COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

CSiBRIDGE® 2017

Integrated 3-D Bridge Analysis, Design and Rating

Steel Frame Design Manual

AISC ASD 01 & AISC ASD 89

Steel Frame Design Manual

AISC ASD-1989 and AISC ASD-01

For CSiBridge®

ISO BRG102816M31 Rev 0
Proudly developed in the United States of America October 2016

Copyright

Further information and copies of this documentation may be obtained from:

Computers & Structures, Inc.
http://www.csiamerica.com/

info@csiamerica.com (for general information)
support@csiamerica.com (for technical support)

DISCLAIMER

CONSIDERABLE TIME, EFFORT AND EXPENSE HAVE GONE INTO THE DEVELOPMENT AND DOCUMENTATION OF THIS SOFTWARE. HOWEVER, THE USER ACCEPTS AND UNDERSTANDS THAT NO WARRANTY IS EXPRESSED OR IMPLIED BY THE DEVELOPERS OR THE DISTRIBUTORS ON THE ACCURACY OR THE RELIABILITY OF THIS PRODUCT.

THIS PRODUCT IS A PRACTICAL AND POWERFUL TOOL FOR STRUCTURAL DESIGN. HOWEVER, THE USER MUST EXPLICITLY UNDERSTAND THE BASIC ASSUMPTIONS OF THE SOFTWARE MODELING, ANALYSIS, AND DESIGN ALGORITHMS AND COMPENSATE FOR THE ASPECTS THAT ARE NOT ADDRESSED.

THE INFORMATION PRODUCED BY THE SOFTWARE MUST BE CHECKED BY A QUALIFIED AND EXPERIENCED ENGINEER. THE ENGINEER MUST INDEPENDENTLY VERIFY THE RESULTS AND TAKE PROFESSIONAL RESPONSIBILITY FOR THE INFORMATION THAT IS USED.

Table of Contents

Chapter I

Introduction

Overview

SAP2000 and 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 program currently supports the following steel design codes:

- U.S. AISC 360-2005/IBC 2006
- U.S. AISC ASD-2001,
- U.S. AISC LRFD-1999,

- U.S. AISC ASD-1989,
- U.S. AISC-LRFD-1994,
- UBC ASD-1997,
- UBC LRFD-1997,
- Canadian CAN/CSA-S16.1-1994,
- British BS 5950-2200,
- British BS 5950-1990, and
- Eurocode 3 (ENV 1993-1-1).

The design is based upon a set of user-specified loading combinations. However, the program provides a set of default load combinations for each design code supported in 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 1997 UBC-ASD or UBC-LRFD design codes, requirements for continuity plates at the beam to column connections are evaluated. The program performs a joint shear analysis to determine if doubler plates are required in any of the joint panel zones. Maximum beam shears required for the beam shear connection design are reported. Also maximum axial tension or compression values that are generated in the member are reported.

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

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

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

Organization

This manual is organized in the following way:

Chapter II outlines various aspects of the steel design procedures of the 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 code (AISC 2001) as implemented in the program.
- Chapter IV gives a detailed description of the AISC ASD steel code (AISC 1989) as implemented in the program.
- Chapter V gives a detailed description of the AISC LRFD code (AISC 1999) as implemented in the program.
- Chapter VI gives a detailed description of the AISC LRFD steel code (AISC 1993) as implemented in the program.
- Chapter VII gives a detailed description of the British code BS 5950 (BSI 2000) as implemented in the program.

- Chapter IIIV gives a detailed description of the British code BS 5950 (BSI 1990) as implemented in the program.
- Chapter IX gives a detailed description of the Canadian code (CISC 1994) as implemented in the program.
- Chapter X gives a detailed description of the Eurocode 3 (CEN 1992) as implemented in the program.
- Chapter XI gives a detailed description of the UBC ASD (UBC 1997) as implemented in the program.
- Chapter XII gives a detailed description of the UBC (UBC 1997) as implemented in the program.

Chapter XIII 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 XIII 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 2001).
- 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 1999)
- 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).
- British Standards Institution's "Structural Use of Steelwork in Building", **BS 5950** (BSI 1990).

- Canadian Institute of Steel Construction's "Limit States Design of Steel Structures", **CAN/CSA-S16.1-94** (CISC 1995).
- European Committee for Standardization's "Eurocode 3: Design of Steel Structures C Part 1.1: General Rules and Rules for Buildings", **ENV 1993-1-1** (CEN 1992).
- International Conference of Building Officials' "1997 Uniform Building Code: Volume 2: Structural Engineering Design Provisions" Chapter 22 Division III "Design Standard for Specification for Structural Steel Buildings Allowable Stress Design and Plastic Design", **UBC-ASD** (ICBO 1997)**.**
- International Conference of Building Officials' "1997 Uniform Building Code: Volume 2: Structural Engineering Design Provisions" Chapter 22 Division II "Design Standard for Load and Resistance factor Design Specification for Structural Steel Buildings", **UBC-LRFD** (ICBO 1997)**.**

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 Canadian code carry the prefix of "**CISC**"

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

– References to the Eurocode carry the prefix of "**EC3**"

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

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.

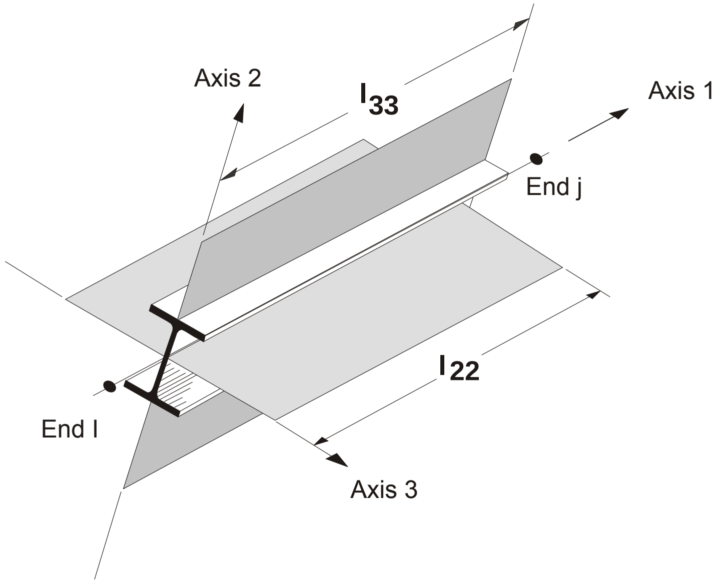

Figure II-1
Major and Minor Axes of Bending

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.

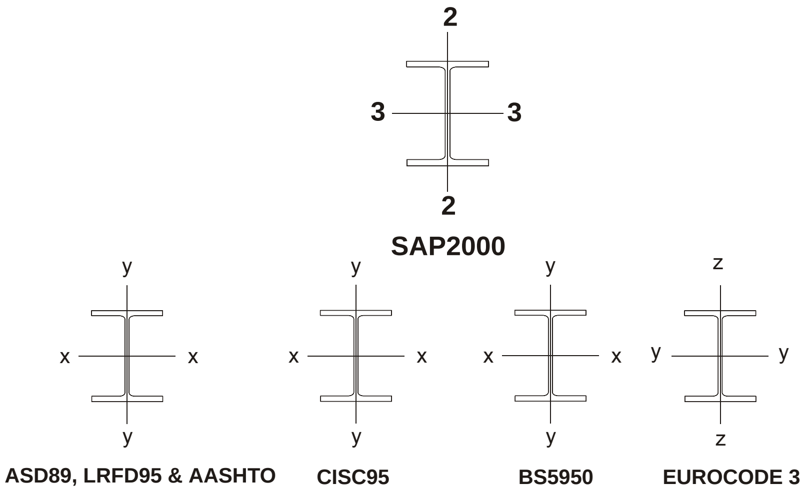

Figure II-2
Correspondence between the program Axes and Code Axes

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.

