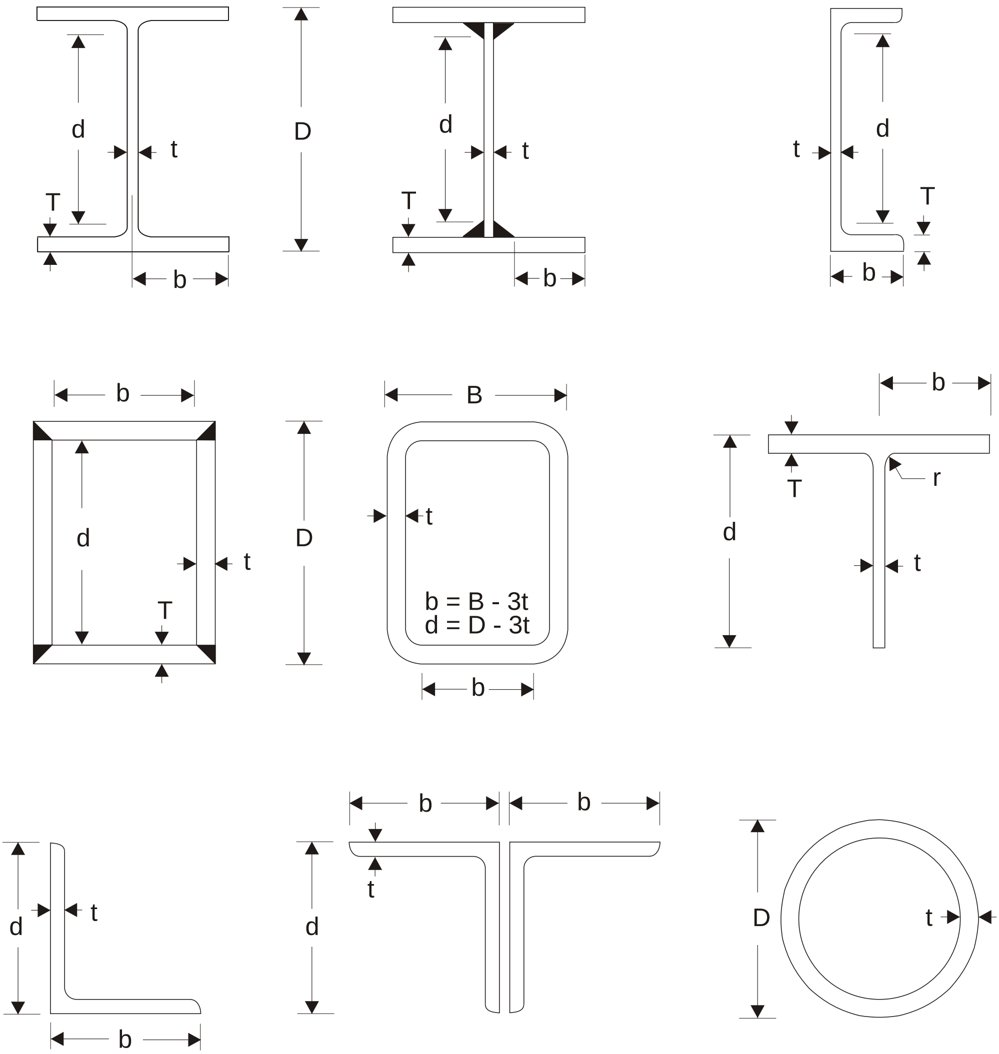
Table V-2

Limiting Width-Thickness Ratios for
Classification of Sections based on BS 5950-2000

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Semi-compact)
BOX	d/t (Generally)	$\frac{64}{1+0.6r_1}$ 40	$\frac{80}{1+r_1}$ 40	$\frac{120}{1+2r_2}$ 40
	d/t (web axial compression)	Not applicable	Not applicable	40
CHANNEL	d/t	40	40	40
T-SHAPE	d/t	8	9	18
ANGLE, **DOUBLE ANGLE** (separated) (Axial compression)	d/t d/t $(b+d)/t$	Not applicable	Not applicable	15 15 24
ANGLE (compression due to bending)	b/t d/t	9 9	10 10	15 15
PIPE (compression due to bending) (Axial compression)	D/t D/t	40 [2] Not applicable	50 [2] Not applicable	140 [2] 80 [2]
SOLID CIRCLE		Assumed Compact		
SOLID RECTANGLE		Assumed Compact		

Table V-2 (cont.)

