COMPUTERS &
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INC.

STEELER®

Stress Check of Steel Frames

A Postprocessor for ETABS®

Version 6.2
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Developed and written in U.S.A.

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

Computers and Structures, Inc.
1995 University Avenue
Berkeley, California 94704 USA
Phone: (510) 845-2177
FAX: (510) 845-4096
e-mail: info@csiberkeley.com

DISCLAIMER

CONSIDERABLE TIME, EFFORT AND EXPENSE HAVE GONE INTO THE DEVELOPMENT AND DOCUMENTATION OF STEELER. THE PROGRAM HAS BEEN THOROUGHLY TESTED AND USED. IN USING THE PROGRAM, 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 THE PROGRAM.

THE PROGRAM IS A VERY PRACTICAL TOOL FOR THE STRESS CHECK OF STEEL FRAME STRUCTURES. PREVIOUS VERSIONS OF THIS PROGRAM HAVE BEEN VERY SUCCESSFULLY USED ON A VARIETY OF BUILDINGS. HOWEVER, THE USER MUST THOROUGHLY READ THE MANUAL AND CLEARLY RECOGNIZE THE ASPECTS OF STEEL DESIGN THAT THE PROGRAM ALGORITHMS DO **NOT** ADDRESS.

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

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Chapter I

Introduction

STEELER is a steel stress check/design post processor for the three-dimensional static and dynamic building analysis computer program ETABS.

The program is intended for the automated code check/design based upon either the American Institute of Steel Construction (AISC) Allowable Stress Design (AISC-ASD89) Specifications [1], or the Uniform Building Code (UBC94) [2], or the AISC Plastic Design Specifications (AISC Plastic89) [1], or the AISC Load and Resistance Factor Design (AISC-LRFD93) Specifications [3], or the Canadian Standards Association Limit States Design of Steel Structures (CISC89) Specifications [4].

Most of the data required by STEELER for the design processing, i. e. material and section properties, member forces and geometry, is recovered directly from the ETABS database. Therefore, the data input typically required by STEELER is very nominal and **if the program defaults are acceptable, no data input is required**.

When using the UBC option the program checks for the additional seismic requirements of the code. The following seismic framing systems are recognized:

- Ordinary Moment Resisting Frames
- Special Moment Resisting Frames

- Braced Frames
- Eccentrically Braced Frames
- Special Concentrically Braced Frames

In the stress check process, maximum stress ratios are calculated based upon user-specified load combinations and/or overriding code specified loading combinations for certain types of framings. All allowable axial and bending stress values are calculated by the program. Tedious calculations associated with evaluating effective length factors for columns in moment frame type structures are automated in the algorithm.

Requirements for continuity plates at the beam to column connections are evaluated.

The program will perform 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 output. Also, maximum axial tension or compression values that are generated in the members are reported.

The ratios of the beam flexural capacities with respect to the column flexural capacities (reduced for axial force effects) associated with the weak beam-strong column aspects of any beam/column intersection, are reported for special moment resisting frames.

Capacity requirements associated with seismic framing systems that require ductility are checked whenever the UBC94 code is requested.

The presentation of the output is in a format that allows the engineer to quickly study the stress conditions that exist in the frame and aids the engineer in taking appropriate remedial measures in the event of member overstress.

The stress conditions can also be viewed graphically through the ETABS and STEELER graphics post processor ETABSOUT.

Changes in structural member section properties are possible at the post processor level to study the effects of member changes without rerunning the ETABS analysis.

English as well as SI and MKS metric units are possible.

Chapter II

Installation and Execution Procedure

This chapter deals with the installation and execution of STEELER on a Windows 95 or Windows NT 4.0 based computer system.

User familiarity with Windows is assumed.

STEELER is an add on steel design postprocessor to the building analysis program ETABS. It is included in the ETABS Plus and ETABS Nonlinear packages.

A. Installing and Testing

The program provided must first be installed on the hard disk. Follow the instructions for installing the ETABS program for this procedure.

Before putting the system into a production mode, the user should test the system by running the sample example provided on the disk. The output files produced should be compared with the corresponding output files that are also provided on this disk.

B. Input Preparation Before Executing STEELER

STEELER is a postprocessor for the ETABS analysis program. Therefore, before running STEELER the user must generate an ETABS input data file and execute ETABS to create the ETABS post processing file.

Say that the ETABS data associated with the structure the user wishes to analyze has been prepared and entered in a data file called EXSTL. A successful execution of ETABS with the data file EXSTL will create a post processing file EXSTL.PST.

The user may then also prepare a STEELER input data file using any text editor. This data file must conform to the specifications detailed in Chapter IV of this manual. This data file is not required if all program defaults are acceptable. Sample data is provided on the disk (filenames EXSTL for ETABS data and CHKSTL for STEELER data) associated with the complete STEELER package.

C. Executing The STEELER Program

This section explains how to execute the STEELER program.

STEELER is a DOS program which can be run in a DOS window or can be launched from the Windows Icon.

Running STEELER in a DOS Window

To execute STEELER in a DOS Window enter the following command at the DOS prompt:

STEELER ***etabsfile steelerfile***

Where ***etabsfile*** is the name of the ETABS input file which has already been run and for which the ETABS post processing file is available; and ***steelerfile*** is the name of the STEELER input file. If no STEELER input file is prepared because all program defaults were acceptable the ***steelerfile*** could be left blank. Both the ***etabsfile*** and the ***steelerfile*** can have paths included. Other command line options identical to options for executing ETABS (/M:nnnnn and /I) are also available. Refer to the ETABS manual for an explanation of these options.

As an example, say that ETABS has been run using an input data file named EXSTL, to create the post processing file EXSTL.PST; and that the data associated with the design of this structure for the STEELER post processor has been prepared and entered into a data file called CHKSTL. In order to execute the STEELER program, enter the following command at the DOS prompt:

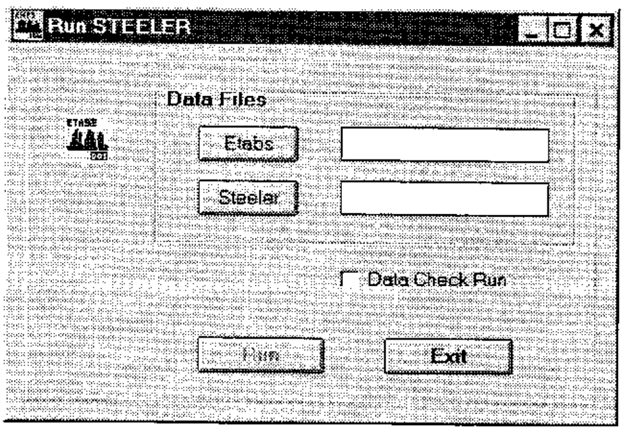
STEELER EXSTL CHKSTL

Note: Since no paths have been specified with the input files the ETABS post processing file EXSTL.PST and the STEELER input data file CHKSTL must reside in the current directory where the command is entered. Also the STEELER executable must reside in the same directory unless a path to the STEELER executable has been activated.

After a few seconds a copyright notice will appear on the screen. The program will then go into execution mode and a series of progress messages will be flashed to the screen until the job has been completed. If the job completes successfully the last screen displays the names of the input files used and the output files created.

Running STEELER from Windows

To execute STEELER from Windows double click on the STEELER icon in the ETABS program group installed by the setup program. The following dialog box will appear:



Click the **Etabs** button which brings up the Open File Dialog box. Select the ETABS postprocessing file. Similarly select the STEELER input file by clicking on the **Steeler** button. If all program defaults are acceptable then the STEELER input filename can be left blank. Clicking on **Run** launches the STEELER program. The program runs minimized in a DOS window.

The output files created by the program are explained in Chapter V. The files can be viewed and printed using any text editor. To print an output file from DOS the **PRINT** command may be used. Appropriate line counts and page ejects are built into the files.

Chapter III

Design Algorithms

This chapter describes in detail the various aspects of the steel stress / capacity check and design procedures that are used by the program STEELER.

Special terminology associated with the input and the output of the program is also described in the following sections.

An engineering background in the general area of multistory structural steel design and user familiarity with References [1], [2], [3] or [4] is assumed.

Sections A, B, C, and D describing design loading combinations, element unsupported length calculations, effective length (-K) factor calculations, and element stations and force components are common to the various codes. Section E describing the check and design procedures is based on the AISC-ASD89 [1] and UBC94 [2] codes. The two codes are similar when seismic design is not requested. Section F describes the additional checks made for the various framing types when seismic design is requested based on the UBC94 code. Section G describes the check and design procedures when design based on AISC Plastic89 [Reference 1, Section N] code is requested. Section H describes the check and design procedures when design based on AISC-LRFD93 [3] code is requested. Section I describes the check and design procedures when design based on CISC89 [4] code is requested.

References to pertinent sections and equations of the AISC-ASD89 [1] are indicated with the "AISC" prefix, and similarly for other codes. For simplicity, all equations and descriptions presented in this chapter correspond to inch-kip-second units unless otherwise noted.

A. Design Load Combinations

The design load combinations are used for determining the various combinations of the load conditions for which the structure needs to be checked.

These load combinations are defined as part of the STEELER input data and are totally independent of the load cases that are specified as part of the ETABS data. The user is referred to the ETABS manual for distinct definitions of load conditions and load cases.

The ETABS post processing file brings across forces and moments associated with the eight independent load conditions (I, II, III, A, B, C, D1 and D2) for each of the members. The load combination multipliers are then applied to values of the forces and moments from these load conditions to form the design forces for each load combination. There is one exception to the above. For dynamic analysis and for SRSS combinations, any correspondence between the signs of the moments and axial loads is lost. The program uses two loading combinations for each such loading combination specified, reversing the sign of axial loads in one of them.

For the AISC-ASD89 code, if a structure is subjected to dead (DL) and live load (LL) only, the stress check may need only one load combination, namely DL + LL (AISC A4). However, if in addition to the dead and live load the structure is subjected to wind forces from two mutually perpendicular directions (WX and WY), and considering that wind forces are reversible, then the following nine load combinations may have to be defined :

1. DL + LL
2. 0.75 (DL + LL + WX)
3. 0.75 (DL + LL - WX)
4. 0.75 (DL + LL + WY)
5. 0.75 (DL + LL - WY)
6. 0.75 (DL + WX)
7. 0.75 (DL - WX)
8. 0.75 (DL + WY)
9. 0.75 (DL - WY)

These are also the default design load combinations whenever the AISC-ASD89 code is used. For the default combinations the program assumes dead load is Load Condition I, live load is Load Condition II and lateral loads are in Load Conditions A and B.

All tension and compression effects are consistently evaluated with these combinations. The stress ratios from the controlling load combinations are reported.

The 0.75 factor associated with the load combinations involving wind or other short-term loads is to allow for the 33 1/3% increase in the stress allowables (AISC A5.2), so that all stress ratios will be compared to an allowable stress ratio of 1.0, irrespective of whether the controlling load combination has contributions from short-term loads or not.

It is possible in the post processing data to establish one of the vertical load conditions (I, II or III) as a live load condition. Live load reduction factors can then be applied to the member forces of this load condition to reduce the contributions of the live load to the stress ratios.

If seismic design is to be checked based on the UBC94 code the following loading combinations may have to be considered:

1. $DL + LL$
2. $0.75\,(DL + LL + \sqrt{EX^2 + EY^2} + ET)$
3. $0.75\,(DL + LL + \sqrt{EX^2 + EY^2} - ET)$
4. $0.75\,(DL + LL - \sqrt{EX^2 + EY^2} + ET)$
5. $0.75\,(DL + LL - \sqrt{EX^2 + EY^2} - ET)$
6. $0.75\,(DL + \sqrt{EX^2 + EY^2} + ET)$
7. $0.75\,(DL + \sqrt{EX^2 + EY^2} - ET)$
8. $0.75\,(DL - \sqrt{EX^2 + EY^2} + ET)$
9. $0.75\,(DL - \sqrt{EX^2 + EY^2} - ET)$

Again the 0.75 factor is for the one third increase in the allowable stresses, so that all stress ratios are compared against unity while checking for overstress.

These are the program defaults whenever the UBC94 code is used, assuming dead load (DL) is gravity Load Condition I, live load (LL) is gravity Load Condition II, X direction seismic load (EX) is static lateral Load Condition A, Y direction seismic load (EY) is static lateral Load Condition B and accidental torsion seismic load (ET) is static lateral Load Condition C.

If checks have to be made based on the AISC Plastic89 code the following loads may have to be considered (AISC N1):

1. 1.7 (DL + LL)
2. 1.3 (DL + LL + WX)
3. 1.3 (DL+ LL + WY)
4. 1.3 (DL + LL - WX)
5. 1.3 (DL + LL - WY)
6. DL + 1.3 WX
7. DL + 1.3 WY
8. DL - 1.3 WX
9. DL - 1.3 WY

These are also the default design load combinations whenever the AISC Plastic89 code is used. For the default combinations the program assumes dead load is Load Condition I, live load is Load Condition II and lateral loads are in Load Conditions A and B.

If checks have to be made based on the AISC-LRFD93 code the following loads may have to be considered (AISC A4.1):

1. 1.4 DL
2. 1.2 DL + 1.6 LL
3. 1.2 DL + 0.5 LL + 1.3 WX
4. 1.2 DL + 0.5 LL - 1.3 WX
5. 1.2 DL + 0.5 LL + 1.3 WY
6. 1.2 DL + 0.5 LL - 1.3 WY
7. 0.9 DL + 1.3 WX
8. 0.9 DL - 1.3 WX
9. 0.9 DL + 1.3 WY
10. 0.9 DL - 1.3 WY

These are also the default design load combinations whenever the AISC-LRFD93 code is used. For the default combinations the program assumes dead load is Load Condition I, live load is Load Condition II and lateral loads are in Load Conditions A and B.

The user should use other appropriate loading combinations if roof live load is separately treated, or if other types of loads are present, or if pattern live loads are to be considered.

When using the AISC-LRFD93 code the program assumes that a P-Delta analysis has been performed in ETABS so that moment magnification factors for moments causing sidesway can be taken as unity. The code would require that the P-Delta analysis be done at the factored load level [5]. The ETABS program uses the specified story masses to calculate the P for the P-Delta analysis, but allows an input factor to modify it. It is recommended that a factor be used to obtain a P equivalent to 1.2 dead load plus 0.5 live load.

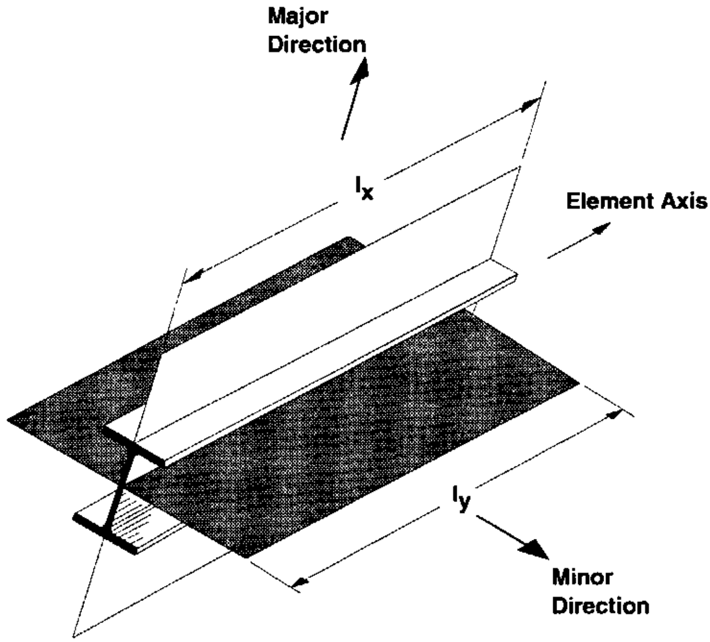
If checks have to be made based on the CISC89 code the following loads may have to be considered (CISC 7.2):

1. 1.25 DL
2. 1.25 DL + 1.5 LL
3. 1.25 DL + 1.05 LL + 1.05 WX
4. 1.25 DL + 1.05 LL - 1.05 WX
5. 1.25 DL + 1.05 LL + 1.05 WY
6. 1.25 DL + 1.05 LL - 1.05 WY
7. 1.25 DL + 1.5 WX
8. 1.25 DL - 1.5 WX
9. 1.25 DL + 1.5 WY
10. 1.25 DL - 1.5 WY
11. 0.85 DL + 1.5 WX
12. 0.85 DL - 1.5 WX
13. 0.85 DL + 1.5 WY
14. 0.85 DL - 1.5 WY

These are also the default design load combinations whenever CISC89 Code is used. For the default combinations the program assumes dead load is Load Condition I, live load is Load Condition II and lateral loads are in Load Conditions A and B.

The user should use other appropriate loading combinations if roof live load is separately treated, or if other types of loads are present, or if pattern live loads are to be considered.

For the gravity load case the code requires (CISC 8.6.2) that the translational moments not be less than the moments produced by pseudo-lateral loads, applied at each story, equal to 0.005 times the factored gravity loads acting at each story. If



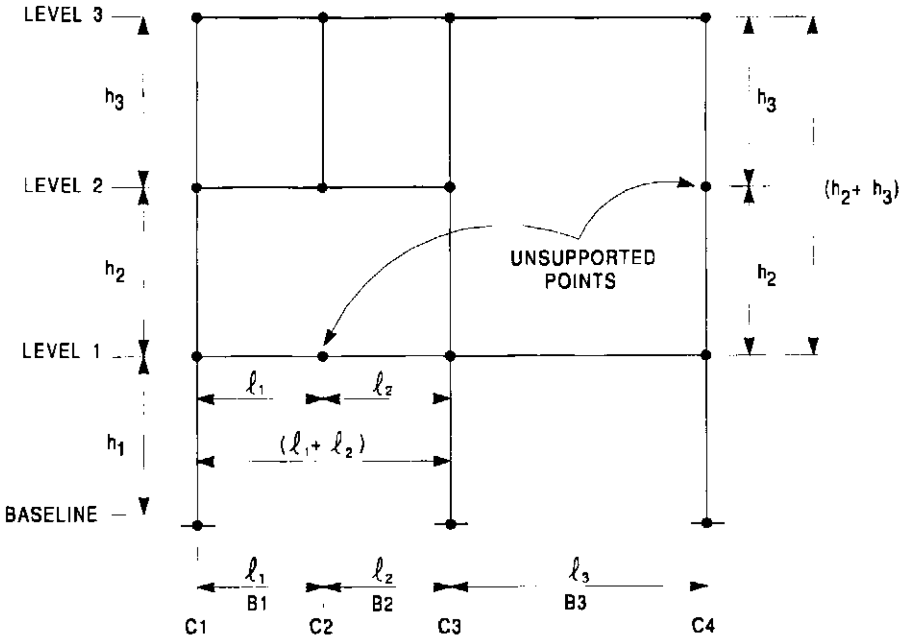
Element Unsupported Length
Figure III-1

extra load conditions are used for such analysis they should be included in the loading combinations with due consideration of the fact that the pseudo-lateral forces can be positive or negative.

When using the CISC89 code the program assumes that a P-Delta analysis has been performed in ETABS so that moment magnification factors for moments causing sidesway can be taken as unity. The code would require that the P-Delta analysis be done at the factored load level [5]. The ETABS program uses the specified story masses to calculate the P for the P-Delta analysis, but allows an input factor to modify it. It is recommended that a factor be used to obtain a P equivalent to 1.25 dead load plus 1.05 live load.

B. Unsupported Lengths of Elements

This section outlines the procedures used for the determination of the unsupported lengths of the column, beam and brace elements.