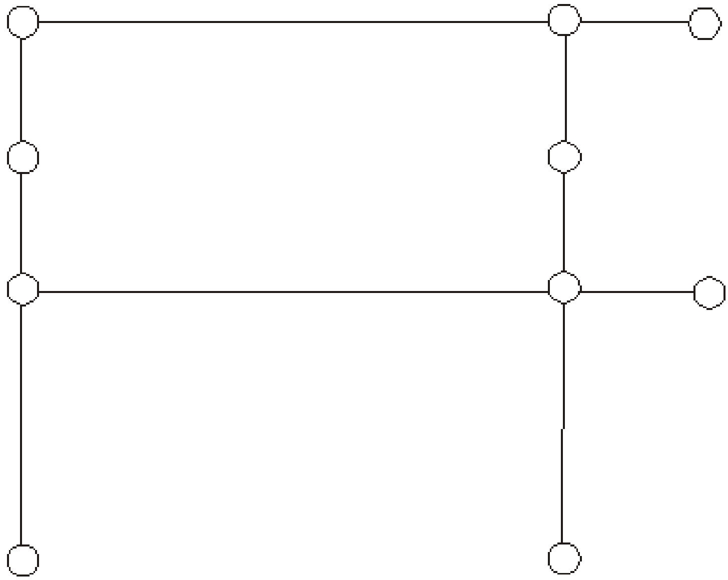

Figure II-3
Unsupported Lengths are Affected by Intermediate Nodal Points

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 :

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

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-ASD01

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-ASD01 design code (AISC 2001). Various notations used in this chapter are described in Table IV-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}	=	$\dfrac{12 \quad {}^{2} E}{23\left(K_{33} l_{33} / r_{33}\right)^{2}}$
F_{e22}	=	$\dfrac{12 \quad {}^{2} E}{23\left(K_{22} l_{22} / r_{22}\right)^{2}}$
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, = $Q_a Q_s$
Q_a	=	Reduction factor for stiffened slender elements
Q_s	=	Reduction factor for unstiffened slender elements
S	=	Section modulus, in^3
S_{33}, S_{22}	=	Major and minor section moduli, in^3

Table III-1

AISC-ASD Notations

$S_{eff,33}, S_{eff,22}$	=	Effective major and minor section moduli for slender sections, in^3
S_c	=	Section modulus for compression in an angle section, in^3
V_2, V_3	=	Shear forces in major and minor directions, kips
b	=	Nominal dimension of plate in a section, in longer leg of angle sections, $b_f \quad 2t_w$ 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
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 $(d \quad 2t_f)$, 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 $h/t_w \quad 70$.
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-ASD01 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). The $DL_{multiplier}$ and factors are specified in ASCE 7-02:

$$1.0\ \text{DL} \qquad \text{(ASCE 2.4.1-1)}$$
$$1.0\ \text{DL} + 1.0\ \text{LL} \qquad \text{(ASCE 2.4.1-2)}$$

$$1.0\ \text{DL} \pm 1.0\ \text{WL} \qquad \text{(ASCE 2.4.1-5)}$$
$$0.6\ \text{DL} \pm 1.0\ \text{WL} \qquad \text{(ASCE 2.4.1-7)}$$
$$1.0\ \text{DL} + 0.75\ (1.0\ \text{LL} \pm 1.0\ \text{WL}) \qquad \text{(ASCE 2.4.1-6)}$$

$$1.0\ \text{DL}\ (0.6 - 0.7\ DL_{multiplier}) \pm 0.7\ \ \text{EL} \qquad \text{(ASCE 2.4.1-8)}$$
$$1.0\ \text{DL}\ (1 + 0.7\ DL_{multiplier}) \pm 1.0\ \ \text{EL} \qquad \text{(ASCE 2.4.1-5)}$$
$$1.0\ \text{DL} + 0.75\ (0.7\ DL_{multiplier} + 1.0\ \text{LL} \pm 0.7\ \ \text{EL}) \qquad \text{(ASCE 2.4.1-6)}$$

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

$$(0.9\text{-}0.2S_{DS})\ \text{DL} \pm\ {}_0\ \text{EL} \qquad \text{(ASCE 9.5.2.7.1, 2.3, LRFD SEISMIC 4.1)}$$

$$(1.2 + 0.2SS_{DS})\ \text{DL} + 1.0\ \text{LL} \pm\ {}_0\ \text{EL}$$

where, ${}_0$ is the seismic force amplification factor which is required to account for structural overstrength. The default value of ${}_0$ is taken as 3.0 in the program. If the user defines one or more auto-seismic loads, then the value of ${}_0$ defined for each auto-seismic load cases. Also if special seismic data is defined by the user, the user specifies an ${}_0$ value, and the user requests the program to include the special seismic design data, then the user specified ${}_0$ takes precedence over the default

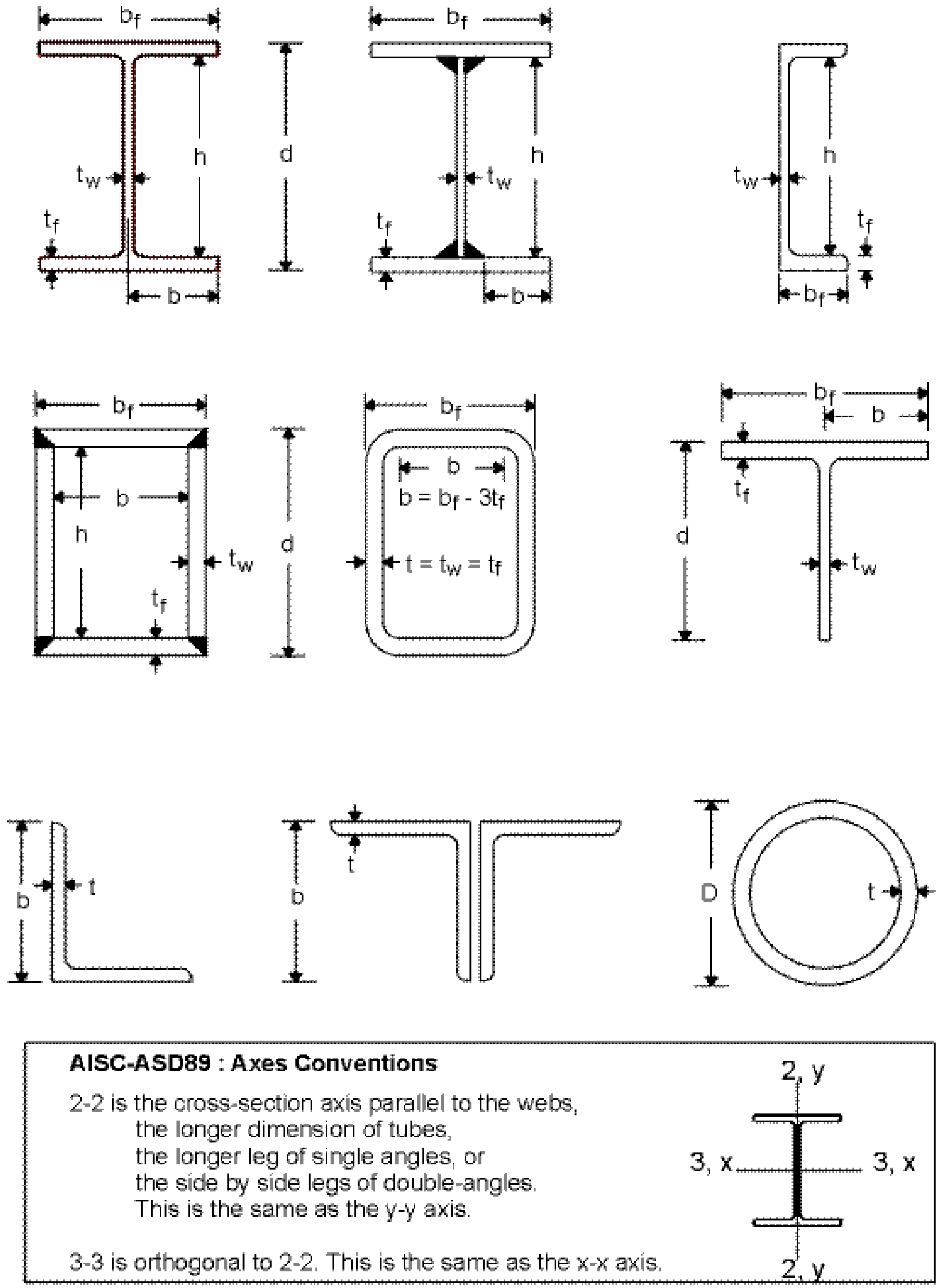

Figure III-1

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 $f_a/F_y \quad 0.16$ $\frac{640}{\sqrt{F_y}}(1 \quad 3.74\frac{f_a}{F_y})$, 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 $760/\sqrt{F_b}$	$\frac{14000}{\sqrt{F_y\left(F_y+16.5\right)}}$ 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 $b_f/d_w \quad 0.25$, $t_f/t_w \quad 3.0$ If rolled $b_f/d_w \quad 0.5$, $t_f/t_w \quad 2.0$