Limiting Width-Thickness Ratios for Classification of Sections based on BS 5950-2000



BS 5950 : Axes Conventions

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

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

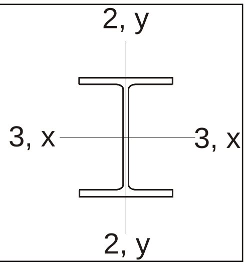


Figure V-1

BS 5950-2000 Definition of Geometric Properties

The moment magnification for non-sidesway moments is included in the overall buckling interaction equations.

$$M = M_g + \frac{1}{1 \quad 200 \quad _{s,max}} M_s \text{, where} \qquad \text{(BS 5.6.6, Appendix E)}$$

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

The moment magnification factor for moments causing sidesway can be taken as unity if a P- analysis is carried out. the program design assumes a P- analysis has been done and, therefore, $_{s,max}$ for both major and minor direction bending is taken as 0. It is suggested that the P- 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, program takes the design strength, $_y$, 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).

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

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 "Overwrites", 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:

Description of Section	Thickness (mm)	Axis of Bending	
		Major	Minor
I-SHAPE (rolled)	40 > 40	2.0 3.5	3.5 5.5
H-SHAPE (rolled)	40 > 40	3.5 5.5	5.5 8.0
I or H-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 V-3
Robertson Constant in BS 5950-2000

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

where ${}_c$ is the compressive strength given by

$$ {}_c = \frac{{}_E \; {}_y}{+(\;{}^2 - {}_E \; {}_y)^{1/2}} \text{, where} \qquad \text{(BS C.1)}$$

$$ = \frac{{}_y + (\; + 1) \; {}_E}{2}, \qquad \text{(BS C.1)}$$

$_E$ = Euler strength, $^2E \diagup {}^2$,

= Perry factor, $0.001\,a\,(\; - \; _0) \quad 0$, (BS C.2)

a = Robertson constant from Table V-3, (BS C2, BS Table 25) $_0$ =

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

= the slenderness ratio in either the major, $_{33} = l_{e33} / r_{33}$, or in the minor, $_{22} = l_{e22} / r_{22}$ direction (BS 4.7.2). 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 is greater than 180, a message to that effect is printed although there is no such limitation exists in BS 5950-2000 code.

Tension Capacity

The tension capacity of a member is given by

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

It should be noted that no net section checks are made. If is greater than 250, a message is displayed although there is no such limitation exists in BS 5950-2000 code.

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

$$M_c = \ {}_y S \quad 1.5 \quad {}_y Z \,, \qquad \text{for} \quad F_v \quad 0.6 P_v \,, \qquad \text{(BS 4.2.5.2)}$$

$$M_c = \ {}_y (S \quad S_v \quad) \quad 1.5 \quad {}_y Z \,, \quad \text{for} \quad F_v > 0.6 P_v \,, \qquad \text{(BS 4.2.5.3)}$$

except for simply supported and cantilever beams, the bending about the 3-3 axis the moment capacities considering the effects of shear force are computed as

$$M_c = \ {}_y S \quad 1.2 \quad {}_y Z \,, \qquad \text{for} \quad F_v \quad 0.6 P_v \,, \qquad \text{(BS 4.2.5.2)}$$

$$M_c = \ {}_y (S \quad S_v \quad) \quad 1.2 \quad {}_y Z \,, \quad \text{for} \quad F_v > 0.6 P_v \,, \qquad \text{(BS 4.2.5.3)}$$

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,

$$= \quad \frac{2F_v}{P_v} \quad 1 \quad ^{2} \,.$$

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 program as

$$M_c = \ {}_y S \quad 1.2 \quad {}_y Z \,., \text{ for simply supported/cantilevered beam} \qquad \text{(BS 4.2.5.2)}$$

$$M_c = \ {}_y S \quad 1.5 \quad {}_y Z \,., \text{ for simply supported/cantilevered beam} \qquad \text{(BS 4.2.5.2)}$$

Semi-compact Sections

For semi-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:

$$M_c = \ _y Z \ , \qquad \text{for} \quad F_v \quad 0.6 P_v \ , \qquad \text{(BS 4.2.5.2)}$$

$$M_c = \ _y (Z \quad S_v \quad / 1.5) \ , \qquad \text{for} \quad F_v > 0.6 P_v \ , \qquad \text{(BS 4.2.5.3)}$$