Tracing Unsupported Element Lengths
Figure III-2

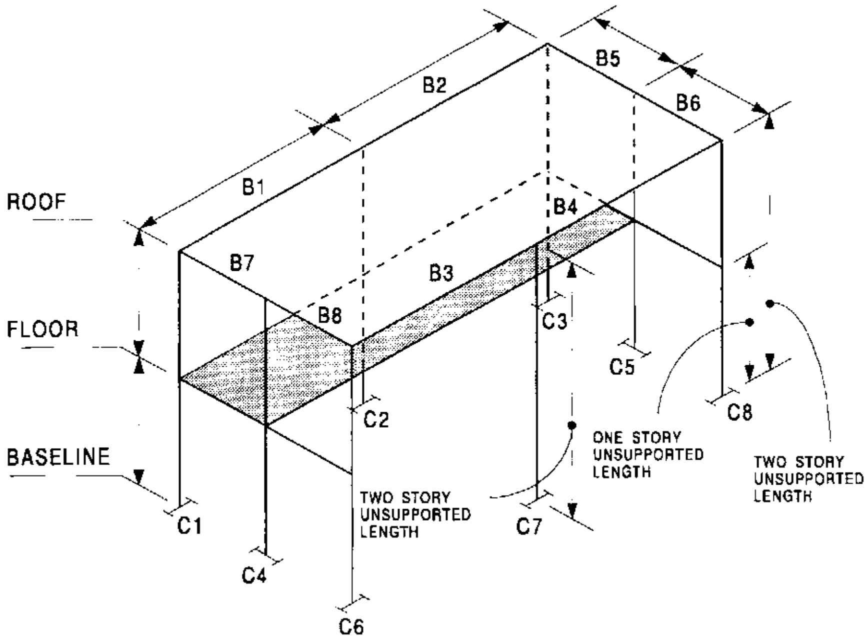
The two unsupported lengths are l_x and l_y, corresponding to instability in the major and minor directions of the element, respectively. See Figure III-1. These are the lengths between the support points of the element in the corresponding directions.

For beam- and column-type elements, the program will actually travel past any unsupported ends of the elements to automatically locate the element support point, and evaluate the corresponding unsupported element length.

Therefore, the unsupported length of a column may actually be evaluated as being greater than the corresponding story height, and the unsupported length of a beam may actually be evaluated as being greater than the bay length.

As shown in Figure III-2, the unsupported length l_x for the beam in Bay 1 at level 1 will not be l_1, but $(l_1 + l_2)$; and the unsupported length for the column at column line 4 at level 3 will not be h_3, but $(h_2 + h_3)$.

Therefore, in determining the values for l_x and l_y for the beam and column elements, the program recognizes various aspects of the structure that have an effect on these



Conditions Affecting Unsupported Lengths
Figure III-3

lengths, such as member connectivity and diaphragm disconnections as described in this section.

Typically for columns, all floor diaphragms are assumed to be lateral support points, therefore, the unsupported length of a column is equal to the story height associated with the level. However, if a column is disconnected from any level, the unsupported lengths of the column are automatically recognized as being longer than the story height as a lateral support point is eliminated.

The program also recognizes that the columns can have different unsupported lengths corresponding to the major and minor directions. For example, beams framing into disconnected column points in the column major or minor directions will give lateral support in the corresponding directions. However, if a beam frames into only one direction of the column at the level where the column has been disconnected from the diaphragm, the beam is assumed to give lateral support only in that direction. As shown in Figure III-3, column line C8 laterally spans two stories in the column major direction and spans only one story in the column minor direction. Column line C1 is laterally supported at both levels in both directions and column line C7 is laterally unsupported for two levels in both directions.

In all such situations, lateral support points and associated lateral unsupported lengths in corresponding directions are automatically recognized by the program and included in the calculation of the unsupported lengths of the columns.

Similarly, the lateral unsupported lengths for beams are as shown in Figure III-2.

For beams any column, brace or wall support is assumed to be the location of the vertical support to the beam in the major direction **as well as the lateral support to the beam in the minor direction**.

For brace elements, the unsupported element length is always assumed equal to the actual element length.

The program has an option to display the calculated unsupported lengths of all the elements. These values may be overridden by user-specified values as part of the STEELER input data.

C. Effective Length (K-) Factors

This section outlines the procedure used for the determination of the effective length (K-) factors of the column, beam and brace elements.

There are two K-factors, K_x and K_y, associated with each element. These values correspond to instability associated with the major and minor directions of the element, respectively.

The calculation of the K-factor for a column in a particular direction involves the evaluation of the stiffness ratios, G^{top} and G^{bot} corresponding to the top and bottom support points of the column, in the direction under consideration where:

$$G^{top} = \frac{\dfrac{E_{ca}\,I_{ca}}{L_{ca}} + \dfrac{E_{cb}\,I_{cb}}{L_{cb}}}{\displaystyle\sum_{n=1}^{nb} \frac{E_{gn}\,I_{gn}}{L_{gn}} \cos^2 \theta_n}$$

where

E_{ca}	=	Modulus of elasticity of column above top lateral support point
E_{cb}	=	Modulus of elasticity of column below top lateral support point
I_{ca}	=	Moment of inertia of column above top lateral support point
I_{cb}	=	Moment of inertia of column below top lateral support point
L_{ca}	=	Unsupported length of column in direction under consideration above top lateral support point
L_{cb}	=	Unsupported length of column in direction under consideration below top lateral support point
E_{gn}	=	Modulus of elasticity of beam, n, at top lateral support point
I_{gn}	=	Major moment of inertia of beam, n, at top lateral support point
L_{gn}	=	Unsupported length of beam, in major direction of beam, n, at top column support point
nb	=	Number of beams that connect to the column at lateral support level
θ_n	=	Angle between the column direction under consideration and the beam, n

For the K-factor calculation above, the unsupported lengths are based on full member lengths and do not consider any rigid end offsets.

The calculation for G^{bot} is similar, as it corresponds to the bottom lateral support point.

The column K-factor for the corresponding direction is then calculated by solving the following relationship for α:

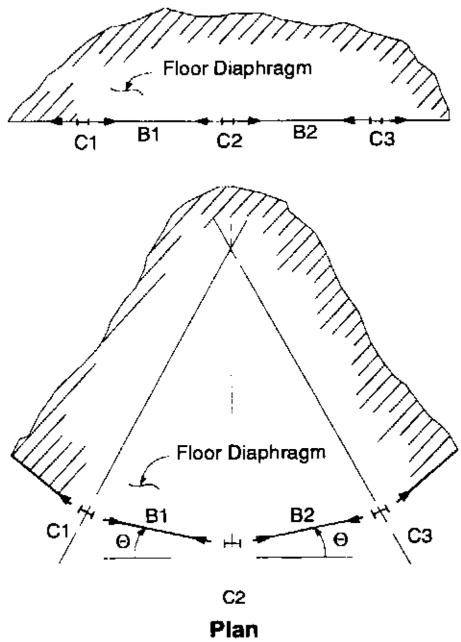
$$\frac{\alpha^2\, G^{top}\, G^{bot} - 36}{6\,(\,G^{top} + G^{bot}\,)} = \frac{\alpha}{\tan\alpha}$$

from which $K = \dfrac{\pi}{\alpha}$

This relationship is the mathematical formulation for K-factor evaluation assuming the sidesway to be uninhibited.

The following are some important aspects associated with the column K-factor algorithm:

- In order to prevent inconsistencies from entering the column K-factor formulation, it is strongly recommended that the section properties between support points of the elements be kept constant.



K-Factor Ambiguity
Figure III-4

- Cantilever beams and beams and columns having pin ends at the joint under consideration are excluded in the calculation of the stiffness EI/L summations.
- A column or beam that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated EI/L value.
- If a pin release exists at a particular end of a column, the corresponding G-value is set to 10.0.
- If no pin exists at the baseline of a column, the corresponding G-value is set to 1.0 in both directions.
- If there are no beams framing into a particular direction of a column, 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 a column, the corresponding K-factors are set to unity.

The automated K-factor calculation procedure can generate artificially high K-factors, under certain circumstances.

For example, in the top part of Figure III-4 column line C2 has no beams framing in a direction parallel to the column minor direction at any level. The G^{top} and G^{bot} values for this column are infinity. The program assumes such columns to be laterally supported by the floor diaphragms and assigns the column a K-factor of unity.

Now consider the condition where the beams framing into a column are slightly inclined with the column major axis, as shown in the bottom part of Figure III-4 for column line C2. The small components of the beam stiffness in the column minor direction will generate small G^{top} and G^{bot} values for the column minor direction, resulting in a large minor direction K-factor.

In general, such columns are laterally supported by the floor diaphragms in the minor directions and should be assigned a K-factor of unity. The program has options whereby the user can override the K-factor calculated by the program.

For braced frames, the column K-factors are always set to unity.

The K-factors for the beam and brace elements are assumed to be unity.

The program has an option to display the calculated K-factors of all the elements. These values may be overridden by user-specified values as part of the STEELER input data. The user may want to use this override option in order to use inelastic K-factors, as opposed to the elastic K-factors calculated by the program.

D. Element Stations and Force Components

The column, beam and brace elements are stress checked for combined axial force and biaxial bending effects.

For columns and braces, the checks are evaluated at the top and bottom ends of the clear element length.

For the beams, these checks are calculated at five equidistant stations along the clear element length, namely, END I, 1/4-PT, MIDDLE, 3/4-PT and END J.

Due to the rigid floor diaphragm modeling, a beam element will generally only have bending about the beam major axis and no axial forces. However, the element will generate axial action and biaxial bending if any one of the column lines at the ends of the beam is disconnected from the rigid diaphragm at the level of the beam.

E. Check/ Design Procedures (AISC-ASD89 and UBC94)

This section defines the stress check and design procedure used by program STEELER as it applies to the column, beam and brace elements and beam to column joints. The procedure in this section is according to the AISC-ASD89 [1]. The procedure used for UBC94 without the seismic requirements is identical. The additional seismic requirements checked under UBC94 are described in Section F.

Various notations used in this section are tabulated in Figure III-5.

First, for each station along the length of the member, for each load combination, the actual member stress components and corresponding stress allowables are calculated.

Then, the elastic stress ratios are evaluated at each station for each member under the influence of each of the design load combinations using the corresponding equations that are defined in this section. The controlling compression and/or tension stress ratio is output, along with the associated station, load combination and equation. A stress ratio greater than 1.0 indicates an overstress. Similarly, a shear stress ratio is separately output.

The program also evaluates the need for continuity and doubler plates in the columns of moment frames at the beam to column joints and reports the sizes required. Beam connection shears are reported for design of these connections.

1. Evaluation of Stress Ratios

a. Calculation of Actual Stresses

The factored member stresses that are calculated for each load combination are:

$$f_a = P/A$$

$$f_{bx} = M_x/S_x$$

$$f_{by} = M_y/S_y$$

A	=	Cross-sectional area, in^2
A_f	=	Area of compression flange , in^2
A_{vx}, A_{vy}	=	Major and minor shear areas, in^2
C_b	=	Bending coefficient
C_m	=	Moment coefficient
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
F_a	=	Allowable axial compressive stress, ksi
F_{bx}, F_{by}	=	Allowable major and minor bending stresses, ksi
F'_{ex}	=	$\dfrac{12\,\pi^2 E}{23\,(K_x l_x / r_x)^2}$
F'_{ey}	=	$\dfrac{12\,\pi^2 E}{23\,(K_y l_y / r_y)^2}$
F_t	=	Allowable axial tensile stress, ksi
F_v	=	Allowable shear stress, ksi
F_y	=	Yield stress of material, ksi
K_x, K_y	=	Effective length K-factors in the major and minor directions
M_x, M_y	=	Factored major and minor moments in member, kip-in
P	=	Factored axial force in member, kips
S_x, S_y	=	Major and minor section moduli, in^3
V_x, V_y	=	Factored major and minor shear forces, kips
b	=	Nominal dimension of longer leg of angles, in
	=	$b_f - 2t_w$ for welded BOX sections, in
	=	$b_f - 3t_f$ for rolled box (TS) sections, in
b_f	=	Flange width, in
d	=	Depth of web, in
h	=	Clear distance between flanges, in
l_x, l_y	=	Major and minor direction unbraced member lengths, in
r_x, r_y	=	Radii of gyration in the major and minor directions, in
t	=	Thickness, in
t_f	=	Flange thickness, in
t_w	=	Thickness of web, in

Notation (AISC-ASD89 and UBC94)
Figure III-5

$$f_{vx} = V_x/A_{vx}$$

$$f_{vy} = V_y/A_{vy}$$

These factored stresses are calculated at each of the previously defined stations corresponding to each element type.

b. Classification of Sections

The allowable stresses for axial compression and flexure are dependent on the classification of the section as compact, non-compact or slender. The program makes the checks shown in Figure III-6 and reports the classifications for the individual members (AISC B5.1). If the section dimensions satisfy the limits shown in Figure III-6, the section is classified as COMPACT or NON-COMPACT. If the limits for non-compact are not met, the section is classified as SLENDER, and the stress check is terminated.

c. Calculation of Allowable Stresses

i. Compression

The allowable axial compressive stress value, F_a, for COMPACT or NON-COMPACT sections, is evaluated as follows:

when $\frac{Kl}{r} \leq C_c$

$$F_a = \frac{\left[1.0 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3\left(\frac{Kl}{r}\right)}{8C_c} - \frac{\left(\frac{Kl}{r}\right)^3}{8C_c^3}} \qquad \text{(AISC E2-1)}$$

where $\frac{Kl}{r}$ is the larger of $\frac{K_x l_x}{r_x}$ and $\frac{K_y l_y}{r_y}$

and $C_c = \sqrt{(2\pi^2 E) / F_y}$

Description of Section	Ratio Checked	COMPACT	NON-COMPACT
I-SHAPES			
flanges	$b_f/2t_f$	$65/\sqrt{F_y}$	$95/\sqrt{F_y}$
webs	d/t_w	for $f_a/F_y \leq 0.16$ $\frac{640}{\sqrt{F_y}}\left(1-3.74\frac{f_a}{F_y}\right)$ for $f_a/F_y > 0.16$ $257/\sqrt{F_y}$	—
webs	h/t_w	—	$760/\sqrt{F_b}$
BOX			
flanges	b/t_f	$190/\sqrt{F_y}$	$238/\sqrt{F_y}$
webs	d/t_w	as for I-shapes	—
webs	h/t_w	—	as for I-shapes
CHANNELS			
flanges	b_f/t_f	Not applicable	$95/\sqrt{F_y}$
webs	h/t_w	Not applicable	as for I-shapes
T-SHAPES			
flanges	$b_f/2t_f$	Not applicable	$95/\sqrt{F_y}$
stem	d/t_w	Not applicable	$127/\sqrt{F_y}$
ANGLES or DOUBLE ANGLES	b/t	Not applicable	$76/\sqrt{F_y}$
PIPES	D/t	$3300/F_y$	—
RECT or CIRCLE	—	Assumed compact	—
USER	—	—	Assumed non-compact

Classification of Sections (AISC-ASD89)
Figure III-6

otherwise, if $Kl/r > C_c$

$$F_a = \frac{12\,\pi^2 E}{23\,(Kl/r)^2} \qquad \text{(AISC E2-2)}$$

Note: for single angles r_z is used in place of r_x and r_y.

For members in compression, if $\frac{Kl}{r}$ is greater than 200, a message to that effect is printed (AISC B7).

ii. Tension

The allowable axial tensile stress value F_t is assumed to be $0.60\,F_y$ (AISC D1). It is noted that no net section checks are made.

For members in tension, if l/r is greater than 300, a message to that effect is printed (AISC B7).

iii. Major and Minor Direction Bending

To obtain the allowable bending stress the program uses the following criteria:

For all I-sections, C-sections, T-sections, angles and double angles the allowable major direction bending stress is computed based on the compactness criteria and the laterally unbraced length, l_y.

If l_y is less than

$$\frac{76 b_f}{\sqrt{F_y}} \text{ and } \frac{20000}{\left(\frac{d}{A_f}\right) F_y} \qquad \text{(AISC F1-2)}$$

and the section is COMPACT, the allowable major direction bending stress is taken as

$$F_{bx} = 0.66\,F_y \qquad \text{(AISC F1-1)}$$

If l_y is less than the above limits and the section is NON-COMPACT,

$$F_{bx} = 0.60\,F_y \qquad \text{(AISC F1-5)}$$

If the unbraced length l_y exceeds the above limits, then (for both COMPACT and NON-COMPACT sections)

$$F_{bx} = \frac{12x10^3 \, C_b}{l_y\left(\frac{d}{A_f}\right)} \leq 0.6 \, F_y \qquad \text{(AISC F1-8)}$$

where

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3$$

M_1 and M_2 are end moments of the unbraced segment and M_1 is less than M_2, $\left(\frac{M_1}{M_2}\right)$ being positive for double curvature bending and negative for single curvature bending. Also, if any moment within the segment is greater than M_2, C_b is taken as 1.0. C_b is also taken as 1.0 for frames braced against joint translation, for cantilevers, for beams spanning multiple bays and for columns disconnected from the diaphragm.

The program defaults C_b to 1.0 if the unbraced length, l_y of the member is overwritten 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 minor direction allowable bending stress, F_{by} is taken as:

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

except in the case of COMPACT I-sections it is taken as

$$F_{by} = 0.75 \, F_y \qquad \text{(AISC F2-1)}$$

For box sections and rectangular tubes, the allowable bending stress in both the major and minor directions is taken as:

$$F_b = 0.66 \, F_y \qquad \text{(AISC F3-1)}$$

provided the section is COMPACT and the unbraced length l_y is less than the greater of

$$\left(1950 + 1200\frac{M_1}{M_2}\right)\frac{b}{F_y} \quad \text{or} \quad 1200\frac{b}{F_y}$$

where M_1 and M_2 have the same definition as noted earlier in the formula for C_b. If l_y is specified by the user, the first formula is ignored.

If the unbraced length l_y exceeds the above limits or the section is NON-COMPACT:

$$F_b = 0.60\,F_y \qquad \text{(AISC F3-3)}$$

For pipe sections the allowable bending stress in all directions is taken as

$$F_b = 0.66\,F_y \qquad \text{(AISC F3-1)}$$

provided the section is COMPACT, otherwise

$$F_b = 0.60\,F_y \qquad \text{(AISC F3-3)}$$

For rectangular and circular sections the allowable bending stress in both major and minor directions is taken as

$$F_b = 0.66\,F_y$$

For user defined sections the allowable bending stress in both major and minor directions is taken as

$$F_b = 0.60\,F_y$$

iv. Shear

The allowable shear stress F_v is assumed to be $0.40F_y$ (AISC F4-1). For very slender webs, where $\frac{h}{t_w} > \frac{380}{\sqrt{F_y}}$, a reduction in the allowable shear stress applies and must be separately investigated by the user (AISC F4).

For SLENDER sections and other singly symmetric and unsymmetric sections requiring consideration of flexural-torsional and torsional buckling, a reduction factor in the allowable stress may be applicable. The user must separately investigate this reduction if such elements are used.

If the user specifies allowable bending stress values in the ETABS and /or STEELER material property data, these values will override all above mentioned program calculated values.

d. Calculation of Stress Ratios

From the allowable axial and bending stress values and the factored axial and bending member stresses at each station, for each of the load combinations, an interaction stress ratio is produced as follows:

If f_a is compressive and $f_a/F_a > 0.15$, the compressive stress ratio CR is given by the larger of CR1a and CR1b, where

$$CR1a = \frac{f_a}{F_a} + \frac{C_{mx}\, f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my}\, f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \qquad \text{(AISC H1-1)}$$

and

$$CR1b = \frac{f_a}{0.60\, F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \qquad \text{(AISC H1-2)}$$

If $f_a/F_a \leq 0.15$, CR = CR2, where

$$CR2 = \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \qquad \text{(AISC H1-3)}$$

where C_{mx} and C_{my} are coefficients representing distribution of moment along member length and are assumed to be 1.0 for all cases except for columns in unbraced frames when they are taken as 0.85. However, users can specify overriding values.