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.)

values and those defined for the auto-seismic load cases. Moreover, $_0$ can be overwritten for each individual member. The overwritten $_0$ gets the highest precedence. The guidelines for selecting a reasonable value for $_0$ can be found in ANSI/AISC 341 SEISMIC section 4.1 and Table I-4-1.

These are also the default design load combinations in the program whenever the AISC-ASD01 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, if other types of loads are present, or if pattern live loads are to be considered.

When designing for combinations involving earthquake and Wind loads, allowable stresses are **NOT** increased by the 4/3 factor per the ASD Supplement No. 1 which references ASCE7. For seismic combinations, the allowable stresses are increased by 1.7 factor in accordance with ANSI/AISC 341 Seismic Section 4.2.

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.

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 the program.**

In general the design sections need not necessarily be Compact to satisfy ANSI/AISC 341-02 codes. However, for certain special seismic cases they have to be Compact and have to satisfy special slenderness requirements. See subsection "Seismic Requirements" later in this manual. The sections which do satisfy these additional requirements are classified and reported as "SEISMIC" in the program. These special requirements for classifying the sections as "SEISMIC" in the program are given in Table III-3 (ANSI/AISC 341SEISMIC 8.2, Table I-8-1). If these criteria are not satisfied, when the code requires them to be satisfied, the user must modify the section property. In this case the program gives an error message in the output file.

Description of Section	**Width-Thickness Ratio** ( )	**COMPACT (SEISMIC ZONE)** ($_{ps}$)	**NONCOMPACT (Uniform Compression)** (M_{22} M_{33} 0) ($_{r}$)
I-SHAPE	$b_f / 2t_f$ (rolled)	$0.3\sqrt{\frac{E}{F_y}}$	$0.56\sqrt{E / F_y}$
	$b_f / 2t_f$ (welded)	$0.3\sqrt{\frac{E}{F_y}}$	$0.56\sqrt{E / F_y}$
	h_c / t_w	For $P_u / {}_bP_y$ 0.125, $3.14\sqrt{\frac{E}{F_y}}\left(1 - 1.54\frac{P_u}{{}_bP_y}\right)$ For $P_u / {}_bP_y > 0.125$ $1.12\sqrt{\frac{E}{F_y}}\left(2.33 - \frac{P_u}{{}_bP_y}\right)$ $1.49\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{E / F_y}$
BOX	b / t_f h_c / t_w	$0.64\sqrt{\frac{E}{F_y}}$ Not applicable	$0.64\sqrt{E / F_y}$ $1.49\sqrt{E / F_y}$
CHANNEL, DOUBLE 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 $0.75\sqrt{E / F_y}$
ANGLE	b / t	$0.3\sqrt{\frac{E}{F_y}}$	$0.45\sqrt{E / F_y}$
DOUBLE-ANGLE (Separated)	b / t	$0.3\sqrt{\frac{E}{F_y}}$	$0.45\sqrt{E / F_y}$

Table III-3

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

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.

Special Seismic Provisions of Member Design

When using the AISC-ASD01 option, the following Framing Systems are recognized (ANSI/AISC 341 SEISMIC 9, 10, 11, 12, 13, 14, 15):

- Ordinary Moment Frame (OMF)
- Intermediate Moment Frame (IMF)
- Special Moment Frame (SMF)
- Ordinary Concentrically Braced Frame (OCBF)
- Special Concentrically Braced Frame (SCBF)
- Eccentrically Braced Frame (EBF)
- Special Truss Moment Frame (STMF)

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

The special seismic requirements checked by the program for member design are dependent on the type of framing used and are described below for each type of framing. Thus special provisions for buildings are only applied if the building frame is classified as seismic design category (SDC) D or E. (ANSI/AISC 341 SEISMIC 1). No special requirement is checked for frames with seismic design category A, B, or C.

Ordinary Moment Frames (OMF)

For this framing system, the following additional requirements are checked and reported (ANSI/AISC 341 SEISMIC 11):

- When $\frac{P_u}{P_n}$ in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the following Special Seismic Load Combinations (ANSI/AISC 341 SEISMIC 8.3, 4.1, ASCE 9.5.2.7.1, 2.3).

$$\left(0.9 \quad 0.2S_{DS}\right)DL \pm \quad {}_{0}EL$$

$$\left(1.2 + 0.2S_{DS}\right)DL + 1.0LL \pm \quad {}_{0}EL$$

Intermediate Moment Frames (IMF)

For this framing system, the following additional requirements are checked and reported (ANSI/AISC 341 SEISMIC 10):

- When $\frac{P_u}{P_n}$ in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the following Special Seismic Load Combinations (ANSI/AISC 341 SEISMIC 8.3, 4.1, ASCE 9.5.2.7.1, 2.3.2).

$$\left(0.9 \quad 0.2SDS\right)DL \pm \quad {}_{0}EL$$

$$\left(1.2 + 0.2SDS\right)DL + 1.0LL \pm \quad {}_{0}EL$$

Special Moment Frames (SMF)

For this framing system, the following additional requirements are checked or reported (ANSI/AISC 341 SEISMIC 9):

- When $\frac{P_u}{P_n}$ in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the following Special Seismic Load Combinations (AISC SEISMIC 8.3, 4.1, ASCE 9.5.2.7.1, 2.3.2).

$$\left(0.9 \quad 0.2S_{DS}\right)DL \pm \quad {}_{0}EL$$

$$\left(1.2 + 0.2S_{DS}\right)DL + 1.0LL \pm \quad {}_{0}EL$$

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

Table III-4

Reduction Factor for Unstiffened Slender Elements, Q_s

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

Table III-5
Effective Width for Stiffened Sections

- The I-, Channel-, and Double-Channel Shaped beams and columns are additionally checked for compactness criteria as described in Table VI-1 (AISC SEISMIC 9.4, 8.2, Table I-8-1). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the, the program issues an error message.
- The program checks the laterally unsupported length of beams to be less than $0.08\left(E/F_y\right)\quad_y$. If this criteria is not satisfied, the program issues an error message.(ANSI/AISC 341 SEISMIC 9.8)
- The program checks the slenderness ratio, L®, for columns to be less than 60 (ANSI/AISC 341 SEISMIC 9.7.b(2)). If the criteria is not satisfied, the program issues an error message.

Ordinary Concentrically Braced Frames (OCBF)

For this framing system, the following additional requirements are checked or reported (ANSI/AISC 341 SEISMIC 14):

- The columns and beams (NOT braces) are designed for the following special amplified seismic load combinations (AISC/ANSI 341 SEISMIC 14.2, ASCI 9.5.2.7.1, 2.3.2.1, 2.3.2).

$$\left(0.9 \quad 0.2S_{DS}\right)DL \pm \quad {}_0 EL$$
$$\left(1.2 + 0.2S_{DS}\right)DL + 1.0LL \pm \quad {}_0 EL$$

- The maximum $\frac{Kl}{r}$ ratio of the braces for V or inverted-V configurations is checked not to exceed $4.23\sqrt{\frac{E}{F_y}}$ (ANSI/AISC 341 SEISMIC 14.2). If this criteria is not met, an error message is reported in the output.