Lateral-Torsional Buckling Moment Capacity

The lateral torsional buckling strength, $_b$, 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

$$_b = \frac{\ _E \ _y}{\ _L + (\ _L^2 - \ _E \ _y)^{1/2}} \ , \ \text{where} \qquad \text{(BS B2.1)}$$

$$_L = \frac{\ _y + (\ _{LT} + 1) \ _E}{2} \ ,$$

$$_E = \ \text{The elastic critical stress,} \ \frac{^2 E}{\ _{LT}^2} \ , \text{and} \qquad \text{(BS B2.3)}$$

$$_{LT} = \ \text{The Perry coefficient.}$$

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

For rolled sections

$$_{LT} = \ L \ \{ \ _{LT} \quad _{L0} \} \quad 0 \ , \ \text{and} \qquad \text{(BS B2.3)}$$

for welded sections

$$_{LT} = \ 2 \ _{LT} \ _{L0} \quad 0,$$

$$\text{with} \ _{LT} (\ _{LT} \quad _{L0}) \quad _{LT} \quad 2 \ _L \ (\ _{LT} \quad _{L0}). \qquad \text{(BS B2.2)}$$

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

$$_{L0} = 0.4\sqrt{\frac{^{2}E}{_{y}}}, \qquad \text{(BS B2.29b)}$$

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

$$_{LT} = 2.25\sqrt{\ _{b}\ }, \qquad \text{(BS B2.3)}$$

and for Box sections $_{LT}$ is defined as

$$_{LT} = 2.25\ n\left(\ _{b}\ \right)^{1/2}, \text{ where} \qquad \text{(BS B2.6)}$$

- is the slenderness and is equivalent to l_{e22}/r_{22}. (BS B4.3.6.7)
- The ratio $_{w}$ should be taken as follows.

  for class 1 plastic or class 2 compact cross-section:

  $_{w} = 1.0$

  for class 3 semi-compact cross-section:

  if $M_b = \ _{b}Z_x$ then $_{w} = \dfrac{Z_x}{S_x}$, and

  if $M_b = \ _{b}S_{x\ ef}$ then $_{w} = \dfrac{S_{x\ eff}}{S_x}$

- 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,

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

$$u = \frac{4S_{33}^{2}}{A^{2}(D \quad T/2)^{2}} \div^{1/4}, \qquad \text{for T section, where} \qquad \text{(BS B2.5b)}$$

$$= 1 \quad \frac{I_{22}}{I_{33}} \div .$$ (BS B2.5b)

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

$$v = \frac{1}{\left[4N(N \quad 1) + \frac{1}{20} \left(\frac{}{x} \right)^2 + \quad ^2 \right]^{½} + \quad ^{½}} , \text{ where}$$ (BS B2.4)

$$N = \begin{cases} 0.5\ , & \text{for I, Channel, Double-Channel sections,} \\ 1.0\ , & \text{for T sections with flange in compression,} \\ 0.0\ , & \text{for T sections with flange in tension, and} \end{cases}$$ (BS B2.4)

$$= \begin{cases} 0.0\ , & \text{for I, Channel, Double-Channel sections,} \\ 0.8\ , & \text{for T sections with flange in compression, and} \\ -1.0\ , & \text{for T sections with flange in tension.} \end{cases}$$ (BS B2.4)

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

$$_b = \left(\frac{S_{33}^2}{AJ} \div \right)^{1/2} , \text{ where}$$ (BS B2.6.1)

$$= 1 \quad \frac{I_{22}}{I_{33}} \div \quad 1 \quad \frac{J}{2.6 I_{33}} \div .$$ (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:

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

$$P_{v3} = 0.6 \quad _y A_{v3} .$$ (BS 4.2.3)

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 $t\left(d \quad T\right)$	$0.9\left(2bT\right)$ $0.9\left(2bT\right)$
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 V-4

Shear Area in BS 5950-2000

The shear areas A_{v3} and A_{v2} are given in Table V-4.

Moreover, the shear capacity computed above is valid only if $d/t \quad 70$ for rolled section and $d/t \quad 62$ for welded section. For $d/t \quad 70$ for rolled section and $d/t \quad 62$ for welded section, 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.

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.

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

Under Axial Compression

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

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

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:

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

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

$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{\max}} \quad \frac{0.8\mathrm{M}_{24}}{M_{\max}}. \qquad \text{(BS Table 26)}$$

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. The moment M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length. If M_2 , M_3 and M_4 all lie on the same side of the axis, their values are taken as positive. If they lie both sides of the axis, the side leading to the larger value is taken as the positive side. The values $M_{\max}$ and M_{24} are always taken as positive. $M_{\max}$ is the moment in the segment and the M_{24} is the maximum moment in the central half of the segment.

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 **VI**

Design Output

Overview

The program 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,
- $_s$-factors,
- $_b$-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 program 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 program 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,
  - $_{b}$-factors, and
  - $_{s}$-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,
  - $_s$-factors, and
  - $_b$-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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