If f_a is tensile or zero, the tensile stress ratio TR is given by the larger of TR1b and TR2, where

$$TR1b = \frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \qquad \text{(AISC H2-1)}$$

and

$$TR2 = \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \qquad \text{(BENDING)}$$

In the calculation of TR1b, F_{bx} and F_{by} have a minimum value of $0.60F_y$.

For circular sections an SRSS combination is first made of the two bending components before adding the axial load component instead of the simple algebraic addition implied by the above formulas for CR and TR.

From the allowable shear stress values and the factored shear stress values at each station, for each of the load combinations, shear stress ratios for each direction are produced as follows:

$$VR_x = \frac{f_{vx}}{F_v} \quad \text{and} \quad VR_y = \frac{f_{vy}}{F_v}$$

2. 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:

a. The steel beam frames in a direction parallel to the column major direction, i.e. the beam frames into the column flange.

b. The steel beam frames in a direction parallel to the column minor direction, i.e. the beam frames into the column web.

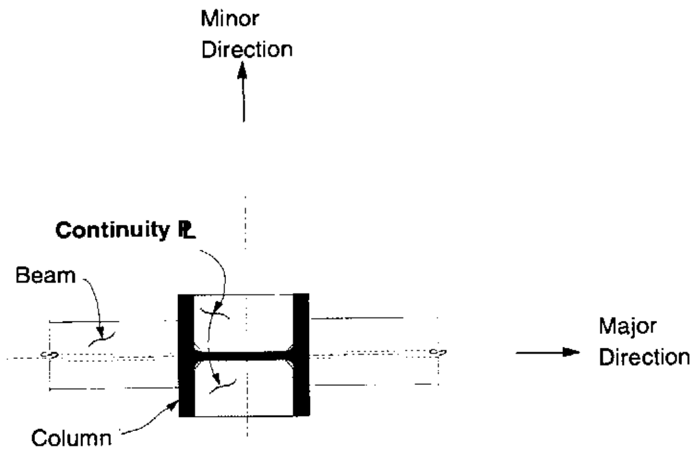
c. The steel beam frames in a direction that is at an angle to both of the principal axes of the column, i.e. the beam frames partially into the column web and partially into the column flange.

To achieve a beam/column moment connection, continuity plates such as shown in Figure III-7 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 by b and c above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam.

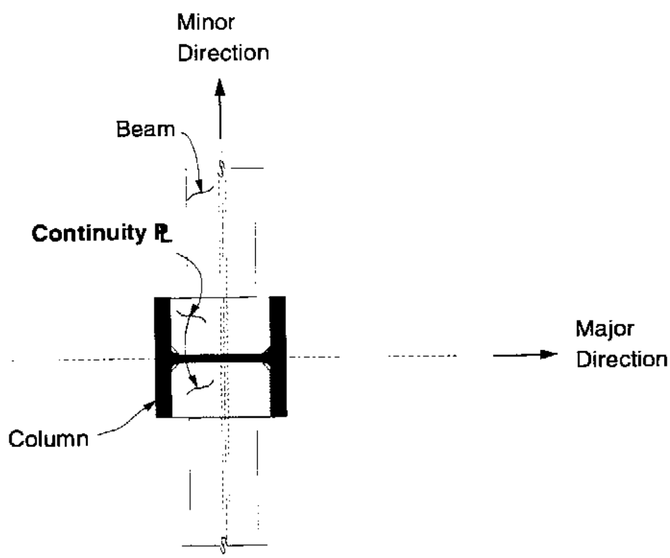
However, for the connection condition described by a above, where the beam frames into the flange of the column, such continuity plates are not always needed. The requirement depends upon the magnitude of the beam-flange force and the properties of the column.

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



PLAN

Beam Framing Into Column Flange



PLAN

Beam Framing Into Column Web

Plan Showing Continuity Plates
Figure III-7

direction) and reports the maximum continuity plate area that is needed for each beam flange. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.

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

$$A_{cp} = \frac{P_{bf}}{F_{yc}} - t_{wc}\,(t_{fb} + 5k_c) \qquad \text{(AISC K1-9)}$$

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

a. The depth of the column clear of the fillets, i.e. $d_c - 2k_c$ is less than or equal to:

$$\frac{4100\, t_{wc}^3 \sqrt{F_{yc}}}{P_{bf}} \qquad \text{(AISC K1-8)}$$

b. The thickness of the column flange, t_{fc}, is greater than or equal to:

$$0.4 \sqrt{\frac{P_{bf}}{F_{yc}}} \qquad \text{(AISC K1-1)}$$

Where $P_{bf} = f_b\, A_{bf}$.

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

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

$$t_{cp}^{min} = 0.5\, t_{fb} \qquad \text{(AISC K1.8.2)}$$

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

$$b_{cp}^{min} = 2\left(\frac{b_{fp}}{3} - \frac{t_{wc}}{2}\right) \qquad \text{(AISC K1.8.1)}$$

so that the minimum area is given by:

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

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

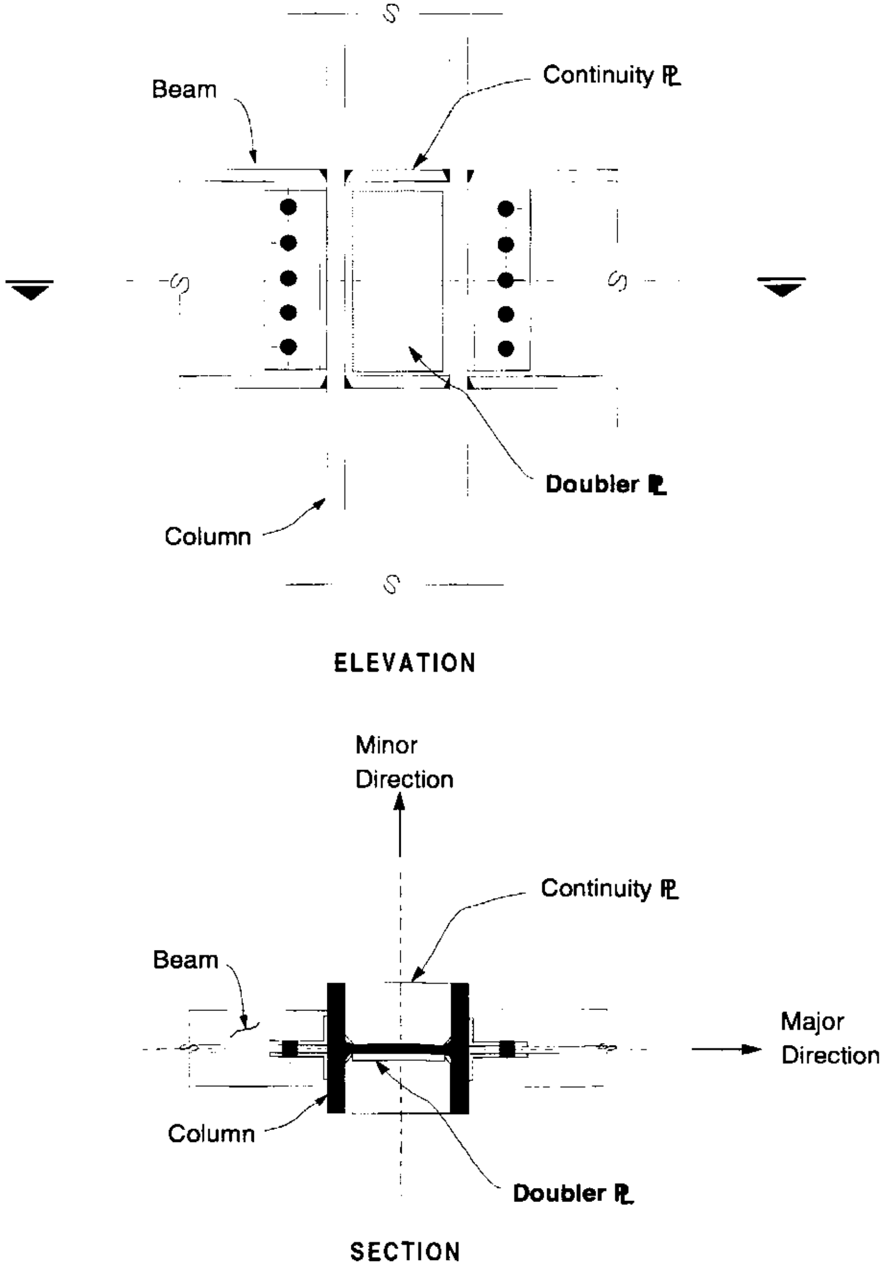
Where

A_{bf}	=	Area of beam flange
A_{cp}	=	Required continuity plate area
F_{yb}	=	Yield stress of beam material
F_{yc}	=	Yield stress of the column and continuity plate material
t_{fb}	=	Beam flange thickness
t_{wc}	=	Column web thickness
k_c	=	Distance between outer face of the column flange and web toe of its fillet
d_c	=	Column depth
d_b	=	Beam depth
f_b	=	Beam flange width
t_{cp}	=	Continuity plate thickness
b_{cp}	=	Continuity plate width
f_b	=	Bending stress calculated from the larger of 5/3 of loading combinations with gravity loads only and 4/3 * 4/3 of the loading combinations with lateral loads (AISC K1.2) (The additional 4/3 in the above equation is included to account for the 0.75 factor that may be used by the user in the lateral load combination to account in turn for the 1/3rd increase in allowable stress for such conditions)

3. 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.



Doubler Plates
Figure III-8

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 III-8. The program investigates such situations and reports the thickness of any required doubler plates. Only columns with I shapes are investigated for doubler plate requirements. Also doubler plate requirements are evaluated for moment frames only. No check is made for braced frames.

The shear force in the panel zone, is given by

$$V_p = P - V_c$$

or

$$V_p = \sum_{n=1}^{nb} \frac{M_{bn}}{d_n - t_{f_n}} \cos \theta_n - V_c$$

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

$$t_r = \frac{V_p}{F_v\, d_c}$$

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

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

where

F_{yc}	=	Yield stress of the column and doubler plate material
t_r	=	Required column web thickness
t_{dp}	=	Required doubler plate thickness
t_{wc}	=	Column web thickness
V_p	=	Panel zone shear
V_c	=	Column shear in column above
P	=	Beam flange forces
n_b	=	Number of beams connecting to column
d_n	=	Depth of n-th beam connecting to column
θ_n	=	Angle between n-th beam and column major direction
d_c	=	Depth of column

M_{bn} = Calculated factored beam moment from the corresponding loading combination

F_v = $0.40\ F_{yc}$

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

4. 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 shears reported are the maxima of the factored shears obtained from the loading combinations.

F. Additional Seismic Requirements (UBC94)

This section describes in detail the various additional design checks used by the program STEELER for design based on the special seismic requirements of the Uniform Building Code, 1994 (UBC94) [2].

The special requirements checked by the program are dependent on the type of framing used and are described below for each type of framing. The requirements checked are based on UBC Section 2211.

It is noted here that whenever **special loading combinations** are required by the code for a specific element of the structure, the program assumes that the ETABS Load condition I is associated with the Dead Load, the ETABS live load condition is as specified in the input data, the seismic loading (**except if a dynamic analysis has been made**) is in Load conditions A, B and C for X and Y direction of loading and torsional loading from accidental eccentricities. For the case of a dynamic analysis, Load conditions D1 and D2 are considered active and correspond to the two orthogonal directions of seismic excitation and Load condition C is assumed to contain statically applied accidental torsional loads. The special loading combination factors are applied directly to the ETABS load conditions. It is assumed that any required scaling (such as may be required to scale response spectra results) has already been applied to the ETABS load conditions.

1. Ordinary Moment Frames

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

a. Whenever the axial stress, f_a, in columns due to the prescribed loading combinations exceeds $0.3F_y$, the following axial load combinations are checked with respect to the column axial load capacity (UBC 2211.5.1):

$$P_{DL} + 0.7P_{LL} \pm \frac{3}{8}R_w\left(\sqrt{P_X^2 + P_Y^2} + |P_T|\right)$$

$$0.85P_{DL} \pm \frac{3}{8}R_w\left(\sqrt{P_X^2 + P_Y^2} + |P_T|\right)$$

where P_{DL}, P_{LL}, P_X, P_Y and P_T are axial loads due to dead load, live load, seismic loads in the X direction, seismic loads in the Y direction, and seismic loads due to accidental torsion, respectively.

For this case the column axial capacity in compression is taken to be $1.7F_aA$ and the capacity in tension is taken as F_yA.

Unless specified in the STEELER input data, the value of R_w defaults to 6 for this framing system.

b. The beam connection shears reported are the maximum of the specified loading combinations and the following additional loading combinations (UBC 2211.6):

$$V_{DL} + V_{LL} \pm \frac{3}{8}R_w\left(\sqrt{V_x^2 + V_y^2} + |V_T|\right)$$

where V_{DL}, V_{LL}, V_X, V_Y and V_T are beam shears due to dead load, live load, seismic loads in the X direction, seismic loads in the Y direction, and seismic loads due to accidental torsion, respectively.

c. The continuity plates reported are for a beam flange force, $P_{bf} = A_{bf}\,F_{yb}$. See Section E.2 for details.

2. Special Moment Resisting Frames

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

a. The additional requirement of column axial strength as noted in Section F.1.a above for ordinary moment frames also applies for the special moment resisting frames except the default value for R_w is assumed as 12.

b. The beam connection shears that are reported allow for the development of the full plastic moment capacity of the beam (UBC 2211.7.1). Thus:

$$V = \frac{C\, M_{pb}}{L} + V_{I,II,III}$$

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, ZF_y
L = Clear length of the beam
$V_{I,II,III}$ = Absolute maximum of the calculated factored beam shears at the corresponding beam ends from the loading combinations, considering the vertical load (I, II and III) components only

c. The panel zone doubler plate requirements that are reported will develop the lesser of beam moments equal to 0.8 of the plastic moment capacity of the beam ($0.8\sum M_{pb}$), or beam moments due to gravity loads plus 1.85 times the seismic load.

The capacity of the panel zone in resisting this shear is taken as (UBC 2211.7.2.1):

$$V_p = 0.55\, F_y\, d_c\, t_r \left(1 + \frac{3\, b_c\, t_{cf}^2}{d_b\, d_c\, t_r} \right)$$

giving the required panel zone thickness as

$$t_r = \frac{V_p}{0.55\, F_y\, d_c} - \frac{3\, b_c\, t_{cf}^2}{d_b\, d_c}$$

and the required doubler plate thickness as

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

where

b_c	=	width of column flange
t_{cf}	=	thickness of column flange
d_b	=	depth of deepest beam framing into the major direction of the column

d. Compact I-shaped beam sections are additionally checked for $b_f / 2t_f$ to be less than $52/\sqrt{F_y}$. Compact I-shaped column sections are additionally checked for $b_f / 2t_f$ to be less than the numbers given for plastic sections in Figure III-11. Compact box shaped column sections are additionally checked for b_f / t_f and d/ t_w to be less than $110/\sqrt{F_y}$. If this criteria is satisfied the section is reported as SEISMIC as described earlier under section classifications in Section E.1. If this criteria is not satisfied the user must modify the section property (UBC 2211.7.3).

e. For determining the need for continuity plates at joints due to tension transfer from the beam flanges, the force P_{bf} is taken as $1.8A_{bf}F_{yb}$ (UBC 2211.7.4).

For design of the continuity plate the beam flange force is taken as $A_{bf}\, F_{yb}$. See Section E.2 for details.

f. To facilitate the review of the strong column weak beam criterion (UBC 2211.7.5) 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) the capacity ratio is obtained as

$$R_{maj} = \sum_{n=1}^{nb} \frac{M_{pbn} \cos \theta_n}{M_{pcax} + M_{pcbx}}$$

For the minor direction of any column the capacity ratio is obtained as

$$R_{min} = \sum_{n=1}^{nb} \frac{M_{pbn} \sin \theta_n}{M_{pcay} + M_{pcby}}$$

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
θ_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
nb	=	Number of beams connecting to the column

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

$$M_{pc} = Z_c \, (F_{yc} - f_a)$$

where

Z_c	=	Plastic modulus of column
F_{yc}	=	Yield stress of column material
f_a	=	Maximum axial stress in the column from any of the loading combinations from gravity loads alone or 4/3 of the loading combinations including lateral loads. (The 4/3 factor is to offset the 0.75 factor that may be used by the user for such combinations)

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.

g. The program checks the laterally unsupported length of beams to be less than $96r_y$. If the check is not satisfied, it is noted in the output (UBC 2211.7.8).

3. Braced Frames

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

a. The additional requirement of column axial strength as noted in Section F.1.a above for ordinary moment frames also applies to braced frames. However, this requirement is checked even when the axial force level does not exceed $0.3F_y$. The default value for R_w is assumed to be 6.

b. The maximum L/r ratio of the braces is checked not to exceed $720 / \sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2211.8.2.1).

c. The allowable compressive stress for braces is reduced by a factor β, where

$$\beta = 1 / [1 + (kl/r) / (2C_c)] \qquad \text{(UBC 2211.8.2.2)}$$

d. Bracing connection force is reported as the smaller of the tensile strength of the brace (F_yA) and 3/8 R_w times the prescribed seismic forces plus gravity loads (UBC 2211.8.3.1).

e. Chevron braces are designed for 1.5 times the specified loading combinations (UBC 2211.8.4.1). (The program conservatively assumes all braces to be Chevron type unless it finds a real column at each end of the brace either in the story above or below.)

4. Eccentrically Braced Frames

For this framing system, the program looks for and recognizes the eccentrically braced frame configurations shown in Figure III-9. The following additional requirements are checked or reported for the beams, columns and braces associated with these configurations. No additional checks over and above the checks noted in Section E for AISC-ASD89 are made for members not part of the eccentrically braced bay. If these additional members need to be evaluated as special moment resisting frame members or braced frame members the program should be rerun and the framing system appropriately identified.

a. Compact I-shaped beam sections are additionally checked for $b_f / 2t_f$ to be less than $52/\sqrt{F_y}$. If this criteria is satisfied the section is reported as SEISMIC. If this criteria is not satisfied the user must modify the section property (UBC

2211.10.2). Other members meeting this criteria are also reported as SEISMIC.

b. The link beam strength in shear $V_s = 0.55\ F_y\ dt_w$ and moment $M_s = ZF_y$ are calculated. If $V_s \leq 2.2\ M_s / e$ the link beam strength is assumed to be governed by shear and is so reported. If the above condition is not satisfied the link beam strength is assumed to be governed by flexure and is so reported . When link beam strength is governed by shear, the axial and flexural properties (area, A and section modulus, S) for use in the interaction equations are calculated based on beam flanges only (UBC 2211.10.3).

c. The link beam rotation, Θ, (calculated as story drift times bay length divided by total lengths of link beams in the bay) relative to the rest of the beam at a total frame drift of 3/8 R_w times the drift under seismic loads (assumed as the largest of the drifts calculated for Load Conditions A, B, D1 or D2) is checked to be less than the following values (UBC 2211.10.4):

   $\Theta \leq 0.060$ where link beam clear length $\leq 1.6\ M_s / V_s$

   $\Theta \leq 0.015$ where link beam clear length $\geq 3.0\ M_s / V_s$

   and $\Theta \leq$ value interpolated between 0.060 and 0.015 as the link beam clear length varies from $1.6\ M_s / V_s$ to $3.0\ M_s / V_s$.