Special Concentrically Braced Frames (SCBF)

For this framing system, the following additional requirements are checked or reported (ANSI/AISC 341 SEISMIC 13):

- When $\frac{P_u}{P_n}$ in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the following Special Seismic Load Combinations (ANSI/AISC 341 SEISMIC 8.34.1, ASCE 9.5.2.7.1, 2.3.2).

$$\left(0.9 \quad 0.2S_{DS}\right)DL \pm \quad {}_{0}EL$$

$$\left(1.2 + 0.2S_{DS}\right)DL + 1.0LL \pm \quad {}_{0}EL$$

- All beam, columns and brace members are checked to be Compact according to Table V-2(ANSI/AISC 341 SEISMIC 13.5, 13.2d, 8.2, Table I-8-1). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the program issues an error message.

  This special criteria is only checked for I, Channel, Double-Channel, Angle, Double-Angle, Box and Pipe sections.

- The compressive strength for braces is taken as $\quad {}_{c}P_{n}$.

  $$P_{u} \qquad {}_{c}P_{n} \qquad \text{(ANSI/AISC 341 SEISMIC 13.26)}$$

- The maximum $K\,l/r$ ratio of the braces is checked not to exceed $5.87\sqrt{\dfrac{F_y}{E}}$. If this check is not met, the program issues an error message.

  Note: Beams intersected by Chevron (V or inverted-V) braces are NOT currently checked to have a strength to support loads for the following two conditions (ANSI/AISC 341 SEISMIC 13.4a):

a A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads form load combinations stipulated by the code, assuming the bracings are not present, and

b A beam that is intersected by braces shall be designed to resist the effects of load combinations stipulated by the code, except that a load $\quad_{b}$ shall be substituted for the term E. $\quad_{b}$ is given by the difference of $R_y F_y A$ for the tension brace and 0.3 $\quad {}_{c}P_{n}$ for the compression brace.

Users need to check for this requirement independently.

Eccentrically Braced Frames (EBF)

For this framing system, the program looks for and recognizes the eccentrically braced frame configurations shown in Figure VI-II. The following additional requirements are checked or reported for the beams, columns and braces associated with these configurations (ANSI/AISC 341 SEISMIC 15).

- When $\frac{P_u}{P_n}$ in columns due to prescribed loading combinations without consideration of amplified seismic load is greater than 0.4, the axial compressive and tensile strengths are checked in absence of any applied moment and shear for the following Special Seismic Load Combinations (ANSI/AISC 341 SEISMIC 8.3, 4.1, ASCE 9.5.2.7.1, 2.3.2).

  $$\left(0.9 \quad 0.2S_{DS}\right)DL \pm \quad {}_0 EL$$

  $$\left(1.2 + 0.2S_{DS}\right)DL + 1.0LL \pm \quad {}_0 EL$$

- The I-shaped, Channel-shaped, and Double-Channel Shaped beams are additionally checked for compactness criteria as described in Table VI-III (ANSI/AISC 341 SEISMIC 15.2, 8.2, Table I-8-1). If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications. If this criteria is not satisfied the user must modify the program issues an error message.

- The link beam yield strength, F_y, is checked not to exceed the following (AISC SEISMIC 15.2):

  $$F_y \quad 50 \text{ ksi} \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

  If the check is not satisfied, the program issue an error message.

- The shear strength for link beams is taken as follows (AISC SEISMIC 15.2):

  $$V_u \quad {}_v V_n\,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

  where,

  $$V_n = \min\left(\quad V_{pa}, \quad 2M_{pa}/e\right), \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

  $$V_{pa} = V_p \sqrt{1 \quad \left(\frac{P_u}{P_y}\right)^2}\,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.1)}$$

  $$M_{pa} = 1.18\, M_p \quad 1 \quad \frac{P_u}{P_y}\,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

$$V_p = 0.6\, F_y (d \quad 2t_f)\, t_w \,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

$$M_p = Z\, F_y \,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

$$= \quad {}_v \quad \text{(default is 0.9)} \,, \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

$$P_y = A_g F_y \,. \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

- If $P_u > 0.15\, A_g F_y$, the link beam length, *e*, is checked not to exceed the following (ANSI/AISC 341 SEISMIC 15.2):

$$e \quad \begin{cases} 1.15 \quad 0.5 \quad \dfrac{A_w}{A_g} \quad 1.6 \dfrac{M_p}{V_p} & if \quad \dfrac{A_w}{A_g} \quad 0.3 \,, \\ 1.6 \dfrac{M_p}{V_p} & if \quad \dfrac{A_w}{A_g} < 0.3 \,, \end{cases}$$

(ANSI/AISC 341 SEISMIC 15.2)

where,

$$A_w = (d \quad 2t_f)\, t_w \,, \text{ and} \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

$$= P_u / V_u \,. \qquad \text{(ANSI/AISC 341 SEISMIC 15.2)}$$

If the check is not satisfied, the program reports an error message.

- The link beam rotation, , of the individual bay relative to the rest of the beam is calculated as the story drift ${}_M$ times bay length divided by the total lengths of link beams in the bay. The link beam rotation, , is checked as follows (ANSI/AISC 341 SEISMIC 15.2).

0.08 radian , where link beam clear length, $e \quad 1.6\, M_s / V_s$,

0.03 radian , where link beam clear length, $e \quad 2.6\, M_s / V_s$, and

value interpolated between 0.08 and 0.02 as the link beam clear length varies from $1.6\, M_s / V_s$ to $2.6\, M_s / V_s$.

- The beam strength outside the link is checked to be at least 1.1 times the beam forces corresponding to the controlling link beam shear strength (ANSI/AISC 341 SEISMIC 15.6). The controlling link beam nominal shear strength is taken as follows:

$$\min\left(V_{pa},\, 2M_{pa} / e\right), \qquad \text{(ANSI/AISC 341 SEISMIC 15.6, 15.2)}$$

The values of V_{pa} and M_{pa} are calculated following the procedure described above (ANSI/AISC 341SEISMIC 15.2). The correspondence between brace force and link beam force is obtained from the associated load cases, whichever has the highest link beam force of interest.

All braces are checked to be at least compact per regular ANSI/AISC 341code (ANSI/AISC 341 SEISMIC 15.6). If this criteria is not satisfied, the program issues an error message.

The brace strength is checked for $1.25R_y$ times the brace forces corresponding to the controlling link beam nominal shear strength (ANSI/AISC 341 SEISMIC 15.6). The controlling link beam nominal shear strength and the corresponding forces are obtained by the process described earlier.

The I-, Channel-, and Double-Channel- shaped column sections are checked to be at least compact per regular ANSI/AISC 341 code (ANSI/AISC 341 SEISMIC 8.2, Table I-8-1, LRFD B.5.1). If this criterion is not satisfied, the program issues an error message.

- The column strength is checked for $1.1R_y$ times the column forces corresponding to the controlling link beam nominal shear strength (ANSI/AISC 341 SEISMIC 15.8). The controlling link beam nominal shear strength and the corresponding forces are obtained by the process described above.

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

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.

$f_a = P/A$ (ASD A-B5.2d)

$f_{b33} = M_{33}/S_{eff,33}$ (ASD A-B5.2d)

$f_{b22} = M_{22}/S_{eff,22}$ (ASD A-B5.2d)

$f_{v2} = V_2/A_{v2}$ (ASD A-B5.2d)

$f_{v3} = V_3/A_{v3}$ (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 design over write form, these values ***will override the above mentioned calculated values for those elements*** *as defined in the following subsections. The specified allowable stresses should be based on the principal axes of bending.*

Allowable Stress in Tension

The allowable axial tensile stress value F_a is assumed to be $0.60\, F_y$.

$F_a = 0.6 F_y$ (ASD D1, ASD SAM 2)

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

Allowable Stress in Compression

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

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

Flexural Buckling

The allowable axial compressive stress value, F_a, depends on the slenderness ratio Kl/r based on gross section properties and a corresponding critical value, C_c, where

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

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

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

For Compact or Noncompact sections F_a is evaluated as follows:

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

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

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

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

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

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

where,

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

For slender sections, if Kl/r is greater than 200, then the calculated value of F_a is taken not to exceed its value calculated by using the equation ASD A-B5-12 (ASD B7, E1).