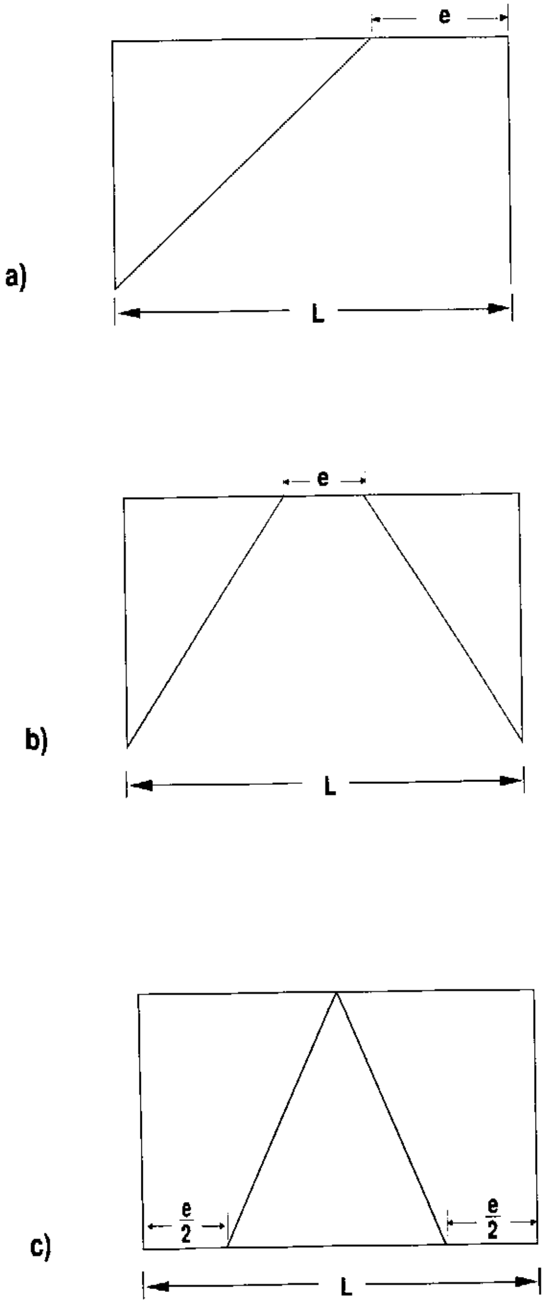
   The value of R_w is defaulted as 12 unless specified by the user.

d. The link beam shear under the specified loading combinations is checked not to exceed 3/4 times 0.8 V_s (UBC 2211.10.5). (The 3/4 factor is to account for the 0.75 factor that may have been used by the user for such combinations.)

e. The brace strength in compression (computed as $P_{sc} = 1.7\ F_aA$) is reported for design of the brace to beam connection (UBC 2211.10.6).

f. The link beam connection shear is reported as equal to the link beam web shear capacity (UBC 2211.10.12).

g. The brace strength in compression (computed in e above) is checked to be at least 1.5 times the axial force corresponding to the controlling link beam strength (UBC 2211.10.13). The controlling link beam strength is either the shear strength, V_s, or the reduced flexural strength, M_{rs}, whichever produces the lower brace force. The value of M_{rs} is taken as $M_{rs} = Z(F_y - f_a)$ (UBC 27211.10.3). Where f_a is the lower of the axial stress in the link beam



Eccentrically Braced Frame Configurations
Figure III-9

corresponding to yielding of the link beam web in shear or the link beam flanges in flexure. The correspondence between brace force and link beam forces is obtained from Load conditions A, B, D1, or D2 whichever has the highest link beam force of interest.

h. The column is checked not to become inelastic for gravity loads plus 1.25 times the column forces corresponding to the controlling link beam strength (UBC 2211.10.14). The controlling link beam strength and the corresponding forces are as obtained by the process described in g above. If this condition governs the interaction ratios reported are calculated as:

$$\frac{P}{1.7AF_a} + \frac{M_x}{S_xF_y} + \frac{M_y}{S_yF_y} \qquad \text{for compressive P}$$

and

$$\frac{P}{AF_y} + \frac{M_x}{S_xF_y} + \frac{M_y}{S_yF_y} \qquad \text{for tensile P}$$

This condition does not govern if the columns meet the additional requirement of column axial strength as noted in Section F.3.a above for braced frames.

i. Axial forces in the beams are included in checking of the beams (UBC 2211.10.17).

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

j. The beam laterally unsupported length is checked to be less than $76\, b_f / \sqrt{F_y}$. If not satisfied it is so noted (UBC 2211.10.18).

k. The continuity plate requirements are checked for a beam flange force of $P_{bf} = A_{fb}\, F_{yb}$.

l. The doubler plate requirements are checked similar to the doubler plate checks for special moment resisting frames as discussed earlier.

Note: The beam strength in flexure, of the beam outside of the link, is **not** currently checked to be at least 1.5 times the moment corresponding to the controlling link beam strength (UBC 2211.10.13). Users need to check for this requirement.

5. Special Concentrically Braced Frames

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

a. The additional requirement of column axial strength as noted in Section F.1.a above for ordinary moment frames also applies to special concentrically braced frames. However, this requirement is checked even when the axial force level does not exceed $0.3F_y$. The default value for R_w is assumed to be 9.

b. The maximum kl/r ratio of the braces is checked not to exceed $1000/\sqrt{F_y}$. If this check is not met, it is noted in the output (UBC 2211.9.2.1).

c. Bracing members are checked to be compact and are so reported. The criteria used is as given in Figure III-6, except for angles b/t is limited to $52/\sqrt{F_y}$, for box sections b_f / t_f and d / t_w is limited to $110/\sqrt{F_y}$, for pipe sections D / t is limited to $1300/F_y$. If this criteria is satisfied the section is reported as COMPACT. If this criteria is not satisfied the user must modify the section property (UBC 2211.9.2.4).

d. Bracing connection force is reported as the smaller of the tensile strength of the brace (F_yA) and 3/8 R_w times the prescribed seismic forces plus gravity loads (UBC 2211.9.3.1).

e. Compact I-shaped column sections are additionally checked for $b_f / 2t_f$ to be less than the numbers given for plastic sections in Figure III-11. Compact box shaped column sections are additionally checked for b_f / t_f and d / t_w to be less than $110/\sqrt{F_y}$. If this criteria is satisfied the section is reported as SEISMIC. If this criteria is not satisfied the user must modify the section property (UBC 2211.9.5).

Note: Beams intersected by Chevron braces are **not** currently checked to have a strength to support loads represented by the following loading combinations (UBC 2211.9.4.1):

$$1.2\,DL + 0.5\,LL \pm P_b$$

$$0.9\,DL \pm P_b$$

where P_b is given by the difference of F_y A for the tension brace and 0.3 times ($1.7F_a$ A) for the compression brace. Users need to check for this requirement.

G. Check/ Design Procedures (AISC Plastic89)

This section defines the capacity check and design procedure used by program STEELER as it applies to the column, beam and brace elements and beam to column joints. The procedure in this section is according to the AISC Plastic89 [Reference 1, Section N].

Various notations used in this section are tabulated in Figure III-10.

First, for each station along the length of the member, for each load combination, the member factored load components (forces and moments) and the corresponding strengths are calculated.

Then, the capacity ratios are evaluated at each station for each member under the influence of each of the design load combinations using the corresponding equations that are defined in this section. The controlling compression and/or tension capacity ratio is output, along with the associated station, load combination and equation. A capacity ratio greater than 1.0 indicates a failure condition. Similarly, a shear capacity ratio is separately output.

The program also evaluates the need for continuity and doubler plates in the columns of moment frames at the beam to column joints and reports the sizes required. Beam connection shears are reported for design of these connections.

1. Evaluation of Capacity Ratios

a. Calculation of Factored Forces and Moments

The factored forces and moments that are calculated for each plastic load combination are

$$P, M_x, M_y, V_x, V_y$$

These factored forces and moments are calculated at each of the previously defined stations corresponding to each element type.

b. Classification of Member

For a member to be designed plastically the section has to meet certain maximum width to thickness ratios and the member laterally unbraced length must not exceed

A	=	Cross-sectional area, in^2
$\mathbf{A_{vx}, A_{vy}}$	=	Major and minor shear areas, in^2
$\mathbf{C_{mx}, C_{my}}$	=	Major and minor direction moment coefficients
E	=	Modulus of elasticity, ksi
$\mathbf{F_a}$	=	Allowable compressive axial stress, ksi
$\mathbf{F_y}$	=	Yield stress of material, ksi
$\mathbf{K_x, K_y}$	=	Effective length K-factors in the major and minor directions
M	=	Lesser of factored major moments at ends of unbraced segment. Positive when member is in double curvature and negative when member is in single curvature, kip-in
$\mathbf{M_{mx}}$	=	$\left(1.07 - \frac{(l_y/r_y)\sqrt{F_y}}{3160}\right) M_{px} \le M_{px}$
$\mathbf{M_{px}, M_{py}}$	=	Major and minor plastic moments, kip-in
$\mathbf{M_x, M_y}$	=	Factored major and minor moments in member, kip-in
P	=	Factored axial force in member, kips
$\mathbf{P_{ex}}$	=	$\pi^2 AE \Big/ \left(K_x \frac{l_x}{r_x}\right)^2$
$\mathbf{P_{ey}}$	=	$\pi^2 AE \Big/ \left(K_y \frac{l_y}{r_y}\right)^2$
$\mathbf{P_{cr}}$	=	Compressive axial strength, kips
$\mathbf{P_y}$	=	Axial force at yield, $\mathbf{AF_y}$, kips
$\mathbf{V_{px}, V_{py}}$	=	Major and minor shear strengths, kips
$\mathbf{V_x, V_y}$	=	Factored major and minor shear forces, kips
$\mathbf{Z_x, Z_y}$	=	Major and minor direction plastic moduli, in^3
b	=	$\mathbf{b_f - 2t_w}$ for welded BOX sections, in
	=	$\mathbf{b_f - 3t_f}$ for rolled BOX (TS) section, in
$\mathbf{b_f}$	=	Flange width, in
d	=	Depth of web, in
$\mathbf{l_x, l_y}$	=	Major and minor direction unbraced member lengths, in
$\mathbf{r_x, r_y}$	=	Radii of gyration in the major and minor directions, in
$\mathbf{t_f}$	=	Flange thickness, in
$\mathbf{t_w}$	=	Thickness of web, in

Notation (AISC Plastic89)
Figure III-10

certain limits. These limits are shown in Figure III-11. If a member satisfies these limits it is classified as PLASTIC and is so reported. If a member does not meet these limits it is classified as NON-PLASTIC, is so reported and a capacity check on it is not made.

c. Calculation of Plastic Capacities

The compressive axial, tensile axial, major and minor bending, and major and minor shear plastic capacities P_{cr}, P_y, M_{px}, M_{py}, V_{px} and V_{py} are evaluated as follows (AISC N4 and N5):

$$P_{cr} = 1.7A\, F_a$$

$$P_y = A\, F_y$$

$$M_{px} = Z_x\, F_y$$

$$M_{py} = Z_y\, F_y$$

$$V_{px} = 0.55\, F_y\, A_{vx}$$

$$V_{py} = 0.55\, F_y\, A_{vy}$$

where F_a is the allowable axial compressive stress and is evaluated as follows:

when $\frac{kl}{r} \leq C_c$,

$$F_a = \frac{\left[1.0 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3\left(\frac{Kl}{r}\right)}{8\,C_c} - \frac{\left(\frac{Kl}{r}\right)^3}{8C_c^3}} \qquad \text{(AISC E2-1)}$$

where $\frac{Kl}{r}$ is the larger of $\frac{K_x\, l_x}{r_x}$ and $\frac{K_y\, l_y}{r_y}$

and $C_c = \sqrt{(2\pi^2 E) / F_y}$

Description of Section	**Ratio Checked**	**Limits for Plastic Member**
I-SHAPES flanges	$b_f/2t_f$	8.5 for $F_y = 36$ 8.0 $= 42$ 7.4 $= 45$ 7.0 $= 50$ 6.6 $= 55$ 6.3 $= 60$ 6.0 $= 65$
webs	d/t_w	$\frac{412}{\sqrt{F_y}}(1 - 1.4\frac{P}{P_y})$ for $\frac{P}{P_y} \le 0.27$ $257/\sqrt{F_y}$ for $\frac{P}{P_y} > 0.27$
unbraced length	l_y/r_y	$\frac{1375}{F_y} + 25$ for $1.0 > \frac{M}{M_{px}} > -0.5$ $\frac{1375}{F_y}$ for $-0.5 \ge \frac{M}{M_{px}} > -1.0$
BOX flanges webs unbraced length	 b/t_f d/t_w l_y/r_y	 $190/\sqrt{F_y}$ as for I-shapes as for I-shapes
CHANNELS	----	Assumed Non-Plastic
T-SHAPES	----	Assumed Non-Plastic
ANGLES or DOUBLE ANGLES	----	Assumed Non-Plastic
PIPES	----	Assumed Non-Plastic
RECT or CIRCLE	----	Assumed Plastic
USER	----	Assumed Non-Plastic

Classification of Sections (AISC Plastic89)
Figure III-11

otherwise, if $Kl/r > C_c$

$$F_a = \frac{12\,\pi^2 E}{23\,(Kl/r)^2} \qquad \text{(AISC E2-2)}$$

For members in compression, if $\frac{Kl}{r}$ is greater than 200, a message to that effect is printed (AISC B7). For members in tension, if l/r is greater than 300, a message to that effect is printed (AISC B7).

d. Calculation of Plastic Capacity Ratios

From the axial and bending plastic capacities and the factored forces and moments at each station, for each of the load combinations, an interaction capacity ratio is produced as given below. The formulas used are extensions to biaxial bending of the formulas given in the code for uniaxial bending.

If P is compressive, the compressive capacity ratio is given by the larger of CR1, CR2 and CR3, where

$$CR1 = \frac{P}{P_{cr}} + \frac{C_{mx} M_x}{\left(1 - \frac{P}{P_{ex}}\right) M_{mx}} + \frac{C_{my} M_y}{\left(1 - \frac{P}{P_{ey}}\right) M_{py}} \qquad \text{(AISC N4-2)}$$

$$CR2 = \frac{P}{P_y} + \frac{M_x}{1.18\,M_{px}} + \frac{M_y}{1.18\,M_{py}} \qquad \text{(AISC N4-3)}$$

$$CR3 = \frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \qquad \text{(BENDING)}$$

Where C_{mx} and C_{my} are coefficients representing distribution of moment along member length and are assumed to be 1.0 for all cases except for columns in unbraced frames when they are taken as 0.85. However, users can specify overriding values.

For elements in the braced frame category, P must not exceed 0.85 P_y (AISC N3.1), and for elements in the moment resisting category, P must not exceed 0.75 P_y (AISC N3.2).

If P is tensile or zero, the tensile stress ratio, TR, is given by

$$TR = \frac{P}{P_y} + \frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \qquad \text{(TENSION)}$$

From the shear capacities and the factored shear force at each station, for each of the load combinations, shear capacity ratios are produced, for each direction, as follows:

$$VR_x = \frac{V_x}{V_{px}} \quad \text{and} \quad VR_y = \frac{V_y}{V_{py}}$$

2. Design of Continuity Plates

The program calculates the required area of continuity plates (see Figure III-7) for AISC Plastic89 similar to the procedure described in Section E.2 for AISC-ASD89, except that the beam flange force, P_{bf} is taken as the yield stress of the beam material times the beam flange area, i.e., F_{yb} A_{bf}.

3. Design of Doubler Plates

The program calculates the required thickness of doubler plates (see Figure III-8) for AISC Plastic89 similar to the procedure described in Section E.2 for AISC-ASD89, except that the shear stress allowable, F_v is taken as $0.55F_y$.

4. 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 shears reported are the maxima of the factored shears obtained from the loading combinations.

H. Check/ Design Procedures (AISC-LRFD93)

This section defines the capacity check and design procedure as it applies to the column, beam and brace elements and beam to column joints. The procedure is according to the AISC-LRFD93 [3].

Various notations used in this section are tabulated in Figure III-12.

First, for each station along the length of the member, for each load combination, the member factored load components (forces and moments) and the corresponding nominal strengths are calculated.

Then, the capacity ratios are evaluated at each station for each member under the influence of each of the design load combinations using the corresponding equations that are defined in this section. The controlling compression and/or tension capacity ratio is output, along with the associated station, load combination and equation. A capacity ratio greater than 1.0 indicates exceedance of a code limit state. Similarly, a shear capacity ratio is separately output.

The program also evaluates the need for continuity and doubler plates in the columns of moment frames at the beam to column joints and reports the sizes required. Beam connection shears are reported for design of these connections.

1. Evaluation of Capacity Ratios

a. Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are P_u, M_{ux}, M_{uy}, V_{ux} and V_{uy} corresponding to factored values of the axial load, the major moment, the minor moment, the major direction shear force and the minor direction shear force, respectively.

These factored loads are calculated at each of the previously defined stations corresponding to each element type.

For loading combinations that cause compression in the member the factored moment M_u (M_{ux} and M_{uy} in the corresponding directions) is magnified to consider second order effects.

A	=	Cross-sectional area, in^2
A_g	=	Gross cross-sectional area, in^2
A_{vx}, A_{vy}	=	Major and minor shear areas, in^2
A_w	=	Shear area, equal **dt_w** per web, in^2
B_1	=	Moment magnification factor for moments not causing sidesway
B_2	=	Moment magnification factor for moments causing sidesway
C_b	=	Bending coefficient
C_m	=	Moment coefficient
C_w	=	Warping constant, in^6
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
F_{cr}	=	Critical compressive stress, ksi
F_r	=	Compressive residual stress in flange assumed 10.0 for rolled sections and 16.5 for welded sections, ksi
F_y	=	Yield stress of material, ksi
G	=	Shear modulus, ksi
I_y	=	Minor moment of inertia, in^4
J	=	Torsional constant for the section, in^4
K	=	Effective length factor
K_x, K_y	=	Effective length K-factors in the major and minor directions
L_b	=	Laterally unbraced length of member, in
L_p	=	Limiting laterally unbraced length for full plastic capacity, in
L_r	=	Limiting laterally unbraced length for inelastic lateral-torsional buckling, in
M_{cr}	=	Elastic buckling moment, kip-in
M_{lt}	=	Factored moments causing sidesway, kip-in
M_{nt}	=	Factored moments not causing sidesway, kip-in
M_{nx}, M_{ny}	=	Nominal bending strength in major and minor directions, kip-in
M_{px}, M_{py}	=	Major and minor plastic moments, kip-in
M_{rx}, M_{ry}	=	Major and minor limiting buckling moments, kip-in
M_u	=	Factored moment in member, kip-in
M_{ux}, M_{uy}	=	Factored major and minor moments in member, kip-in
P_e	=	Euler buckling load, kips
P_n	=	Nominal axial load strength, kips
P_u	=	Factored axial force in member, kips

Notation (AISC-LRFD93)
Figure III-12

P_y	=	A_gF_y, kips
S	=	Section modulus, in^3
S_x, S_y	=	Major and minor section moduli, in^3
V_{nx}, V_{ny}	=	Nominal major and minor shear strengths, kips
V_{ux}, V_{uy}	=	Factored major and minor shear loads, kips
Z	=	Plastic modulus, in^3
Z_x, Z_y	=	Major and minor plastic moduli, in^3
b	=	Nominal dimension of longer leg of angles, in
	=	$b_f - 2t_w$ for welded and $b_f - 3t_f$ for rolled BOX (TS) sections, in
b_f	=	Flange width, in
d	=	Overall depth of member, in
h_c	=	Clear distance between flanges less fillets, in assumed $d - 2k$ for rolled sections and $d - 2t_f$ for welded sections
k	=	Distance from outer face of flange to web toe of fillet , in
k_c	=	$\frac{4}{\sqrt{h/t_w}}$ but between 0.35 and 0.763
l_x, l_y	=	Major and minor direction unbraced member lengths, in
r	=	Radius of gyration, in
r_x, r_y	=	Radii of gyration in the major and minor directions, in
r_z	=	Minimum Radius of gyration for angles, in
t	=	Thickness, in
t_f	=	Flange thickness, in
t_w	=	Thickness of web, in
λ	=	Slenderness parameter
λ_c	=	Column slenderness parameter
λ_p	=	Limiting slenderness parameter for compact element
λ_r	=	Limiting slenderness parameter for non-compact element
ϕ	=	Resistance factor
ϕ_b	=	Resistance factor for bending, 0.9
ϕ_c	=	Resistance factor for compression, 0.85
ϕ_t	=	Resistance factor for tension, 0.9
ϕ_v	=	Resistance factor for shear, 0.9

Notation (AISC-LRFD93)
Figure III-12 (continued)

The magnified moment in a particular direction is given by:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(AISC C1-1)}$$

where

B_1	=	Moment magnification factor for non-sidesway moments
B_2	=	Moment magnification factor for sidesway moments
M_{nt}	=	Factored moments not causing sidesway (assumed due to gravity load conditions I, II, III)
M_{lt}	=	Factored moments causing sidesway (assumed due to lateral load conditions A, B, C, D1 and D2)

The moment magnification factors and moments above all correspond to a particular direction.

The moment magnification factor B_1 for moments not causing sidesway is given by

$$B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1.0 \qquad \text{(AISC C1-2)}$$

where P_e is the Euler buckling load (equal to $\frac{\pi^2 EA}{(Kl/r)^2}$, with $K = 1.0$) and $C_m = 0.6 - 0.4\ M_1/M_2$, where M_1/M_2 is the ratio of the smaller to the larger moment at the ends of the member, M_1/M_2 being positive for double curvature bending and negative for single curvature bending. For beams where transverse load on the member is possible C_m is assumed as 1.0. When M_2 is essentially zero, C_m is taken as 1.0. C_m is also taken as 1.0 for columns disconnected from the diaphragm.

The program defaults C_m to 1.0 if the unbraced length, l_y of the member is overwritten by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of C_m for any member by specifying it.

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

The moment magnification factor B_2 for moments causing sidesway can be taken as 1.0 if a P-Delta analysis is carried out [5]. **The program assumes a P-Delta analysis has been made in ETABS and B_2 for both directions of bending is taken as 1.0.**

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

b. Classification of Sections

The nominal strengths for axial compression and flexure are dependent on the classification of the section as compact, non-compact or slender. The program makes the checks shown in Figure III-13 and reports the classifications for the individual members. If the section dimensions satisfy the limits shown, the section is classified as COMPACT or NON-COMPACT. If the limits for non-compact are not met, the section is classified as SLENDER and the capacity check is terminated.

c. Calculation of Nominal Strengths

i. Compression

The nominal axial compressive strength value, P_n, for compact or non-compact sections, is evaluated as follows:

$$P_n = A_g F_{cr} \qquad \text{(AISC E2-1)}$$

where

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

and

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

and

$$\lambda_c = \frac{Kl}{r\pi} \sqrt{F_y/E} \qquad \text{(AISC E2-4)}$$

where $\frac{Kl}{r}$ is the larger of $\frac{K_x l_x}{r_x}$ and $\frac{K_y l_y}{r_y}$

Note: for single angles r_z is used in place of r_x and r_y.

Description of Section	Ratio Checked (λ)	COMPACT (λ_p)	NON-COMPACT (λ_r)
I-SHAPES flanges (rolled) flanges (welded) webs	$b_f/2t_f$ $b_f/2t_f$ h_c/t_w	$65/\sqrt{F_y}$ $65/\sqrt{F_y}$ for $P_u/\phi_b P_y \le 0.125$ $\frac{640}{\sqrt{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ $\frac{191}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}$	$141/\sqrt{F_y - 10.0}$ $162/\sqrt{(F_y - 16.5)/k_c}$ $970/\sqrt{F_y}$
BOX flanges webs	b/t_f h_c/t_w	$190/\sqrt{F_y}$ as for I-shapes	$238/\sqrt{F_y}$ as for I-shapes
CHANNELS flanges webs	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-SHAPES flanges stem	$b_f/2t_f$ d/t_w	Not applicable Not applicable	$95/\sqrt{F_y}$ $127/\sqrt{F_y}$
ANGLES or DOUBLE ANGLES	b/t	Not applicable	$76/\sqrt{F_y}$
PIPES	D/t	$1300/F_y$	$3300/F_y$
RECT or CIRCLE	—	Assumed compact	—
USER	—	—	Assumed non-compact

Classification of Sections (AISC-LRFD93)
Figure III-13

For members in compression, if $\frac{Kl}{r}$ is greater than 200, a message to that effect is printed (AISC B7).

ii. Tension

The nominal axial tensile strength value P_n is assumed to be A_gF_y (AISC D1-1). It should be noted that no net section checks are made.

For members in tension, if $\frac{l}{r}$ is greater than 300, a message to that effect is printed (AISC B7).

iii. Major and Minor Direction Bending

The nominal bending strengths in the major and minor directions are based on the compactness criteria, the unbraced length of the member and the type of section and are evaluated as follows:

For compact I-shapes, channels, boxes and rectangular bars bent about the major axis,

$$M_{nx} = C_b\, [M_{px} - (M_{px} - M_{rx})\left(\frac{L_b - L_p}{L_r - L_p}\right)] \le M_{px} \qquad \text{(AISC F1-2)}$$

when $L_b \le L_r$

and

$$M_{nx} = M_{crx} \le M_{px} \qquad \text{(AISC F1-12)}$$

when $L_b > L_r$.

Where

M_{nx}	=	Nominal major bending strength
M_{px}	=	Major plastic moment, Z_xF_y
M_{rx}	=	Major limiting buckling moment, $(F_y - F_r)S_x$ for I-shapes, channels and boxes (AISC F1-7) and F_yS_x for rectangular bars (AISC F1-11)
M_{crx}	=	Critical elastic moment,

$$\frac{C_b\pi}{L_b}\sqrt{EI_yGJ + \left(\frac{\pi E}{L_b}\right)^2 I_yC_w}$$

for I-shapes and channels (AISC F1-13)

and $\dfrac{57000C_b\sqrt{JA}}{L_b/r_y}$ for boxes and rectangular bars (AISC F1-14)

L_b = Laterally unbraced length, l_y

L_p = Limiting laterally unbraced length for full plastic capacity,

$\dfrac{300r_y}{\sqrt{F_y}}$ for I-shapes and channels (AISC F1-4)

and $\dfrac{3750r_y}{M_{px}}\sqrt{JA}$ for boxes and rectangular bars (AISC F1-5)

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

$\dfrac{r_yX_1}{(F_y - F_r)}\left(1 + [1 + X_2(F_y - F_r)^2]^{1/2}\right)^{1/2}$

for I-shapes and channels (AISC F1-6)

and $\dfrac{57000r_y\sqrt{JA}}{M_{rx}}$ for boxes and rectangular bars (AISC F1-10)

$X_1 = \dfrac{\pi}{S_x}\sqrt{\dfrac{EGJA}{2}}$ (AISC F1-8)

$X_2 = 4\dfrac{C_w}{I_y}\left(\dfrac{S_x}{GJ}\right)^2$ (AISC F1-9)

$C_b = \dfrac{12.5\ M_{max}}{2.5\ M_{max} + 3\ M_A + 4\ M_B + 3\ M_C}$ (AISC F1-3)

Where M_{max}, M_A, M_B and M_C are absolute values of maximum moment, 1/4 point, center of span and 3/4 point major moments, respectively in the member. C_b is taken as 1.0 for cantilevers, for beams spanning multiple bays and columns disconnected from the diaphragm. The program also defaults C_b to 1.0 if the unbraced length, l_y of the member is overwritten 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.

For compact pipes and circular bars bent about any axis,

$$M_{nx} = M_{ny} = M_p = ZF_y$$

For all compact sections bent about their minor axis,

$$M_{ny} = M_{py} = Z_yF_y$$

For non-compact I-shapes, channels, boxes and pipes the nominal bending strength is computed as the lower of the value given above for compact sections and the values given below for local buckling.

For non-compact I-shapes, channels and boxes the nominal bending strengths are given by the lowest value calculated in the formulas below for the various local buckling modes possible for that section,

$$M_{nx} = M_{px} - (M_{px} - M_{rx})\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{(AISC A-F1-3)}$$

for major direction bending, and

$$M_{ny} = M_{py} - (M_{py} - M_{ry})\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{(AISC A-F1-3)}$$

for minor direction bending.

Where

M_{nx}	=	Nominal major bending strength
M_{ny}	=	Nominal minor bending strength
M_{px}	=	Major plastic moment, Z_xF_y
M_{py}	=	Minor plastic moment, Z_yF_y
M_{rx}	=	Major limiting buckling moment, (AISC Table A-F1.1) $(F_y - F_r)S_x$ for flange buckling of I-shapes and channels, F_yS_x for web buckling of I-shapes and channels and F_yS_x for flange and web buckling of boxes
M_{ry}	=	Minor limiting buckling moment, (AISC Table A-F1.1) F_yS_y for flange buckling of I-shapes, channels and boxes
λ	=	Controlling slenderness parameter (Figure III-13)
λ_p	=	Largest value of λ for which $M_n = M_p$ (Figure III-13)
λ_r	=	Largest value of λ for which buckling is inelastic (Figure III-13)

For non-compact pipe sections the nominal major and minor direction bending strength is

$$M_{nx} = M_{ny} = \left(\frac{600}{D/t} + F_y\right)S \qquad \text{(AISC Table A-F1.1)}$$

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

$$M_{nx} = \frac{\pi\sqrt{EI_y GJ}}{L_b}[B + \sqrt{1+B^2}] \leq F_y S_x \qquad \text{(AISC F1-15)}$$

where

$$B = \pm\ 2.3(d/L_b)\sqrt{I_y/J} \qquad \text{(AISC F1-16)}$$

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

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

$$M_{ny} = F_y Sy$$

For non-compact single angles and for USER sections the nominal major and minor direction bending strengths are assumed as,

$$M_{nx} = F_y Sx \quad \text{and} \quad M_{ny} = F_y Sy$$

iv. Shear

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

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

$$V_{nx} = 0.6 F_y A_w \qquad \text{(AISC F2-1)}$$

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

$$V_{nx} = 0.6\ F_y\ A_w\ (418/\sqrt{F_y})\ /\ (h/t_w) \qquad \text{(AISC F2-2)}$$

and for $\frac{523}{\sqrt{F_y}} < \frac{h}{t_w}$

$$V_{nx} = 132000 A_w\ /\ (h/t_w)^2 \qquad \text{(AISC F2-3 and A-F2-3)}$$

The nominal shear strength for all other sections and for minor direction shears is assumed as:

$$V_{nx} = 0.6F_yA_{vx} \quad \text{and} \quad V_{ny} = 0.6F_yA_{vy}$$

For SLENDER sections and singly symmetric and unsymmetric sections requiring consideration of local buckling (AISC Appendix B5), or flexural-torsional and torsional buckling (AISC Appendix E3), or web buckling (AISC Appendix G) reduced nominal strengths may be applicable. The user must separately investigate this reduction if such elements are used.

If the user specifies allowable bending stress values in the material property data, these values are used to calculate the nominal bending strengths and will override all above mentioned program calculated values. In this case the nominal bending strength in a particular direction is evaluated as the allowable bending stress in that direction times the section modulus in that direction.

d. Calculation of Capacity Ratios

From the factored axial loads and bending moments at each station, for each of the load combinations, and the nominal strengths for axial tension and compression and major and minor bending an interaction capacity ratio is produced as follows:

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

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \qquad \text{(AISC H1-1a)}$$

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

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \qquad \text{(AISC H1-1b)}$$

Where P_n is the nominal axial tensile strength and $\phi = \phi_t = 0.9$ if P_u is tensile; and P_n is the nominal axial compressive strength and $\phi = \phi_c = 0.85$ if P_u is compressive. The resistance factor for bending, $\phi_b = 0.9$.

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

Similarly, from the factored shear force values and the nominal shear strength values at each station, for each of the load combinations, shear capacity ratios for each direction are produced as follows (AISC F2.2):

$$VR_x = \frac{V_{ux}}{\phi_v V_{nx}} \quad \text{and} \quad VR_y = \frac{V_{uy}}{\phi_v V_{ny}}$$

Where $\phi_v = 0.9$.

2. Design of Continuity Plates

The program calculates the required area of continuity plates (see Figure III-7) for AISC-LRFD93 similar to the procedure described in Section E.2 for AISC-ASD89, except that the following algorithms are used.

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

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

$$\phi R_n = (0.9)6.25 t_{fc}^2 F_{yc} \geq P_{bf} \qquad \text{(AISC K1-1)}$$

b. 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.,

$$\phi R_n = (1.0)(5.0k_c + t_{fb}) F_{yc} t_{wc} \geq P_{bf} \qquad \text{(AISC K1-2)}$$

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

$$\phi R_n = (0.75) 68 t_{wc}^2 \left[1 + 3\left(\frac{t_{fb}}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \sqrt{F_{yc} t_{fc} / t_{wc}} \geq P_{bf} \qquad \text{(AISC K1-5a)}$$

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

$$\phi R_n = (0.9)\frac{4100\, t_{wc}^3 \sqrt{F_{yc}}}{d_c} \geq P_{bf} \qquad \text{(AISC K1-8)}$$

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

$$A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{yc})} - 12t_{wc}^2$$

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

The formula above assumes the continuity plate plus a width of web equal to $12t_{wc}$ act as a compression member to resist the applied load (AISC K1.9). The formula also assumes $\phi = 0.85$ and $F_{cr} = 0.9F_{yc}$. This corresponds to an assumption of $\lambda = 0.5$ in the column formulas (AISC 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 (AISC K1.9) the required minimum radius of gyration of the stiffener cross-section would be r = 0.02h to obtain $\lambda = 0.5$ (AISC E2-4).

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 $M_u/(d_b - t_{fb})$
R_n	=	Nominal strength
t_{fb}	=	Beam flange thickness
t_{fc}	=	Column flange thickness
t_{wc}	=	Column web thickness
ϕ	=	Resistance factor

3. Design of Doubler Plates

The program calculates the required thickness of doubler plates (see Figure III-8) for AISC-LRFD93 similar to the procedure described in Section E.2 for AISC-ASD89, except that the following algorithms are used.

The shear force in the panel zone, is given by

$$V_p = P - V_c$$

or

$$V_p = \sum_{n=1}^{nb} \frac{M_{bn}}{d_n - t_{fn}} \cos\theta_n - V_c$$

The nominal panel shear strength is given by

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

and

$$R_v = 0.6F_y d_c t_r\,[1.4 - (P_u/P_y)] \quad \text{for } P_u > 0.4P_y \qquad \text{(AISC K1-10)}$$

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

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

$$t_{dp} = t_r - t_w$$

where

F_y	=	Column and doubler plate yield stress
t_r	=	Required column web thickness
t_{dp}	=	Required doubler plate thickness
t_w	=	Column web thickness
V_p	=	Panel zone shear
V_c	=	Column shear in column above
P	=	Beam flange forces
nb	=	Number of beams connecting to column
d_n	=	Depth of n-th beam connecting to column

θ_n	=	Angle between n-th beam and column major direction
d_c	=	Depth of column clear of fillets, equals $d - 2k$
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_yA

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.

4. 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 shears reported are the maxima of the factored shears obtained from the loading combinations.

I. Check/ Design Procedures (CISC89)

This section defines the capacity check and design procedure as it applies to the column, beam and brace elements and beam to column joints. The procedure is according to the CAN/CSA-S16.1-M89 [4].

Various notations used in this section are tabulated in Figure III-14.

First, for each station along the length of the member, for each load combination, the member factored load components (forces and moments) and the corresponding factored strengths are calculated.

Then, the capacity ratios are evaluated at each station for each member under the influence of each of the design load combinations using the corresponding equations that are defined in this section. The controlling compression and/or tension capacity ratio is output, along with the associated station, load combination and equation. A capacity ratio greater than 1.0 indicates a failure condition. Similarly, a shear capacity ratio is separately output.

A	=	Cross-sectional area, mm^2
$\mathbf{A_g}$	=	Gross cross-sectional area, mm^2
$\mathbf{A_{vx}}$, $\mathbf{A_{vy}}$	=	Major and minor shear areas, mm^2
$\mathbf{A_w}$	=	Shear area, equal $\mathbf{dt_w}$ per web, mm^2
$\mathbf{C_e}$	=	Euler buckling strength, N
$\mathbf{C_f}$	=	Factored compressive axial load, N
$\mathbf{C_r}$	=	Factored compressive axial strength, N
$\mathbf{C_w}$	=	Warping constant, mm^6
$\mathbf{C_y}$	=	Compressive axial load at yield stress, $\mathbf{A_gF_y}$, N
D	=	Outside diameter of pipes, mm
E	=	Modulus of elasticity, MPa
$\mathbf{F_y}$	=	Specified minimum Yield stress, MPa
G	=	Shear modulus, MPa
$\mathbf{I_y}$	=	Minor moment of inertia, mm^4
J	=	Torsional constant for the section, mm^4
K	=	Effective length factor
$\mathbf{K_x}$, $\mathbf{K_y}$	=	Effective length K-factors in the major and minor directions (assumed as 1.0 for this code unless overwritten by user)
L	=	Laterally unbraced length of member, mm
$\mathbf{M_{fx}}$, $\mathbf{M_{fy}}$	=	Factored major and minor bending loads, N mm
$\mathbf{M_{px}}$, $\mathbf{M_{py}}$	=	Major and minor plastic moments, N mm
$\mathbf{M_{rx}}$, $\mathbf{M_{ry}}$	=	Factored major and minor bending strengths, N mm
$\mathbf{M_u}$		Critical elastic moment, N mm
$\mathbf{M_{yx}}$, $\mathbf{M_{yy}}$	=	Major and minor yield moments, N mm
$\mathbf{S_x}$, $\mathbf{S_y}$	=	Major and minor section moduli, mm^3
$\mathbf{T_f}$	=	Factored tensile axial load, N
$\mathbf{T_r}$	=	Factored tensile axial strength, N
$\mathbf{U_1}$		Moment magnification factor to account for deformation of member between ends
$\mathbf{U_2}$		Moment magnification factor (on sidesway moments) to account for P- Delta
$\mathbf{V_{fx}}$, $\mathbf{V_{fy}}$	=	Factored major and minor shear loads, N
$\mathbf{V_{rx}}$, $\mathbf{V_{ry}}$	=	Factored major and minor shear strengths, N
$\mathbf{Z_x}$, $\mathbf{Z_y}$	=	Major and minor plastic moduli, mm^3 .
b	=	Nominal dimension of longer leg of angles
	=	$\mathbf{b_f - 2t_w}$ for welded $\mathbf{b_f - 3t_f}$ for rolled BOX (TS) sections, mm

Notation (CISC89)
Figure III-14

b_f	=	Flange width, mm
d	=	Overall depth of member, mm
h	=	Clear distance between flanges , taken as $d - 2t_f$, mm
k	=	Web plate buckling coefficient, assumed as 5.34 (no stiffeners)
k	=	Distance from outer face of flange to web toe of fillet , mm
l	=	Unbraced length of member, mm
l_x, l_y	=	Major and minor direction unbraced member lengths, mm
r	=	Radius of gyration, mm
r_x, r_y	=	Radii of gyration in the major and minor directions, mm
r_z	=	Minimum Radius of gyration for angles, mm
t	=	Thickness, mm
t_f	=	Flange thickness, mm
t_w	=	Thickness of web, mm
λ	=	Slenderness parameter
ϕ	=	Resistance factor, taken as 0.9
ω_1	=	Moment Coefficient
ω_{1x}, ω_{1y}	=	Major and minor direction moment coefficients
ω_2	=	Bending coefficient

Notation (CISC89)
Figure III-14 (continued)

The program also evaluates the need for continuity and doubler plates in the columns of moment frames at the beam to column joints and reports the sizes required. Beam connection shears are reported for design of these connections.

All equations and descriptions presented in this section correspond to Newton millimeter units.

1. Evaluation of Capacity Ratios

a. Calculation of Factored Forces and Moments

The factored member loads that are calculated for each load combination are T_f or C_f, M_{fx}, M_{fy}, V_{fx} and V_{fy} corresponding to factored values of the tensile or compressive axial load, the major moment, the minor moment, the major direction shear load and the minor direction shear load, respectively.

These factored loads are calculated at each of the previously defined stations corresponding to each element type.