For slender Pipe sections F_a is evaluated as follows:

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

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

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

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

Q_a = reduction factor for stiffened slender elements. (ASD A-B5.2.c)

The Q_s factors for slender sections are calculated as described in Table III-4 (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:

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

b_e for unstiffened elements is taken equal to b, and b_e for stiffened elements is taken equal to or less than b as given in Table III-5 (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):

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

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

where,

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

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

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,

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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,

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

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:

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

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

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

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

A_w = Area of web, in^2,

A_f = Area of compression flange, in^2,

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

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.

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

Q_s is defined earlier.

Minor Axis of Bending

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

For Compact sections:

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

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

For Noncompact and Slender sections:

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

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

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:

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

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

Minor Axis of Bending

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

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

T-sections and Double angles

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

$$F_b = 0.60 \; F_y \; .$$

Box Sections and Rectangular Tubes

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

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

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 $\dfrac{1200 \; b}{F_y}$ 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):

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

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

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

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

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,

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

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.

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

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

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

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

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

where,

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = slenderness reduction factor for local buckling. (ASD A-B5-2, SAM 4)

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

Minor Axis of Bending

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

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

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

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

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

General Sections

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

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

Allowable Stress in Shear

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

Major Axis of Bending

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

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

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

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

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

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

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

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

- 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.

Shear Stresses

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

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

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

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.

Joint Design

When using AISC-ASD01 design code, the structural joints are checked and/or designed for the following:

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

Design of Continuity Plates

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

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

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

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

The program first evaluates the need for continuity plates. Continuity plates will be required if **any** of the following four conditions are not satisfied:

- The column flange design strength in bending must be larger than the beam flange force, i.e.,

$$R_n = (0.9)6.25\, t_{fc}^2\, F_{yc} \quad P_{bf} \text{ if not at top story} \qquad \text{(LRFD K1-1)}$$

$$R_n = (0.5)(0.9)6.25\, t_{fc}^2\, F_{yc} \quad P_{bc} \text{ if at top story} \qquad \text{(LRFD K1-2)}$$

- The design strength of the column web against local yielding at the toe of the fillet must be larger than the beam flange force, i.e.,

$$R_n = (1.0)\,(5.0\, k_c + t_{fb})\, F_{yc} t_{wc} \quad P_{bf}, \text{ if not at top story} \qquad \text{(LRFD K1-2)}$$

$$R_n = (1.0)(2.5 k_c + t_{fb}) F_{yc} t_{wc} \quad P_{bf}, \text{ if at top story} \qquad \text{(LRFD K1-3)}$$

- The design strength of the column web against crippling must be larger than the beam flange force, i.e.,

$$R_n = (0.75)\ 0.80\, t_{wc}^2 \left(1 + 3 \left(\frac{t_{fb}}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \sqrt{E F_{yc} \frac{t_{fc}}{t_{wc}}} \quad P_{bf},$$

if not at top story (LRFD K1-4)

$$R_n = (0.75)(0.40)\, t_{wc}^2 \left(1 + 3 \left(\frac{t_{fb}}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \sqrt{E F_{yc} \frac{t_{fc}}{t_{wc}}} \quad P_{bf},$$

it at top story (LRFD K1-5a)

- The design compressive strength of the column web against buckling must be larger than the beam flange force, i.e.,

$$R_n = (0.9) \frac{24\, t_{wc}^3 \sqrt{E F_{yc}}}{d_c} \quad P_{bf}, \text{ if not at top story} \qquad \text{(LRFD K1-8)}$$

$$R_n = (0.9) \frac{12\, t_{wc}^3 \sqrt{E F_{yc}}}{d_c} \quad P_{bf}, \text{ if at top story} \qquad \text{(LRFD K1.9, E2)}$$

If any of the conditions above are not met the program calculates the required continuity plate area as,

$$A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{yc})} \quad 25t_{wc}^{2}, \text{ if not at top story} \qquad \text{(LRFD K1.9, E2)}$$

$$A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{yc})} \quad 12t_{wc}^{2}, \text{ if at top story} \qquad \text{(LRFD K1.9, E2)}$$

If A_{cp} 0, no continuity plates are required.

The formula above assumes the continuity plate plus a width of web equal to $12t_{wc}$ or $25t_{wc}$ act as a compression member to resist the applied load (LRFD K1.9). The formula also assumes $= 0.85$ and $F_{cr} = 0.9F_{yc}$. This corresponds to an assumption of $_{c} = 0.5$ in the column formulas (LRFD E2-2). The user should choose the continuity plate cross-section such that this is satisfied. As an example when using $F_{yc} = 50$ ksi and assuming the effective length of the stiffener as a column to be $0.75h$ (LRFD K1.9) the required minimum radius of gyration of the stiffener cross-section would be $r = 0.02h$ to obtain $_{c} = 0.5$ (LRFD E2-4).

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

- The minimum thickness of the stiffeners is taken in th program as follows:

$$t_{cp}^{min} = \max \; 0.5 \; t_{fb}, 1.79\frac{\sqrt{F_{yc}}}{E} \; b_{fb} \qquad \text{(LRFD K1.9.2)}$$

- The minimum width of the continuity plate on each side plus ½ the thickness of the column web shall not be less than 1/3 of the beam flange width, or

$$b_{cp}^{min} = 2 \; \frac{b_{fp}}{3} \quad \frac{t_{wc}}{2} \div \qquad \text{(LRFD K1.9.1)}$$

- So that the minimum area is given by:

$$A_{cp}^{min} = t_{cp}^{min} \; b_{cp}^{min} \qquad \text{(LRFD K1.9.1)}$$

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

In the equations above,

A_{cp}	=	Required continuity plate area
F_{yc}	=	Yield stress of the column and continuity plate material
d_b	=	Beam depth
d_c	=	Column depth
h	=	Clear distance between flanges of column less fillets for rolled shapes
k_c	=	Distance between outer face of the column flange and web toe of its fillet.
M_u	=	Factored beam moment
P_{bf}	=	Beam flange force, assumed as $M_u / \left(d_b \quad t_{fb} \right)$
R_n	=	Nominal strength
t_{fb}	=	Beam flange thickness
t_{fc}	=	Column flange thickness
t_{wc}	=	Column web thickness
	=	Resistance factor

The special seismic requirements additionally checked by the program are dependent on the type of framing used and the Seismic Design Category. If the structure is identified as Seismic Design Category D or E, the special seismic requirements are satisfied (ANSI/AISC 341 SEISMIC 1). No special check is made if the Seismic Design Category is A, B, or C.

Continuity plate requirements for seismic design are evaluated for moment frames (OMF, IMF, SMF) only. No checks are done for braced frames (LCBF, SCBF, and EBF).

- For OMF the continuity plates are checked and designed for a beam flange force, $P_{bf} = M_{pb} / \left(d_b \quad t_{fb} \right)$.

$$P_{bf} = M_{pb} / \left(d_b \quad t_{fb} \right) \qquad \text{(ANSI/AISC 341 SEISMIC 11.5)}$$

- For SMF and IMF, the continuity plates are checked and designed for a beam flange force, $P_{bf} = R_y F_y b_{fb} t_{fb}$.

$$P_{bf} = R_y F_y b_{fb} t_{fb}$$

Note that the code insists on designing continuity pate to match with tested connection (ANSI/AISC 341 SEUISMIC 9.5, 10.5)

Design of Doubler Plates

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

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

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

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

$$V_p = {}^{n_b}_{n=1} \frac{M_{bn} \cos \quad _n}{d_n - t_{fn}} \qquad V_c$$

The nominal panel shear strength is given by

$$R_v = 0.6 F_y d_c t_r, \quad \text{for } P_u \quad 0.4 P_y \text{ or if } P_u \text{ is tensile, and} \qquad \text{(LRFD K1-9)}$$

$$R_v = 0.6 F_y d_c t_r \;\; 1.4 \quad \frac{P_u}{P_y} \;, \quad \text{for } P_u > 0.4 P_y \;. \qquad \text{(LRFD K1-10)}$$

By using $V_p = \ R_v$, with $\ = 0.9$ (by default), the required column web thickness t_r can be found.

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

$$t_{dp} = t_r \quad t_w \;, \qquad \text{(LRFD F2-1)}$$

where,

F_y = Column and doubler plate yield stress
t_r = Required column web thickness
t_{fn} = Flange thickness of n-th beam connecting to the column
t_{dp} = Required doubler plate thickness
t_{fc} = Column Flange thickness

t_w = Column web thickness
h = $d_c \quad 2t_{fc}$ if welded, $d_c \quad 2k_c$ if rolled
V_p = Panel zone shear
V_c = Column shear in column above
F_y = Beam flange forces
n_b = Number of beams connecting to column
d_n = Depth of n-th beam connecting to column
${}_n$ = Angle between n-th beam and column major direction
d_c = Depth of column clear of fillets, equals $d \quad 2k$
M_{bn} = Calculated factored beam moment from
the corresponding loading combination
R_v = Nominal shear strength of panel
P_u = Column factored axial load
P_y = Column axial yield strength, $F_y A$

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

The special seismic requirements additionally checked by the program are dependent on the type of framing used and the Seismic Design Category. If the structure is identified as Seismic Design Category D or E, the special seismic requirements are satisfied (ANSI/AISC 341 SEISMIC 1). No special check is made if the Seismic Design Category is A, B, or C.

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

- For Special Moment-Resisting Frames, the panel zone doubler plate requirements that are reported will develop at least the beam moments equal to of the plastic moment capacity of the beam or beam moments due to specified load combinations involving seismic load (ANSI/AISC 341 SEISMIC 9.3a).
- For seismic design, V_p is calculated using the same equation as given above, except that M_{pb} is taken as $R_y F_y Z_{33}$.

The capacity of the panel zone in resisting this shear is taken as (ANSI/AISC 341 SEISMIC 9-5):

$$\phi_v V_n = 0.60\, \phi_v F_y\, d_c\, t_p \left(1 + \frac{3\, b_{cf}\, t_{cf}^2}{d_b\, d_c\, t_p}\right)$$

for $P_u \le 0.75 P_y$ (ANSI/AISC 341 SEISMIC 9-5)

$$\phi_v V_n = 0.6\, \phi_v F_y d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p}\right)\left(1.9 - 1.2 \frac{P_u}{P_y}\right)$$

for $P_u > 0.75 P_y$ (ANSI/AISC 341 SEISMIC 9.3a, LRFD K1-12)

giving the required panel zone thickness as

$$t_p = \frac{V_p}{0.6\, \phi_v F_y\, d_c} - \frac{3\, b_{cf} t_{cf}^2}{d_b\, d_c}, \text{ if } P_u \le 0.75 P_y$$

$$t_p = \frac{V_p}{0.6\, \phi_v F_y d_c \left(1.9 - 1.2 \left(\frac{P_u}{P_y}\right)\right)} - \frac{3 b_{cf} t_{cf}^2}{d_b d_c}, \text{ if } P_u > 0.75 P_y \quad \text{(by default)},$$

(ANSI/AISC 341 SEISMIC 9.3a)

and the required doubler plate thickness as

$$t_{dp} = t_p - t_{wc}$$

where,

ϕ_v	=	0.90 by default,
b_{cf}	=	width of column flange,
t_{cf}	=	thickness of column flange,
t_p	=	required column web thickness,
h	=	$d_c - 2t_{fc}$ if welded, $d_c - 2k_c$ if rolled, and
d_b	=	depth of deepest beam framing into the major direction of the column.
P_y	=	$F_y A$
P_u	=	Axial force in column

- For Special Moment-Resisting Frames, the panel zone column web thickness requirement the program checks the following:

$$t \geq \frac{(d_c - 2t_{fc}) + (d_b - 2t_{fb})}{90} \qquad \text{(ANSI/AISC 341 SEISMIC 9.36)}$$

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

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

Weak Beam Strong Column Measure

Only for Special Moment Frames with Seismic Design Category D and E, the code requires that the sum of column flexure strengths at a joint should be more than the sum of beam flexure strengths (ANSI/AISC 341 SEISMIC 1, 9.6). The column flexure strength should reflect the presence of axial force present in the column. The beam flexural strength should reflect potential increase in capacity for strain hardening to facilitate the review of the strong column weak beam criterion, the program will report a beam/column plastic moment capacity ratio for every joint in the structure.

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

$$R_{maj} = \frac{\sum_{n=1}^{n_b} M^*_{pbn} \cos \theta_n}{M^*_{pcax} + M^*_{pcby}} \qquad \text{(ANSI/AISC 341 SEISMIC 9.6)}$$

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

$$R_{min} = \frac{\sum_{n=1}^{n_b} M_{pbn} \sin \theta_n}{M_{pcay} + M_{pcby}}, \qquad \text{(ANSI/AISC 341 SEISMIC 9.6)}$$

where,

$R_{maj,min}$ = Plastic moment capacity ratios, in the major and minor directions of the column, respectively

M^*_{pbn} = Plastic moment capacity of n-th beam connecting to column

${}_n$ = Angle between the n-th beam and the column major direction

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

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

n_b = Number of beams connecting to the column

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

$$M^*_{pc} = Z_c\left(F_{yc} - \left|P_{uc}/A_g\right|\right), \qquad \text{(ANSI/AISC 341 SEISMIC 9.6)}$$

The plastic moment capacities of the beams are amplified for potential increase in capacity for strain hardening as,

$$M^*_{pb} = 1.1 R_y F_{yb} Z_b f_{mv},$$

where,

Z_b = Plastic modulus of beam,

Z_c = Plastic modulus of column,

F_{yb} = Yield stress of beam material,

F_{yc} = Yield stress of column material,

P_{uc} = Axial compression force in column for the load combination under consideration,

A_{gc} = Gross area of column,

f_{mv} = The moment amplification factor. It is taken as the ratio of beam moment at the centerline of column to the moment the column face. This factor takes care of the M_v of the code (ANSI/AISC 341 SEISMIC 9.6). F_{mv} is taken as follows:

$$1 + \frac{d_c}{L_b},$$

d_c = Depth of column section, and

L_b = Clear span length of the beam.

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

Evaluation of Beam Connection Shears

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

For special seismic design, the beam connection shears are not taken less than the following special values for different types of framing. The special seismic requirements additionally checked by the program are dependent on the type of framing used and the Seismic Design Category. If the structure is identified as Seismic Design Category D or E, the special seismic requirements are satisfied (ANSI/AISC 341 SEISMIC 1). No special check is made if the Seismic Design Category is A, B, or C.

- For special moment frames, the beam connection shear is taken as the maximum of those from regular load combinations and those required for the development of full plastic moment capacity of the beams. The connection shear for the development of the full plastic moment capacity of beam is as follows:

$$V_u = \frac{C\,M_{pb}}{L} + 1.2\,V_{DL} + 0.5\,V_{LL} \qquad \text{(ANSI/AISC 341 SEISMIC 9.2.a(3))}$$

where

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

shears at the corresponding beam ends from the live load only.

- For Intermediate Moment Frames and Ordinary Moment Frames, the beam connection shear is taken as the maximum of those from regular load combinations and those from special seismic consideration. the beam connection shear from special seismic consideration is taken as the minimum of those required for the development of full plastic moment capacity of the beam and those required for amplified seismic load and those required (ANSI/AISC 341 SEISMIC 10.2, 11.2). The connection shear for the development of the full plastic moment capacity of beam is as follows:

$$V_u = \frac{CM_{pb}}{L} + 1.2\,V_{DL} + 0.5V_{LL} \qquad \text{(ANSI/AISC 341 SEISMIC 10.2, 11.2)}$$

All parameters in the above equation have been described earlier.