The program assumes that a P-Delta analysis has been made in ETABS and therefore any magnification of sidesway moments due to second order effects is already accounted for, i.e. U_2 in CISC 8.6.1.(a) is taken as 1.0 [4]. However, the user can overwrite the values of U_2 for both major and minor direction bending. In this case M_f in a particular direction is taken as:

$$M_f = M_{fg} + U_2 M_{ft}$$

where

U2	=	Moment magnification factor for translational moments
M_{fg}	=	Factored moments not causing translation (assumed due to gravity load conditions I, II, III)
M_{ft}	=	Factored moments causing sidesway (assumed due to lateral load conditions A, B, C, D1, and D2)

b. Classification of Sections

The nominal strengths for axial compression and flexure are dependent on the classification of the section as Class 1 (plastic), Class 2 (compact), Class 3 (non-compact) or Class 4 (slender). The program makes the checks shown in Figure III-15 and reports the classifications for the individual members. If the section

Description of Section	Ratio Checked	Class 1 (Plastic)	Class 2 (Compact)	Class 3 (Non-compact)
I-SHAPES flanges webs	$b_f/2t_f$ h/t_w	$145/\sqrt{F_y}$ $\frac{1100}{\sqrt{F_y}}\left(1-.39\frac{C_f}{C_y}\right)$	$170/\sqrt{F_y}$ $\frac{1700}{\sqrt{F_y}}\left(1-.61\frac{C_f}{C_y}\right)$	$200/\sqrt{F_y}$ $\frac{1900}{\sqrt{F_y}}\left(1-.65\frac{C_f}{C_y}\right)$
BOX flanges webs	b/t_f h/t_w	$420/\sqrt{F_y}$ as for I-shapes	$525/\sqrt{F_y}$ as for I-shapes	$670/\sqrt{F_y}$ as for I-shapes
CHANNELS flanges webs	b_f/t_f h/t_w	Not applicable Not applicable	Not applicable Not applicable	$200/\sqrt{F_y}$ as for I-shapes
T-SHAPES flanges stem	$b_f/2t_f$ d/t_w	Not applicable Not applicable	Not applicable Not applicable	$200/\sqrt{F_y}$ $340/\sqrt{F_y}$
ANGLES or DOUBLE ANGLES	b/t	Not applicable	Not applicable	$200/\sqrt{F_y}$
PIPES	D/t	$13000/F_y$	$18000/F_y$	$23000/F_y$
RECT or CIRCLE	—	—	Assumed Class 2	—
USER	—	—	—	Assumed Class 3

Classification of Sections (CISC89)
Figure III-15

dimensions satisfy the limits shown in Figure III-15, the section is classified as Class 1, Class 2 or Class 3 as applicable. If the limits for Class 3 are not met, the section is classified as Class 4 and the capacity check is terminated.

c. Calculation of Factored Strengths

The factored strengths for axial load, bending and shear are calculated as described below. The strength reduction factor, ϕ is taken as 0.9 (CISC 13.1)

i. Compression

The factored axial compressive strength value, C_r, for class 1,2 or 3 sections, is evaluated as follows (CISC 13.3.1):

$\lambda \leq 0.15$	$C_r = \phi A F_y$
$0.15 < \lambda \leq 1.0$	$C_r = \phi A F_y (1.035 - .202\,\lambda - .222\,\lambda^2)$
$1.0 < \lambda \leq 2.0$	$C_r = \phi A F_y (-.111 + .636\lambda^{-1} + .087\,\lambda^{-2})$
$2.0 < \lambda \leq 3.6$	$C_r = \phi A F_y (.009 + .877\,\lambda^{-2})$
$3.6 < \lambda$	$C_r = \phi A F_y\,\lambda^{-2}$

where

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

where $\frac{Kl}{r}$ is the larger of $\frac{K_x\, l_x}{r_x}$ and $\frac{K_y\, l_y}{r_y}$

Note: for single angles r_z is used in place of r_x and r_y.

The above calculated value of C_r is conservative for hot-formed or stress-relieved hollow sections where the code allows a higher value (CISC 13.3.2).

For members in compression, if $\frac{Kl}{r}$ is greater than 200, a message to that effect is printed (CISC 10.2.1).

ii. Tension

The factored axial tensile strength value, T_r, is assumed to be $\phi A_g F_y$ (CISC 13.2.(a).(i)).

For members in tension, if $\frac{l}{r}$ is greater than 300, a message to that effect is printed (CISC 10.2.2).

iii. Major and Minor Direction Bending

The factored bending strength in the major and minor directions are based on the section classification, the unbraced length of the member and the type of section and are evaluated as follows (CISC 13.6):

For Class 1 and 2 sections of I-shapes, boxes and rectangular bars bent about the major axis, and for pipes and circular rods bent about any axis

$$M_{rx} = 1.15\,\phi\,\pi\,M_{px}\left(1 - 0.28\,\frac{M_{px}}{M_u}\right) \leq \phi\,M_{px}$$

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

and

$$M_{rx} = \phi\,M_u$$

when $M_u \leq 0.67\,M_{px}$.

where

M_{rx}	=	Factored major bending strength
M_{px}	=	Major plastic moment, Z_xF_y
M_u	=	Critical elastic moment, $\frac{\omega_2\,\pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_yC_w}$
L	=	Laterally unbraced length, l_y
C_w	=	Warping constant assumed as 0.0 for boxes, pipes, rectangular bars and circular rods
ω_2	=	$1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.5$

M_1 and M_2 are end moments of the unbraced segment and M_1 is less than M_2; $\left(\frac{M_1}{M_2}\right)$ being positive for double curvature bending and negative for single curvature bending. Also, if any moment within the segment is greater than M_2, ω_2 is taken as

1.0. ω_2 is also taken as 1.0 for cantilevers, for beams spanning multiple bays and for columns disconnected from the diaphragm.

The program defaults ω_2 to 1.0 if the unbraced length, l_y of the member is overwritten by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of ω_2 for any member by specifying it.

For Class 1 and 2 sections of I-shapes, boxes and rectangular bars bent about their minor axis,

$$M_{ry} = \phi\, M_{py} = \phi\, Z_y F_y$$

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

$$M_{rx} = 1.15\, \phi\, M_{yx} \left(1 - 0.28\, \frac{M_{yx}}{M_u}\right) \leq \phi\, M_{yx}$$

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

and

$$M_{rx} = \phi\, M_u$$

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

where

M_{rx} and M_u are as defined earlier for Class 1 and 2 sections and
M_{yx} = Major Yield Moment, $S_x F_y$

For Class 3 sections of I-shapes, channels and boxes bent about their minor axis,

$$M_{ry} = M_{yy} = S_y F_y$$

For Class 3 sections of T-shapes and double angles the factored major bending strength is given as,

$$M_{rx} = \phi\, \frac{\omega_2 \pi \sqrt{E I_y G J}}{L} [B + \sqrt{1 + B^2}] \leq \phi\, F_y S_x$$

where

$$B = \pm\ 2.3(d/L)\sqrt{I_y/J}$$

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

For Class 3 sections of T-shapes and double angles the factored minor bending strength is assumed as,

$$M_{ry} = \phi\ F_y S_y$$

For class 3 single angles and for USER sections the factored major and minor direction bending strengths are assumed as,

$$M_{rx} = \phi\ F_y S_x \quad \text{and} \quad M_{ry} = \phi\ F_y S_y$$

iv. Shear

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

$$\frac{h}{t_w} \leq 439\sqrt{\frac{k}{F_y}} \qquad V_{rx} = \phi A_w\,(0.66F_y)$$

$$439\sqrt{\frac{k}{F_y}} < \frac{h}{t_w} \leq 621\sqrt{\frac{k}{F_y}} \qquad V_{rx} = \phi A_w\, 290\,\frac{\sqrt{kF_y}}{h/t_w}$$

$$\frac{h}{t_w} > 621\sqrt{\frac{k}{F_y}} \qquad V_{rx} = \phi A_w\,\frac{180000k}{(h/t_w)^2}$$

where k is assumed to be 5.34 (no stiffeners).

The factored shear strength for all other sections and for minor direction shears in I-shapes, boxes and channels is assumed as (CISC 13.4.2):

$$V_{rx} = 0.66\,\phi F_y A_{vx} \qquad \text{and} \qquad V_{ry} = 0.66\phi F_y A_{vy}$$

For Class 4 (slender) sections and singly symmetric and unsymmetric sections requiring consideration of local buckling, or flexural-torsional and torsional buckling, or web buckling, reduced nominal strengths may be applicable. The user must separately investigate this reduction if such elements are used.

If the user specifies allowable bending stress values in the material property data, these values are used to calculate the factored bending strengths and will override all above mentioned program calculated values. In this case the factored bending strength in a particular direction is evaluated as the allowable bending stress in that direction times the section modulus in that direction times a ϕ of 0.90.

d. Calculation of Capacity Ratios

From the factored axial loads and bending moments at each station, for each of the load combinations, and the factored strengths for axial tension and compression and major and minor bending an interaction capacity ratio is produced as follows:

If the axial load is compressive the capacity ratio is given by:

$$\frac{C_f}{C_r} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{M_{ry}} \qquad \text{(CISC 13.8.1)}$$

for all sections except Class 1 I-shaped sections.

For Class 1 I-shaped sections the capacity is given by:

$$\frac{C_f}{C_r} + 0.85 \frac{U_{1x} M_{fx}}{M_{rx}} + 0.6 \frac{U_{1y} M_{fy}}{M_{ry}} \qquad \text{(CISC 13.8.2)}$$

The above ratios are calculated three times for the following conditions and the worst ratio reported:

a. Cross-sectional strengths, assumes

$C_r = \phi A F_y$
$U_{1x} = U_{1y} = 1.0$
M_{rx} and M_{ry} are calculated based on unbraced length of 0.0.

b. Overall member strength, assumes

M_{rx} and M_{ry} are calculated based on unbraced length of 0.0.

c. Lateral torsional buckling strength, assumes

C_r is calculated based on $\frac{K_y l_y}{r_y}$ only.

For Class 1 I-shapes the following ratio is also checked:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}}$$

In the above equations:

$$U_1 = \frac{\omega_1}{1 - \frac{C_f}{C_e}}$$

Where $C_e = \dfrac{\pi^2 E I}{L^2}$

and $\omega_1 = 0.6 - 0.4\, M_1/M_2 \geq 0.4$

Where M_1/M_2 is the ratio of the smaller to the larger moment at the ends of the member, M_1/M_2 being positive for double curvature bending and negative for single curvature bending. For beams where transverse load on the member is possible, ω_1 is assumed as 1.0. When M_2 is essentially zero, ω_1 is taken as 1.0. ω_1 is also taken as 1.0 for columns disconnected from the diaphragm.

The program defaults ω_1 to 1.0 if the unbraced length, l_y of the member is overwritten by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of ω_1 for any member by specifying it.

The factor U_1 must be a positive number. Therefore C_f must be less than C_e. If this is not true, a failure condition is declared.

If the axial load is tensile the capacity ratio is given by the larger of:

$$\frac{T_f}{T_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \qquad \text{(TENSION)}$$

assuming M_{rx} and M_{ry} are calculated based on unbraced length of 0.0

and

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} - \frac{T_f Z_x}{M_{rx} A} \text{ for Class 1 and 2 sections} \qquad \text{(BENDING)}$$

or

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} - \frac{T_f\,S_x}{M_{rx}\,A} \text{ for Class 3 sections} \qquad \text{(BENDING)}$$

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

Similarly, from the factored shear force values and the factored shear strength values at each station, for each of the load combinations, shear capacity ratios for each direction are produced as follows:

$$VR_x = \frac{V_{fx}}{V_{rx}} \quad \text{and} \quad VR_y = \frac{V_{fy}}{V_{ry}}$$

2. Design of Continuity Plates

The program calculates the required area of continuity plates (see Figure III-7) for CISC89 similar to the procedure described in Section E.2 for AISC-ASD89, except that the following algorithms are used.

The program first evaluates the need for continuity plates. Continuity plates will be required if either of the following two conditions is not satisfied:

$$T_r = 7\,\phi t_{fc}^2 F_{yc} \geq \frac{M_f}{d_b} \qquad \text{(CISC 21.3.(b))}$$

and

$$B_r = \phi\, t_{wc}\,(t_{fb} + 5k_c)F_{yc} \geq \frac{M_f}{d_b} \qquad \text{(CISC 21.3.(a))}$$

or for columns with Class 3 webs

$$B_r = \phi\, \frac{640{,}000}{(h_c/t_{wc})^2} t_{wc}\,(t_{fb} + 5k_c) \geq \frac{M_f}{d_b}$$

If either of the conditions above are not met the program calculates the required continuity plate area to resist the larger of the following two force values:

$$F_{st} = \left(\frac{M_f}{d_b}\right) - B_r \qquad \text{and} \qquad F_{st} = \left(\frac{M_f}{d_b}\right) - T_r$$

The required area of continuity plate is then calculated as

$$A_{cp} = \frac{F_{st}}{\phi\,(0.9)F_{yc}}$$

The formula assumes $F_c = 0.9\ F_{yc}$. This corresponds to an assumption of $\lambda = 0.45$ in the column formulas (CISC 13.3.1). The user should choose the continuity plate cross-section such that this is satisfied.

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
h_c	=	Clear distance between flanges of column less fillets
k_c	=	Distance from outer face of flange to web toe of fillet
M_f	=	Factored beam moment
t_{fb}	=	Beam flange thickness
t_{fc}	=	Column flange thickness
t_{wc}	=	Column web thickness
ϕ	=	Resistance factor, taken as 0.9

3. Design of Doubler Plates

The program calculates the required thickness of doubler plates (see Figure III-8) for CISC89 similar to the procedure described in Section E.2 for AISC-ASD89, except that the following algorithms are used.

The shear force in the panel zone is given by

$$V_p = P - V_c$$

or

$$V_p = \sum_{n=1}^{nb} \frac{M_{bn}}{d_n} \cos \theta_n \; - \; V_c$$

The factored panel shear strength is given by (CISC 21.3)

$$V_r = 0.55\ \phi F_y d_c t_r$$

By using $V_p = V_r$ the required column web thickness t_r can be found.

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

$$t_{dp} = t_r - t_w$$

where

F_y	=	Column and doubler plate yield stress
t_r	=	Required column web thickness
t_{dp}	=	Required doubler plate thickness
t_w	=	Column web thickness
V_p	=	Panel zone shear
V_c	=	Column shear in column above
V_r	=	Panel zone factored shear strength
P	=	Beam flange forces
nb	=	Number of beams connecting to column
d_n	=	Depth of n-th beam connecting to column
θ_n	=	Angle between n-th beam and column major direction
d_c	=	Depth of column
M_{bn}	=	Calculated factored beam moment from the corresponding loading combination

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

4. 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 shears reported are the maxima of the factored shears obtained from the loading combinations.

Chapter IV

STEELER Input Data File

In order to execute the STEELER program, an ETABS post processing file and a STEELER input data file are required. **However, if the program defaults are acceptable, the STEELER input data is optional**. The program defaults to the UBC94 code and special moment resisting frames.

The ETABS post processing file contains information pertaining to the structural geometry and loading and the analytical results from the corresponding ETABS analysis. This file forms the interface between ETABS and the STEELER post processor. Only those elements with material properties of S (Steel) will be processed by STEELER.

The user must read and understand the contents of Chapter III, before proceeding with the data preparation described in this section.

The user should also be thoroughly familiar with the STEELER control variables described below, and with the main control variables of ETABS. Repeated references are made to these variables throughout this chapter.

The user is reminded of the following items which must be considered in preparing the ETABS input when it will be post processed by STEELER.

- For the built in loading combinations when using any code the STEELER program assumes that dead load is specified in ETABS Load Condition I, live load is specified in Load Condition II and the lateral loads are in Load Conditions A, B, C, D1, and D2. The user must account for this when preparing the ETABS data. All scaling of loads (for example, dynamic response spectrum loads) should be done in the ETABS eight basic load conditions.

- For the AISC-LRFD93 and the CISC89 specifications the STEELER program assumes that a P-Delta analysis has been made in ETABS. That is, the bending moments causing sidesway do not have to be further amplified by moment magnification factors. The ETABS program uses the specified story masses to calculate the vertical load, P to account for the P-Delta effects. A factor is provided to increase this effect to account for factored vertical loads and for inelastic displacements. The user should use this factor to account for factored dead and live loads due to which P-Delta effect should be accounted for in steel design. The STEELER program also assumes that the ETABS Load Conditions I, II and III are used for gravity loads which do not cause significant sidesway and the remaining load conditions are used for lateral loads. If the above assumptions of the program are not correct for a particular problem, the user must provide over-riding moment magnification factors in the STEELER data.

- To automatically determine bracing configurations (Chevron, concentric, eccentric) and unbraced beam length the program differentiates between real columns and dummy columns. It is necessary that dummy column locations not have real property numbers even if the section properties themselves are specified as zeros.

- All input data for ETABS and STEELER must be prepared using the same set of consistent units. Either English (Kip-inch) or MKS metric (Kilogramforce-meter) or SI metric (KiloNewton-meter) units are possible. The type of unit used is specified in the ETABS input data. This is irrespective of the fact that all the numerical techniques described in Chapter III are presented in inch-kip-second units or millimeter-Newton-second units.

All input data for STEELER is prepared in free format form similar to the ETABS input data. The formating rules described for the preparation of the ETABS input data (Chapter V of the ETABS User's Manual) also apply to the STEELER input data.

Data Block	When Needed
1. Main Control Data	Always
2. Load Combination Data	Only if **NLC** > 0
3. Material Property Redefinition Data	Only if **NRMP** > 0
4. Section Property Redefinition Data	
i. Column Properties	Only if **NRCP** > 0
ii. Beam Properties	Only if **NRBP** > 0
iii. Brace Properties	Only if **NRBRP** > 0
5. Frame Stress Check Activation Data	
i. Frame Control Data	Only if **NFR** > 0
ii. Element Reassignment Data	
a. Column	Only if **IRCP** > 0
b. Beam	Only if **IRBP** > 0
c Brace	Only if **IRBRP** > 0

Typical Data Setup for STEELER
Figure IV-1

There are basically five data sections associated with the STEELER input. A summary of the data setup is shown in Figure IV-1. The sequence of data lines described herein will establish the data file required by STEELER.

The following is the convention used in this chapter to define each data line:

First, the format giving the sequence of the entries of each data line is presented as a series of abbreviations of the options (or variables).

Each data section is then followed by a tabular description in the form:

Variable **Field** **Note** **Entry**

The **Variable** is the abbreviation of the entry made on the data line.

The **Field** is a number that corresponds to the sequence in which the variable exists on the data line. Thus if a variable is the fourth entry on a data line, it will have a field number of 4.

The **Note** number refers to the series of notes that exist at the end of the corresponding data section. The notes describe the data options in more detail and give important information to aid the user in better understanding the options of the program.

The **Entry** is a brief description of the option.

1. Main Control Data

Prepare thc following data as defined in Sections a, b, c and d below. This data is always needed. A total of 5 data lines are required.

Format

a. Program Name and Version

Prepare one line of data to give the program name and version as follows:

STEELER 6.1

b. Heading Data

Prepare two lines of data for output labeling, up to 70 characters per line. This information will appear on every page.

c. Execution Control Data

Prepare one line of data to define the program execution options in the following form:

ICODE NFR NLC LLC NRMP NRCP NRBP NRBRP PROPFILE

d. Output Control Data

Prepare one line of data to define the program output options in the following form:

RI RS IDC IDB IDBR ISC ISB ISBR MCI MBI MCC MCD

If no STEELER input file is provided, the **Heading Data** defaults to the **Heading Data** of ETABS; the **ICODE** parameter defaults to 1 (i.e. the UBC94 code for steel); **NFR** defaults to designing all frames; **NLC** defaults to using the loading combination defaults given in Section 2 later; **LLC** defaults to 2; **NRMP**, **NRCP**, **NRBP** and **NRBRP** all default to zero; **RI** and **RS** default to 0.005; **IDC**, **IDB**, **IDBR**, **ISC**, **ISB**, **ISBR**, **MCI**, **MBI**, **MCC**, and **MCD** all default to 1.