The load combinations for amplified seismic loads are (ANSI/AISC 341 SEISMIC 8.3, 4.1, ASCI 9.5.2.7.1, 2.3.2):

$$\left(0.9 + 0.2S_{DS}\right)DL \pm \ {}_0EL$$

$$\left(1.2 + 0.2S_{DS})DL + 1.0LL \pm \ {}_0EL\right)$$

- For OCBF, the beam connection shear is taken as the maximum of those from regular load combinations and those from amplified seismic load combinations (ANSI/AISC 341 SEISMIC 14.2).
- For SCBF, the beam connection shear is taken as the maximum of those from regular load combinations and those from amplified seismic load combination (ANSI/AISC 341 SEISMIC 13.4a(2)).

Note: Beams intersected by Chevron (V or inverted-V) braces are NOT currently checked to have a strength to support loads for the following two conditions (ANSI/AISC 341 SEISMIC 13.4a):

a A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads form load combinations stipulated by the code, assuming the bracings are not present, and

b A beam that is intersected by braces shall be designed to resist the effects of load combinations stipulated by the code, except that a load $_b$ shall be substituted for the term E. $_b$ is given by the difference of $R_y F_y A$ for the tension brace and 0.3 $_c P_n$ for the compression brace.

Users need to check for this requirement independently.

- For EBF, the beam connection shear is taken as the beam connection shear is taken as the maximum of those from regular load combinations and those from special seismic considerations. The beam connection shear from special seismic consideration is taken as the minimum of those required for yielding of link beam and those required for amplified seismic load (ANSI/AISC 341 SEISMIC 15.1, 15.4, 15.6). The load factor for the seismic component of loads in the combination is calculated to achieve forces related to yielding of link beam. For connection shear determination the forces are further amplified by $1.1 R_y$ (ANSI/AISC 341 SEISMIC 15.6(2)). The load combinations for Amplified Seismic Loads are given earlier.

Evaluation of Brace Connection Forces

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

For special seismic design, the brace connection forces are not taken less than the following special values for different types of framing. The special seismic requirements additionally checked by the program are dependent on the type of framing used and the Seismic Design Category. If the structure is identified as Seismic Design Category D or E, the special seismic requirements are satisfied (ANSI/AISC 341 SEISMIC 1). No special check is made if the Seismic Design Category is A, B, or C.

Brace axial forces for seismic design are evaluated for braced frames (OCBF, SCBF, EBF) only. No special checks are done for moment frames (OMF, IMF, SMF).

- For OBF, the bracing connection force is reported at least the expected tensile strength of the brace ($R_y F_y A_g$) (ANSI/AISC 341 SEISMIC 14.2):
- For SCBF, the bracing connection force is reported at least as the expected the tensile strength of the brace ($R_y F_y A_g$) (ANSI/AISC 341 SEISMIC 13.3a).

For EBF, the brace connection force is taken as the maximum of those from regular load combinations and those from special seismic consideration. The brace connection force from special seismic consideration is taken as the minimum of those required for yielding of link beam and those required for Amplified Seismic Load (ANSI/AISC 341 SEISMIC 15.1, 15.4, 15.6). The load factor for the seismic component of loads in the combination is calculated to achieve forces related to yielding of Link beam. for connection force determination, the forces are further amplified by $1.25R_y$ (ANSI/AISC 341 SEISMIC 15.6). The load combinations for Amplified Seismic Load are given earlier in this document.

Chapter IV

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 III-1.

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

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

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

A	=	Cross-sectional area, in^2
A_e	=	Effective cross-sectional area for slender sections, in^2
A_f	=	Area of flange , in^2
A_g	=	Gross cross-sectional area, in^2
A_{v2}, A_{v3}	=	Major and minor shear areas, in^2
A_w	=	Web shear area, dt_w, in^2
C_b	=	Bending Coefficient
C_m	=	Moment Coefficient
C_w	=	Warping constant, in^6
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
F_a	=	Allowable axial stress, ksi
F_b	=	Allowable bending stress, ksi
F_{b33}, F_{b22}	=	Allowable major and minor bending stresses, ksi
F_{cr}	=	Critical compressive stress, ksi
F_{e33}	=	$\dfrac{12\ \ ^{2}E}{23\left(K_{33}l_{33}/r_{33}\right)^{2}}$
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 IV-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, $b_f \quad 2t_w$ 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
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 $(d \quad 2t_f)$, 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, $\dfrac{4.05}{\left[h/t_w\right]^{0.46}}$ if $h/t_w > 70$, 1 if $h/t_w \quad 70$.
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 IV-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):

$$DL \qquad \text{(ASD A4.1)}$$
$$DL + LL \qquad \text{(ASD A4.1)}$$

$$DL \pm WL \qquad \text{(ASD A4.1)}$$
$$DL + LL \pm WL \qquad \text{(ASD A4.1)}$$

$$DL \pm EL \qquad \text{(ASD A4.1)}$$
$$DL + LL \pm EL \qquad \text{(ASD A4.1)}$$

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.

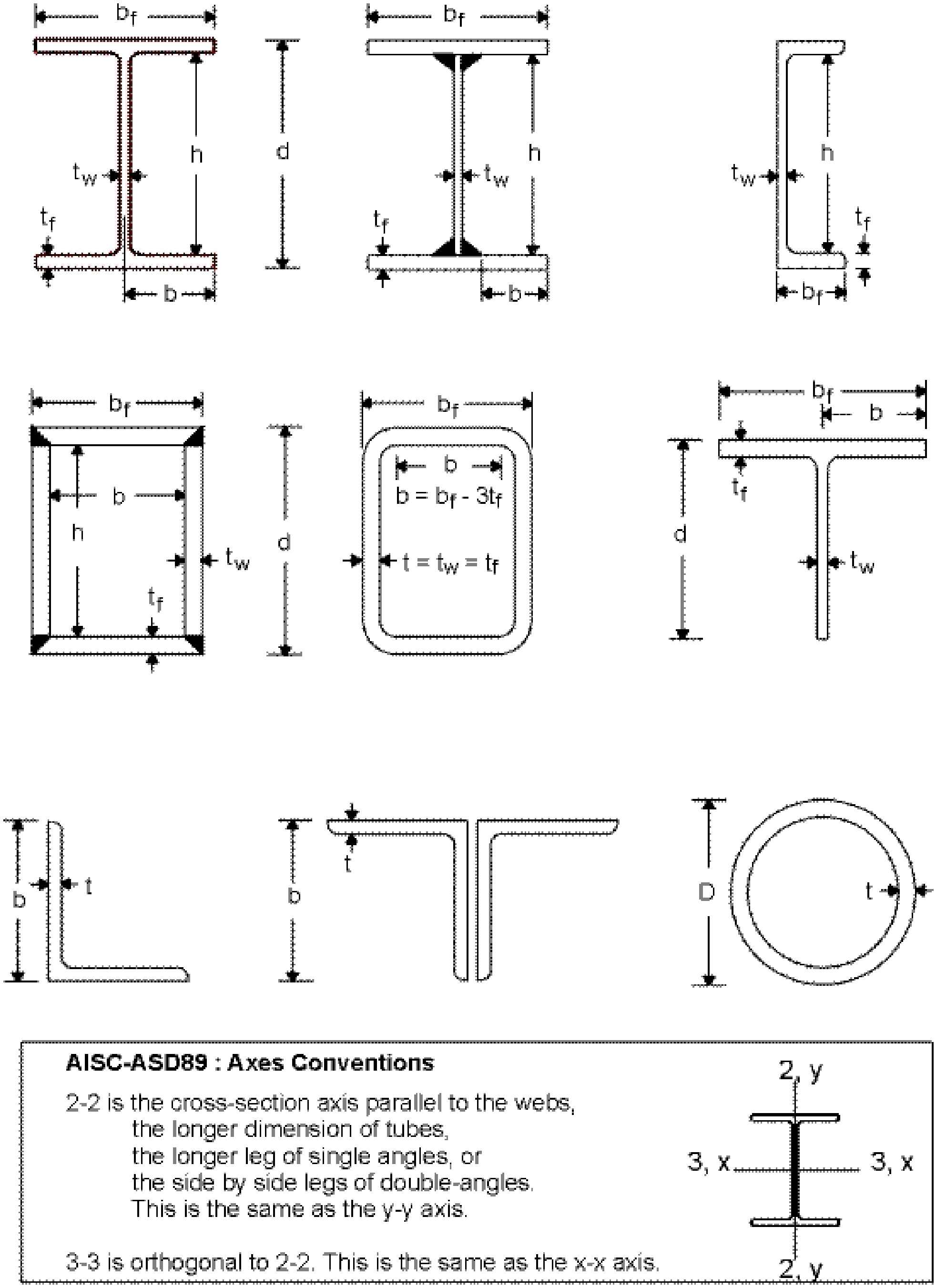

Figure IV-1
AISC-ASD Definition of Geometric Properties

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 $f_a / F_y \quad 0.16$ $\frac{640}{\sqrt{F_y}}(1 \quad 3.74\frac{f_a}{F_y})$, 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 $b_f / d_w \quad 0.25$, $t_f / t_w \quad 3.0$ If rolled $b_f / d_w \quad 0.5$, $t_f / t_w \quad 2.0$

Table IV-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 IV-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.:

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

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.

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

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 $0.60\,F_y$.

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

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

Allowable Stress in Compression

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

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

Flexural Buckling

The allowable axial compressive stress value, F_a, depends on the slenderness ratio Kl/r based on gross section properties and a corresponding critical value, C_c, where

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

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

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:

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

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

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:

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

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

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-4 (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-5 (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):

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

Section Type	Reduction Factor for Unstiffened Slender Elements (Q_s)	Equation Reference
I-SHAPE	$Q_s = \begin{cases} 1.0 & \textit{if} \quad b_f/2t_f \quad 95/\sqrt{F_y/k_c}, \\ 1.293 \quad 0.00309[b_f/2t_f]\sqrt{F_y/k_c} & \textit{if} \quad 95/\sqrt{F_y/k_c} < b_f/2t_f < 195/\sqrt{F_y/k_c}, \\ 26{,}200\, k_c/\{[b_f/2t_f]^2 F_y\} & \textit{if} \quad b_f/2t_f \quad 195/\sqrt{F_y/k_c}. \end{cases}$	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. $Q_s \quad \begin{cases} 1.0, & \textit{if} \quad d/t_w \quad 127/\sqrt{F_y}, \\ 1.908 \quad 0.00715[d/t_w]\sqrt{F_y}, & \textit{if} \quad 127/\sqrt{F_y} < d/t_w < 176/\sqrt{F_y}, \\ 20{,}000/\{[d/t_w]^2 F_y\}, & \textit{if} \quad d/t_w \quad 176/\sqrt{F_y}. \end{cases}$	ASD A-B5-3, ASD A-B5-4, ASD A-B5-5, ASD A-B5-6
DOUBLE-ANGLE	$Q_s = \begin{cases} 1.0, & \textit{if} \quad b/t \quad 76/\sqrt{F_y}, \\ 1.340 \quad 0.00447[b/t]\sqrt{F_y}, & \textit{if} \quad 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500/\{[b/t]^2 F_y\}, & \textit{if} \quad b/t \quad 155/\sqrt{F_y}. \end{cases}$	ASD A-B5-1, ASD A-B5-2, SAM 4-3
ANGLE	$Q_s = \begin{cases} 1.0, & \textit{if} \quad b/t \quad 76/\sqrt{F_y}, \\ 1.340 \quad 0.00447[b/t]\sqrt{F_y}, & \textit{if} \quad 76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}, \\ 15{,}500/\{[b/t]^2 F_y\}, & \textit{if} \quad b/t \quad 155/\sqrt{F_y}. \end{cases}$	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
RECTANGULAR	$Q_s = 1$	ASD A-B5.2c
GENERAL	$Q_s = 1$	ASD A-B5.2c

Table IV-3

Reduction Factor for Unstiffened Slender Elements, Q_s

Section Type	Effective Width for Stiffened Sections	Equation Reference
I-SHAPE	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \quad \frac{195.74}{\sqrt{f}}, \\ \frac{253\, t_w}{\sqrt{f}} \;\; 1 \quad \frac{44.3}{(h/t_w)\sqrt{f}}, & \text{if } \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	ASD A-B5-8
BOX	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \quad \frac{195.74}{\sqrt{f}}, \\ \frac{253\, t_w}{\sqrt{f}} \;\; 1 \quad \frac{44.3}{(h/t_w)\sqrt{f}}, & \text{if } \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (compression only, $f = \frac{P}{A_g}$)	ASD A-B5-8
	$b_e = \begin{cases} b, & \text{if } \frac{b}{t_f} \quad \frac{183.74}{\sqrt{f}}, \\ \frac{253\, t_f}{\sqrt{f}} \;\; 1 \quad \frac{50.3}{(h/t_f)\sqrt{f}}, & \text{if } \frac{b}{t} > \frac{183.74}{\sqrt{f}}. \end{cases}$ (compr., flexure, $f = 0.6F_y$)	ASD A-B5-7
CHANNEL	$h_e = \begin{cases} h, & \text{if } \frac{h}{t_w} \quad \frac{195.74}{\sqrt{f}}, \\ \frac{253\, t_w}{\sqrt{f}} \;\; 1 \quad \frac{44.3}{(h/t_w)\sqrt{f}}, & \text{if } \frac{h}{t_w} > \frac{195.74}{\sqrt{f}}. \end{cases}$ (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 IV-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),

$r_0 = \sqrt{x_0^2 + y_0^2 + \dfrac{I_{22} + I_{33}}{A_g}}$ = 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

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

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:

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

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

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

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

$$F_{b33} = 0.60\,F_y\,, \tag{ASD F1-6}$$

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

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

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

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

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

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

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,

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

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:

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

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

For Noncompact and Slender sections:

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

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

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

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

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:

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

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

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

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:

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

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:

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

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:

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

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

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:

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

$F_{b22} = 0.60\, F_y$ (for Noncompact and Slender sections) (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:

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

Pipe Sections

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

$F_b = 0.66\, F_y$ (for Compact sections), and (ASD F3-1)

$F_b = 0.60\, F_y$ (for Noncompact and Slender sections). (ASD F3-3)

Round Bars

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

$F_b = 0.75\, F_y$. (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,

$F_b = 0.75\, F_y$. (ASD F2-1)

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

$F_b = 0.60\, F_y$, And

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

$F_b = 0.75\, F_y$. (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):

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

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

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

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

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,

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

z = coordinate along the major principal axis,

w = coordinate along the minor principal axis, and

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

β_w is a special section property for angles. It is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles (ASD SAM 5.3.2). However, for conservative design in 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.

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

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

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

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

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

where,

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = slenderness reduction factor for local buckling. (ASD A-B5-2, SAM 4)

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

Minor Axis of Bending

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

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

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

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

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

General Sections

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

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

Allowable Stress in Shear

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

Major Axis of Bending

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

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

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

where,

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

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

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

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

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

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 **XIII**

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.

Design of Steel Structures, Part 1.1 : General Rules and Rules for Buildings, ENV 1993-1-1 : 1992, European Committee for Standardization, Brussels, Belgium, 1992.

CISC, 1995

Handbook of Steel Construction, CAN/CSA-S16.1-94, 6th Edition, Canadian Institute of Steel Construction, Willowdale, Ontario, Canada, 1995.

CSI, 2005a

SAP2000 Getting Started, Computers and Structures, Inc., Berkeley, California, 2005.

CSI, 2005b

Welcome to ETABS,, Computers and Structures, Inc., Berkeley, California, 2005.

CSI, 2005c

CSI Analysis Reference Manual, Computers and Structures, Inc., Berkeley, California, 2005.

ICBO, 1997

Uniform Building Code, 1997, International Conference of Building Officials, Whittier, California, 1997.

D. W. White and J. F. Hajjar, 1991

"Application of Second-Order Elastic Analysis in LRFD: Research to Practice," *Engineering Journal, American Institute of Steel Construction, Inc.*, Vol. 28, No. 4, 1991.