Description

Variable	Field	Note	Entry
c. Execution Control Data			
ICODE	1	(1)	Code Identifier: = 1 UBC94 (includes seismic) = 2 AISC-ASD89 = 3 AISC Plastic89 = 4 AISC-LRFD93 = 5 CISC89
NFR	2	(2)	Number of frames to be checked.
NLC	3	(3)	Number of stress check load combinations.
LLC	4	(4)	ETABS load condition number that corresponds to live load: = 1 Vertical load condition I = 2 Vertical load condition II = 3 Vertical load condition III
NRMP	5	(5,6)	Number of redefined (or new) material property types.
NRCP	6	(5,7)	Number of redefined (or new) column section property types.
NRBP	7	(5,8)	Number of redefined (or new) beam section property types.
NRBRP	8	(5,9)	Number of redefined (or new) brace section property types.
PROPFILE	9	(10)	User section property filename.

Variable	Field	Note	Entry

d. Output Control Data

Variable	Field	Note	Entry
RI	1	(11)	Interaction stress ratio cutoff.
RS	2		Shear stress ratio cutoff.
IDC	3	(12)	Flag for column interaction detail information.
IDB	4		Flag for beam interaction detail information.
IDBR	5		Flag for brace interaction detail information.
ISC	6	(13)	Flag for column shear stress check
ISB	7		Flag for beam shear stress check.
ISBR	8		Flag for brace shear stress check.
MCI	9	(14)	Flag for map of column stress ratios.
MBI	10		Flag for map of beam stress ratios.
MCC	11		Flag for map of column continuity plates.
MCD	12		Flag for map of column doubler plates.

Notes

1. The STEELER program has options to check the steel frame with respect to several different codes. This option allows the user to choose the code to be used. Refer to Chapter III for details of the check/ design algorithms for the various codes. The loading combinations specified in Section 2 later must correspond to the design level at which the check is made.

   It is re-iterated here that only the design checks listed in Chapter III are performed by the STEELER program. Other significant aspects of steel frame design, for example serviceability, detailing, connections, etc., are not addressed by the program and should be addressed separately by the user.

2. This variable defines the number of frame stress activation data sets to be provided in Section 5 below. If this number is zero, no data is expected or read in Section 5, but all frames are designed with default values.

3. This variable defines the number of design load combinations and controls the number of data lines to be read in Section 2 below. If this number is zero, no data is expected or read in Section 2, but default values of loading combinations listed in Section 2 are used.

4. This entry defines the vertical load condition of ETABS that corresponds to the live load. The live load reduction factors are then applied to the member forces associated with this load condition, before they are summed into the combinations.

5. It is possible to redefine or add new material properties or section properties in the material and section property tables that the user originally defined in the ETABS data.

   Via this option, the user can modify the steel frame and make stress check iteration runs with STEELER without rerunning the analysis runs of ETABS. After the stress check is satisfactory, the user must then incorporate the changes into the ETABS data and rerun analysis and stress check runs to final convergence.

6. The entry **NRMP** defines the number of material property sets that are defined in Section 3 below.

7. The entry **NRCP** defines the number of column section property sets that are defined in Section 4.i. below.

8. The entry **NRBP** defines the number of beam section property sets that are defined in Section 4.ii. below.

9. The entry **NRBRP** defines the number of brace section property sets that are defined in Section 4.iii. below.

10. The program has the ability to extract section property information from a database. The program always searches for the section labels first in a file named SECTIONS.PRO. The user also can specify one additional file to be searched if a section label is not found in file SECTIONS.PRO. **PROPFILE** is the name of this additional file. **PROPFILE** is a filename (maximum 8 characters) with a .PRO extension. Both the file SECTIONS.PRO and the one specified through **PROPFILE** must be prepared using the ETABS utility program PROPER. Both these files must be present either in the directory of the input file or in the directory where the program STEELER resides. If a **PROPFILE** is used in the ETABS run the file used in STEELER must be identical.

11. In general, stress information on elements with low stress ratios is not critical. The entries **RI** and **RS** allow the user to suppress such unimportant stress information, thereby reducing the size of the output files. The value of **RI** is a threshold value for interaction stress ratios. The program will produce interaction stress information for only those elements which have an interaction stress ratio greater than or equal to **RI**. Similarly, **RS** is a threshold value for shear stress ratios. The program will produce shear stress ratio information for only those elements which have a shear stress ratio greater than or equal to **RS**.

12. If this entry is 0, the program will not produce detailed backup information associated with the member interaction stress ratio calculation for this element type. If this entry is 1, the backup information will exist in the CHKSTL.DTL output file as described in Chapter V.

13. If this entry is 0, no shear stress check will be performed for this element type. If this entry is 1, the shear stress check will be implemented and the corresponding stress check information will appear in the CHKSTL.SHR output file as described in Chapter V.

14. If this entry is 0, the program will not produce display maps associated with this parameter. If this entry is 1, display maps will be created and will exist in the CHKSTL.MAP output file as described in Chapter V.

2. Load Combination Data

Load combinations for the building are defined as summations of the eight basic load conditions, namely:

- The vertical static load conditions I, II and III.
- The lateral static load conditions A, B and C.
- The lateral dynamic load conditions D1 and D2.

The data provided in this data section is completely independent of the load case data that is provided in the corresponding ETABS analysis run.

Format

Provide one data line to define each of the **NLC** load combinations in the following form:

L LTYP XI XII XIII XA XB XC XD1 XD2

Description

Variable	Field	Note	Entry
L	1	(1)	Load combination number.
LTYP	2	(2)	Load combination type: = 0 Linear combination, consider all signs. = 1 Linear combination, use absolute value of responses, but consider sign of multipliers. = 2 SRSS A and B load conditions, combine linearly with others. = 3 SRSS D1 and D2 load conditions, combine linearly with others.
XI	3	(3)	Multiplier for vertical static load condition I.
XII	4		Multiplier for vertical static load condition II
XIII	5		Multiplier for vertical static load condition III.
XA	6		Multiplier for lateral static load condition A.
XB	7		Multiplier for lateral static load condition B.
XC	8		Multiplier for lateral static load condition C.
XD1	9		Multiplier for lateral dynamic load condition D1.
XD2	10		Multiplier for lateral dynamic load condition D2.

Notes

1. This number must be in ascending consecutive numerical sequence starting with the number 1.

2. If this entry is zero, linear combinations are produced and all signs are considered.

   If this entry is 1, linear combinations are produced, except that absolute values of responses are used, although signs of multipliers are considered. This type of combination is not recommended for STEELER. It has been kept here for consistency with ETABS.

   If this entry is 2, a square root of the sum of the squares (SRSS) combination of the Load Conditions A and B responses with the specified multipliers is first made, before combining linearly with the other load conditions. The SRSS value is assigned the sign of **XA**. This type of combination is required in some design codes to consider orthogonal effects of seismic excitations.

   If this entry is 3, a SRSS combination of the Load Conditions D1 and D2 responses with the specified multiplier is first made before combining linearly with the other load conditions. The SRSS value is assigned the sign of **XD1**. This type of combination is commonly used for dynamic analysis and is required by some design codes to consider orthogonal effects of seismic excitations.

3. Each member is checked for each of the specified load combinations. The compression or tension stress ratios from corresponding controlling load combinations are reported.

   Typically, structures are subjected to vertical loads due to dead and live loads which act downwards. In addition to the vertical loads, the structures are usually subjected to lateral loads, resulting from wind or seismic forces, which act along different directions (usually assumed to be in two mutually orthogonal directions), and the directions are reversible.

   Therefore, when using UBC94, the required loading combinations, if the structure is subjected to dead load (DL) and live load (LL) only, will be,

   DL + LL

   as dead load and live load are not reversible.

However, if in addition to the dead load (DL) and the live load (LL) the structure is subjected to seismic loads from two mutually perpendicular reversible directions and accidental torsion (EX, EY, ET), the user needs to specify the following load combinations:

1. DL + LL
2. $0.75\,(DL + LL + \sqrt{EX^2 + EY^2} + ET)$
3. $0.75\,(DL + LL + \sqrt{EX^2 + EY^2} - ET)$
4. $0.75\,(DL + LL - \sqrt{EX^2 + EY^2} + ET)$
5. $0.75\,(DL + LL - \sqrt{EX^2 + EY^2} - ET)$
6. $0.75\,(DL + \sqrt{EX^2 + EY^2} + ET)$
7. $0.75\,(DL + \sqrt{EX^2 + EY^2} - ET)$
8. $0.75\,(DL - \sqrt{EX^2 + EY^2} + ET)$
9. $0.75\,(DL - \sqrt{EX^2 + EY^2} - ET)$

The 0.75 factor is for the one third increase in the allowable stresses, so that all stress ratios are compared against unity while checking for overstress.

These are the program defaults whenever the UBC94 code is used, assuming DL is gravity Load Condition I, LL is gravity Load Condition II, EX is static lateral Load Condition A, EY is static lateral Load Condition B and ET is static lateral Load Condition C.

The loading combination defaults for other codes would be different and the user is referred to Section A of Chapter III.

3. Material Property Redefinition Data

This data section is only needed if the material property table that has been previously defined in the ETABS data is to be modified (or expanded).

If **NRMP** is 0, this data section is not needed and must be skipped. In this case, the values of the material properties used by STEELER will retain values that were assigned in the ETABS data. However, if for any S material type the value of **FY** has not been defined, it will be set to the default value defined herein (Note 4 below) irrespective of whether any material properties are being redefined.

If **NRMP** is not 0, provide **NRMP** data lines to redefine material property types that were previously defined in the ETABS data, or to define additional material property types.

Format

Prepare the data in the following form:

MID MTYPE E U W M ALPHA FY FBMAJ FBMIN

Description

Variable	Field	Note	Entry
MID	1	(1)	Material identification number.
MTYPE	2	(2)	Material type: = S Steel = C Concrete (frames) = W Concrete (walls) = M Masonry (walls) = O Other
E	3		Modulus of elasticity.
U	4		Poisson's ratio.
W	5	(3)	Weight density (weight/volume).
M	6	(3)	Mass density (mass/volume).
ALPHA	7	(3)	Coefficient of thermal expansion.
FY	8	(5)	Yield stress of structural steel.
FBMAJ	9	(6)	Allowable bending stress for major direction bending.
FBMIN	10		Allowable bending stress for minor direction bending.

Notes

1. The material property sets may be entered in any order; however, the identification numbers must lie between 1 and **NMAT + NRMP**.

   If the identification number is less than or equal to **NMAT**, this property set will replace the corresponding material property set that was previously defined in the ETABS data. If the identification number is greater than **NMAT**, the material property table is expanded, and a new material property set corresponding to this identification number is created.

2. A series of design/ check post processors operating off the ETABS post processing data base are available. The material type designation is basically an indicator for the post processors. The steel checking post processor STEELER will only check those members that have a material type S. Material types C, W, M, and O are ignored by the STEELER program.

3. These values are not used by program STEELER, however, they are input here in the same order as in the ETABS input data for consistency with it.

4. If the yield stress has not been specified, it is assumed to be 36.0 ksi.

5. **FBMAJ** and **FBMIN** are elastic stress check allowables, and are usually not specified. These allowables are usually calculated based upon the structural dimensions and properties, as described in Chapter III, for the various codes. However, if these values are input by the user, they will override all program calculated values.

4. Section Property Redefinition Data

This data section is only needed if the column, beam or brace section property tables that have been previously defined in ETABS are to be modified (or expanded).

Prepare one (or up to three) of the following data sections, 4.i. through 4.iii. below, to redefine the section property tables of the various element types that make up the frames in the structure.

4.i Column Property Redefinition Data

If **NRCP** is 0, skip this data section. Otherwise, provide **NRCP** data sets, to redefine the column property types that were previously defined in the ETABS data, or to define additional column property types.

Format

Each data set consists of a first data line immediately followed by a possible second data line.

a. First Data Line

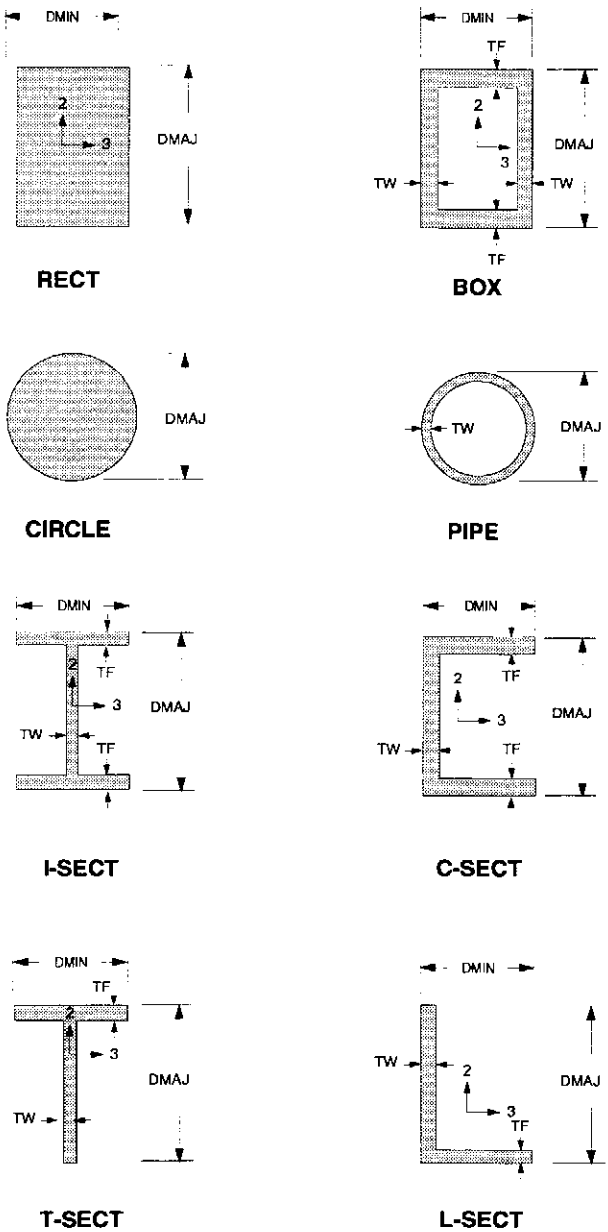
Prepare the first data line in the following form:

ID ITYPE IMAT DMAJ DMIN TF TW

b. Second Data Line

This data line is only needed if the entry for **ITYPE** on the first data line is USER. If the line is needed, it should be prepared in the following form:

A AMAJ AMIN J IMAJ IMIN SMAJ SMIN ZMAJ ZMIN RMAJ RMIN

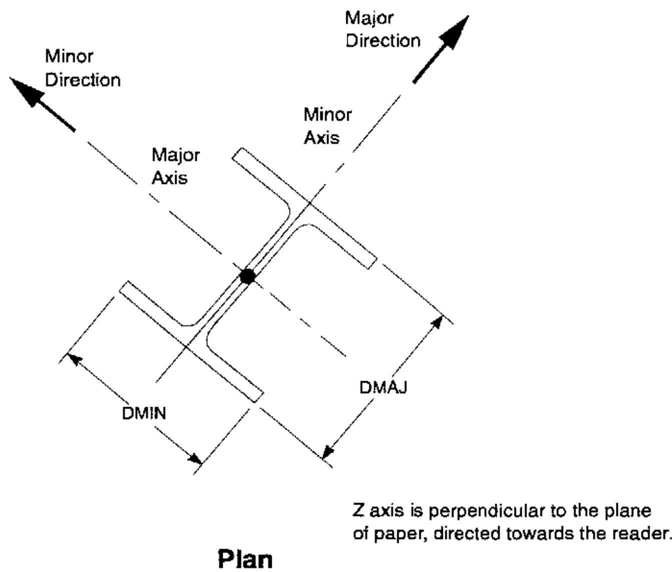


Note: 2 is Major Direction, 3 is Minor Direction

Required Dimensions for Automatic Section Property Calculation
Figure IV-2

Description

Variable	Field	Note	Entry
a. First Data Line			
ID	1	(1)	Identification number of column section property set.
ITYPE	2		Section type:
		(2)	= USER
		(3)	= RECT
			= CIRCLE
			= PIPE
			= BOX
			= I-SECT
			= C-SECT
			= T-SECT
			= L-SECT
		(4)	= W14x230 or other label from database
IMAT	3	(5)	Material identification number for this section property.
DMAJ	4	(3,6)	Section dimension in major direction.
DMIN	5		Section dimension in minor direction.
TF	6	(3)	Flange thickness.
TW	7		Web thickness.
b. Second Data Line For ITYPE = USER			
A	1	(2)	Cross-sectional axial area.
AMAJ	2		Shear area corresponding to major direction shear forces.
AMIN	3		Shear area corresponding to minor direction shear forces.
J	4		Torsional constant.



Column Section Orientation
Figure IV-3

Variable	Field	Note	Entry
IMAJ	5		Moment of inertia, about major axis.
IMIN	6		Moment of inertia, about minor axis.
SMAJ	7		Section modulus, about major axis.
SMIN	8		Section modulus, about minor axis.
ZMAJ	9		Plastic modulus, about major axis.
ZMIN	10		Plastic modulus, about minor axis.
RMAJ	11		Radius of gyration, about major axis.
RMIN	12		Radius of gyration, about minor axis.

Notes

1. The column property sets may be entered in any order; however, the identification numbers must be between 1 and **NCP + NRCP**.

   If the identification number is less than or equal to **NCP**, this property set will replace the corresponding column property set that was previously defined in the ETABS data. If the identification number is greater than **NCP**, the column property table is expanded and a new column property set corresponding to this identification number is created.

2. If **ITYPE** is **USER**, the user is to calculate all the section properties and provide them on the second data line (the second data line must immediately follow the first data line). Only the first six entries of the second data line are read if the material type associated with this section is not steel (S). If the material associated with this section property set is of type S, i.e. steel, all 12 properties must be nonzero.

3. If **ITYPE** is **RECT, CIRCLE, PIPE, BOX, I-SECT, C-SECT, T-SECT** or **L-SECT**, the program recognizes the shapes given in Figure IV-2 and calculates the section properties from the dimensions **DMAJ, DMIN, TF** and **TW**.

4. If **ITYPE** is a label from the database, the complete property set associated with the section (including the dimensions of the section) is recovered by the program from the data base and assigned to the section property identification number.

   The program searches for the label first in the file SECTIONS.PRO and then in any file specified through **PROPFILE** in the Main Control Data, Section 1.c.

5. This entry references the material property types that were previously defined in the ETABS data or subsequently redefined in Section 3 above. This entry must not be less than 1 and must not be greater than the maximum number of material types which exist in the material property table.

6. In addition to being used by the automatic section property calculation options, the column dimensions **DMAJ** and **DMIN** are also used for determining the lengths of the rigid end offsets on the beams that frame into the columns defined with this property set. For column section orientation see Figure IV-3.

4.ii. Beam Property Redefinition Data

If **NRBP** is 0, skip this data section. Otherwise, provide **NRBP** data sets, to redefine the beam property types that were previously defined in the ETABS data, or to define additional beam property types.

Format

Each data set consists of a first data line immediately followed by a possible second data line.

a. First Data Line

Prepare the first data line in the following form:

ID ITYPE IMAT DBMAJ DAMAJ DMIN TF TW

b. Second Data Line

This data line is only needed if the entry for **ITYPE** on the first data line is USER. If this line is needed, it should be prepared in the following form:

A AMAJ AMIN J IMAJ IMIN SMAJ SMIN ZMAJ ZMIN RMAJ RMIN

Description

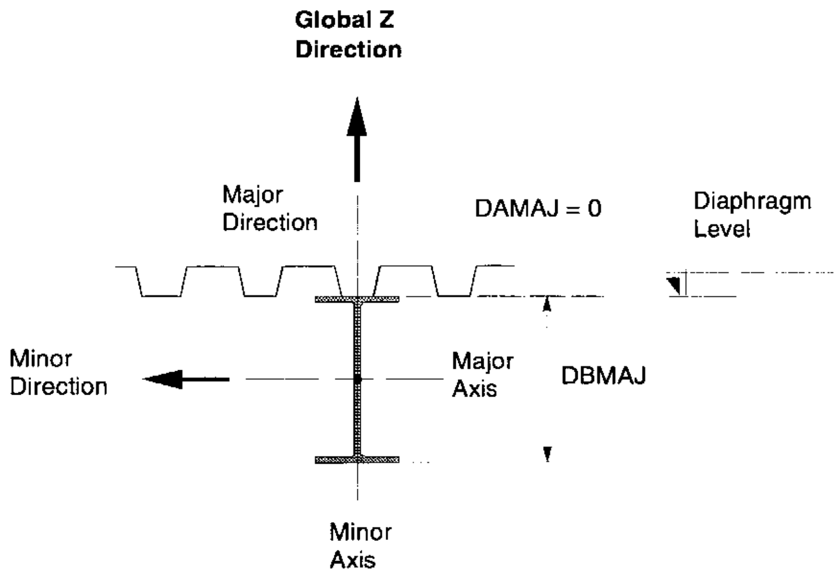
Variable	Field	Note	Entry

a. First Data Line

Variable	Field	Note	Entry
ID	1	(1)	Identification number of beam section property set.
ITYPE	2		Section type:
		(2)	= USER
		(3)	= RECT
			= CIRCLE
			= PIPE
			= BOX
			= I-SECT
			= C-SECT
			= T-SECT
			= L-SECT
		(4)	= W14x230 or other label from database
IMAT	3	(5)	Material identification number for this section property.
DBMAJ	4	(3,6)	Section dimension in major direction, depth below diaphragm.
DAMAJ	5	(3,6)	Section dimension in major direction, depth above diaphragm.
DMIN	6	(3,6)	Section dimension in minor direction.
TF	7	(3)	Flange thickness.
TW	8	(3)	Web thickness.

b. Second Data Line For ITYPE = USER

Variable	Field	Note	Entry
A	1	(2)	Cross-sectional axial area.
AMAJ	2		Shear area corresponding to major direction shear forces.



Beam Section Orientation
Figure IV-4

Variable	Field	Note	Entry
AMIN	3		Shear area corresponding to minor direction shear forces.
J	4		Torsional constant.
IMAJ	5		Moment of inertia, about major axis.
IMIN	6		Moment of inertia, about minor axis.
SMAJ	7		Section modulus, about major axis.
SMIN	8		Section modulus, about minor axis.
ZMAJ	9		Plastic modulus, about major axis.
ZMIN	10		Plastic modulus, about minor axis.
RMAJ	11		Radius of gyration, about major axis.
RMIN	12		Radius of gyration, about minor axis.

Notes

1. The beam property sets may be entered in any order; however, the identification numbers must be between 1 and **NBP + NRBP**.

   If the identification number is less than or equal to **NBP**, this property set will replace the corresponding beam property set that was previously defined in the ETABS data. If the identification number is greater than **NBP**, the beam property table is expanded and a new beam property set corresponding to this identification number is created.

2. If **ITYPE** is **USER**, the user is to calculate all the section properties and provide them on the second data line (the second data line must immediately follow the first data line). Only the first six entries of the second data line are read if the material type associated with this section is not steel (S). If the material associated with this section property set is of type S, i.e. steel, all 12 properties must be nonzero.

3. If **ITYPE** is **RECT, CIRCLE, PIPE, BOX, I-SECT, C-SECT, T-SECT** or **L-SECT**, the program recognizes the shapes given in Figure IV-2 and calculates the section properties from the dimensions **DMAJ, DMIN, TF** and **TW**. In calculating the section properties of shapes in Figure IV-2 for beams, **DMAJ = DBMAJ + DAMAJ**.

4. If **ITYPE** is a label from the database, the complete property set associated with the section (including the dimensions of the section) is recovered by the program from the data base and assigned to the section property identification number.

   The program searches for the label first in the file SECTIONS.PRO and then in any file specified through **PROPFILE** in the Main Control Data, Section 1.c.

5. This entry references the material property types that were previously defined in the ETABS data or subsequently redefined in Section 3 above. This entry must not be less than 1 and must not be greater than the maximum number of material types which exist in the material property table.

6. In addition to being used by the automatic section property calculation options, the beam dimensions, **DBMAJ**, **DAMAJ** and **DMIN** are also used for determining the lengths of the rigid end offsets on the ends of the columns that support the beams defined with this property set. For beam section orientation see Figure IV-4. Beam depths recovered from the AISC data base are assigned to **DBMAJ** (**DAMAJ** is set to 0) unless **DBMAJ** is specified, in which case **DAMAJ = DMAJ - DBMAJ**.

4.iii. Brace Property Redefinition Data

If **NRBRP** is 0, skip this data section. Otherwise, provide **NRBRP** data sets, to redefine the brace property data sets that were previously defined in the ETABS data, or to define any additional brace property types.

Format

Each data set consists of a first data line immediately followed by a possible second data line.

a. First Data Line

Prepare the first data line in the following form:

ID ITYPE IMAT DMAJ DMIN TF TW

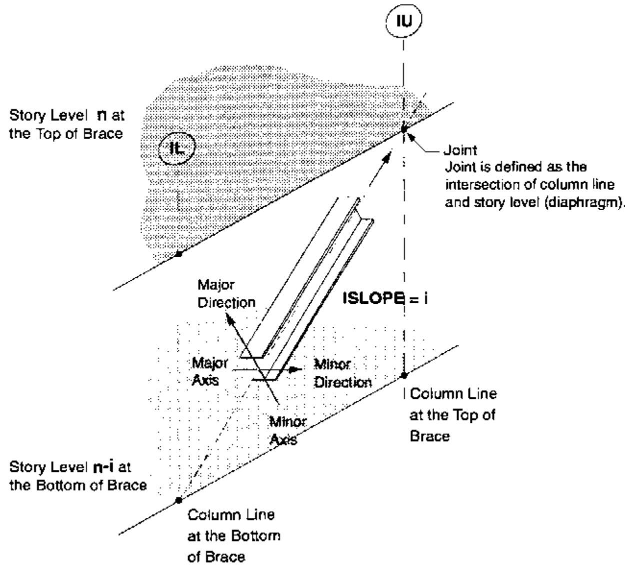
b. Second Data Line

This data line is only needed if the entry for **ITYPE** on the first data line is equal to USER. If this line is needed, it should be prepared in the following form:

A AMAJ AMIN J IMAJ IMIN SMAJ SMIN ZMAJ ZMIN RMAJ RMIN

Description

Variable	Field	Note	Entry
a. First Data Line			
ID	1	(1)	Identification number of brace section property set.
ITYPE	2		Section type:
		(2)	= USER
		(3)	= RECT
			= CIRCLE
			= PIPE
			= BOX
			= I-SECT
			= C-SECT
			= T-SECT
			= L-SECT
		(4)	= W14x230 or other label from database
IMAT	3	(5)	Material identification number for this section property.
DMAJ	4	(3,6)	Section dimension in major direction.
DMIN	5		Section dimension in minor direction.
TF	6	(3)	Flange thickness.
TW	7		Web thickness.
b. Second Data Line For ITYPE = USER			
A	1	(2)	Cross-sectional axial area.
AMAJ	2		Shear area corresponding to major direction shear forces.
AMIN	3		Shear area corresponding to minor direction shear forces.
J	4		Torsional constant.



Brace Section Orientation
Figure IV-5

Variable	Field	Note	Entry
IMAJ	5		Moment of inertia, about major axis.
IMIN	6		Moment of inertia, about minor axis.
SMAJ	7		Section modulus, about major axis.
SMIN	8		Section modulus, about minor axis.
ZMAJ	9		Plastic modulus, about major axis.
ZMIN	10		Plastic modulus, about minor axis.
RMAJ	11		Radius of gyration, about major axis.
RMIN	12		Radius of gyration, about minor axis.

Notes

1. The brace property sets may be entered in any order, however, the identification numbers must be between 1 and **NBRP** + **NRBRP**.

   If the identification number is less than or equal to **NBRP**, this property set will replace the corresponding brace property set that was previously defined in the ETABS data. If the identification number is greater than **NBRP**, the brace property table is expanded and a new brace property set corresponding to this identification number is created.

2. If **ITYPE** is **USER**, the user is to calculate all the section properties and provide them on the second data line (the second data line must immediately follow the first data line). Only the first six entries of the second data line are read if the material type associated with this section is not steel (S). If the material associated with this section property set is of type S, i.e. steel, all 12 properties must be nonzero.

3. If **ITYPE** is **RECT, CIRCLE, PIPE, BOX, I-SECT, C-SECT, T-SECT** or **L-SECT**, the program recognizes the shapes given in Figure IV-2 and calculates the section properties from the dimensions **DMAJ, DMIN, TF** and **TW**.

4. If **ITYPE** is a label from the database, the complete property set associated with the section (including the dimensions of the section) is recovered by the program from the data base and assigned to the section property identification number.

   The program searches for the label first in the file SECTIONS.PRO and then in any file specified through **PROPFILE** in the Main Control Data, Section 1.c.

5. This entry references the material property types that were previously defined in the ETABS data or subsequently redefined in Section 4 above. This entry must not be less than 1 and must not be greater than the maximum number of material types which exist in the material property table.

6. The brace section dimensions **DMAJ** and **DMIN** are not used in the determination of any rigid end offsets. For brace element orientations see Figure IV-5.

5. Frame Stress Check Activation Data Sets

Provide **NFR** data sets, one for each of the ETABS frames that are to be stress checked. Each set consists of one line of Frame Stress Check Control Data and as many lines as needed of Element Reassignment Data as described in the following sections.

5.i. Frame Stress Check Control Data

One line of control data is required.

Format

Prepare the data in the following form:

I ITYP IRCP IRBP IRBRP RW IDBL

Description

Variable	Field	Note	Entry
I	1	(1)	Frame sequence number that uniquely identifies this frame among the **NTF** total frames.
ITYP	2	(2)	Framing type: = 1 Ordinary moment frame = 2 Special moment resisting frame = 3 Braced frame = 4 Eccentrically braced frame = 5 Special concentrically braced frame
IRCP	3	(3)	Column reassignment flag: = 0 No column reassignment provided = 1 Column reassignment provided
IRBP	4		Beam reassignment flag: = 0 No beam reassignment provided = 1 Beam reassignment provided
IRBRP	5		Brace reassignment flag: = 0 No brace reassignment provided = 1 Brace reassignment provided
RW	6	(4)	UBC structural system coefficient (UBC94 code only).
IDBL	7	(5)	Doubler Plate flag (UBC94 code only): = 0 Minimum requirements = 1 Based on $0.8\Sigma M_s$

Notes

1. This is a positive nonzero number, not greater than **NTF**. This sequence number refers to the sequence in which the frames are entered in the ETABS Frame Location data. In this data section the frame that is entered first has a sequence number of 1, and the frame that is entered last has a sequence number of **NTF**.

   The frames may be stress checked in any sequence.

2. The framing type definitions are essentially for seismic design under the UBC94 code. Special stress check considerations associated with different framing types are activated in the stress ratio calculation sequence by this flag. The special considerations associated with each framing type are identified in Chapter III, Section F. The default value of **ITYP** for the UBC94 code is 2 (special moment resisting frame).

   For all other codes (AISC-ASD89, AISC Plastic89, AISC-LRFD93 and the CISC89) **ITYP** of 1 and 3 are the only valid frame types.

3. Irrespective of any material or section property redefinitions in the material or section property data above, it is possible to reassign section properties; live load reduction factors; major or minor direction K-factors; major or minor direction unsupported lengths; bending coefficient, C_b; major or minor direction moment coefficients, C_m; and moment magnification factors on an element-by-element basis.

   If any of these parameters are to be reassigned for any of the column elements in this frame, **IRCP** must be set to 1. The program will then expect column element reassignment data in Section 5.ii.a. below.

   Similarly, **IRBP** and **IRBRP** apply to beam and brace reassignments, respectively.

4. This entry overwrites the built-in value of R_w for the particular framing type. This value is used to calculate the built-in special loading combinations for the design of seismic frames under the UBC94 code.

5. This entry selects the criterion used in computing doubler plates under the UBC94 code. A value of 0 forces the program to select doubler plates based upon the panel zone shear force calculated from the gravity loads plus 1.85 times the prescribed seismic forces or calculated from $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. (UBC 2211.7.2.1), whichever is lower. A value of 1 only uses $0.8\Sigma M_s$ of the girders to compute doubler plates. The default value is 0.

5.ii. Element Reassignment Data

This data section is only needed if the column, beam or brace element section property identifications; live load reduction factors; effective length (K-) factors; bending coefficient, C_b; moment coefficients, C_m; moment magnification factors; and/or eccentric brace frame link beam clear lengths and active/inactive status are to be modified or overridden.

Prepare one (or up to three) of the following data Sections a, b and c below, as required.

5.ii.a. Column Reassignment Data

If **IRCP** is 0, none of the column element parameters are to be reassigned, therefore, skip this data section (including the blank termination line defined below).

Otherwise, provide as many data lines as needed to define the required parameters. The order of input is immaterial, and parameter assignments for any column element at any level may be repeated. The last values read (or generated) will be used. **End this data section with a blank line.**

Format

Prepare the data in the following form:

NT NC1 NC2 NSAME SD1 SD2 P1 P2 P3 P4

Description

Variable	Field	Note	Entry
NT	1	(1)	Data line type: = I Property type = R Live load factor = K K-factors = L Unsupported lengths = C C_b and C_m factors = M Moment Magnification factors
NC1	2	(2,5)	Column line number of first column line being reassigned.
NC2	3		Column line number of last column line being reassigned.
NSAME	4	(3)	Column line number, the properties of which are to be repeated at column lines **NC1** through **NC2**.
SD1	5	(4,5)	Identification of the first story level associated with the column being reassigned.
SD2	6		Identification of the last story level associated with the column being reassigned.
P1	7	(6)	Parameter 1: = Column property ID (**NT** = I) = Live load factor (**NT** = R) = Major K-factor (**NT** = K) = Major length (**NT** = L) = C_{mx} (**NT** = C) = Major moment magnification factor (sidesway moments) (**NT**= M)

Variable	Field	Note	Entry
P2	8		Parameter 2: = Minor K-factor (**NT** = K) = Minor length (**NT** = L) = C_{my} (**NT** = C) = Minor moment magnification factor (sidesway moments) (**NT** = M)
P3	9		Parameter 3: = C_b (**NT** = C) = Major moment magnification factor (non-sidesway moments) (**NT** = M)
P4	10		Parameter 4: = Minor moment magnification factor (non-sidesway moments) (**NT** = M)

Notes

1. This entry identifies the type of data that is being defined by this data line. For example, if **NT** = L, the parameters **P1** and **P2** on this data line are major and minor direction unsupported lengths, respectively.

2. This entry is a column line number. The number must be positive and not greater than **NC**.

3. If **NSAME** is nonzero, it is a column line number, the properties of which are already defined by default or by user specifications in preceding data lines of this data section. The nonzero entry for **NSAME** puts the program into a duplication mode. In this mode, the member properties (as identified by the **NT** entry) for the column elements at all levels on column lines **NC1** through **NC2** are set identical to that of column line **NSAME** as it stands defined at the time of this entry.

   Subsequent reassignment of properties to column line **NSAME** in later data lines will not result in automatic corresponding reassignment of the properties on column lines **NC1** through **NC2**, or vice versa.

   In this duplication mode (i.e. when **NSAME** is non zero), the entries for **SD1, SD2, P1, P2, P3** and **P4** are meaningless and must not be entered. These entries are only needed if **NSAME** is 0.

4. This entry is an alphanumeric story identifier that must correspond to one of the story level identifiers previously defined in the ETABS data. The story level associated with a column element is the story level at the top of the column.

5. All column elements existing on column lines **NC1** through **NC2** associated with levels **SD1** through **SD2** will be assigned the properties identified by the entries **NT**, **P1**, **P2**, **P3** and **P4** on this data line.

6. If **NT** = I, the data line is for reassigning member property identifications and the parameter **P1** is an integer entry referring to the section property tables originally defined in the ETABS data or redefined in Section 4 above. If **NT** = I, **P2, P3** and **P4** are not used. The default properties are as originally defined in the ETABS data.

   If **NT** = R, the data line is for reassigning live load reduction factors. The entry **P1** is the live load reduction factor for the element.

   Thus, for instance, if the axial force in an element at a particular level for load condition **LLC** is 50k, and the entry for **P1** is 0.7, then the axial force in load condition **LLC** (that will further be scaled by the design load combinations) will be taken as

$$0.7 \times 50^k = 35^k$$

The program does not have any algorithm based upon tributary area to calculate the live load reduction factor automatically.

If **NT** = R, **P2, P3** and **P4** are not used. The default value for the element live load reduction factor is 1.0.

If **NT** = K, the data line is for reassigning element K-factors. The entries **P1** and **P2** are the K-factors of the element in the major and minor directions, respectively. If **NT** = K, **P3** and **P4** are not used. The default values are as calculated in the algorithm defined in Chapter III.

If **NT** = L, the data line is for reassigning element unsupported lengths. The entries **P1** and **P2** are the unsupported lengths of the element in the major and minor directions of the element, respectively. If **NT** = L, **P3** and **P4** are not used. The default values are as obtained by the algorithm defined in Chapter III.

If **NT** = C, the data line is for defining the bending coefficient, C_b and for defining the moment coefficients, C_m. The entries **P1** and **P2** are taken as the values for C_{mx} and C_{my} for the major and minor direction moment coefficients, respectively. The entry **P3** is taken as the value of C_b for all loading combinations. If **NT** = C, **P4** is not used. The default values are as calculated in the algorithms defined in Chapter III for the various design codes.

If **NT** = M and the AISC-LRFD93 or the CISC89 codes are being used, the data line is for defining moment magnification factors. The entries **P1** and **P2** are the moment magnification factors for sidesway moments (B_2 for AISC-LRFD93 and U_2 for CISC89) (from Load Conditions A, B, C, D1, and D2) in the major and minor directions, respectively. The default value for these is 1.0 as the program assumes a P-Delta analysis has been made. For the AISC-LRFD93 code the entries **P3** and **P4** are moment magnification factors for non-sidesway moments (B_1) (from Load conditions I, II and III) in the major and minor directions, respectively. The default values for these are as calculated in the algorithms defined in Chapter III, Section H. For the CISC89 code the entries **P3** and **P4** are moment magnification factors for total moments to account for deformation of member between its ends (U_1) in the major and minor directions, respectively. The default values for these are as calculated in the algorithms defined in Chapter III, Section I.

5.ii.b. Beam Reassignment Data

IIf **IRBP** is 0, none of the beam element parameters are to be reassigned, therefore, skip this data section (including the blank termination line defined below). Also, if **NB** is 0 there are no bays defined in this frame. Therefore, skip this data section completely (including the blank termination line defined below).

Otherwise, provide as many data lines as needed to define the required parameters. The order of input is immaterial, and parameter assignments for any beam element at any level may be repeated. The last values read (or generated) will be used. **End this data section with a blank line.**

Format

Prepare the data in the following form:

NT NB1 NB2 NSAME SD1 SD2 P1 P2 P3 P4

Description

Variable	Field	Note	Entry
NT	1	(1)	Data line type: = I Property type = R Live load factor = K K-factors = L Unsupported lengths = C C_b and C_m factors = M Moment Magnification factors = E Link Beam length and status
NB1	2	(2,5)	Bay number of first bay being reassigned.
NB2	3		Bay number of last bay being reassigned.
NSAME	4	(3)	Bay number, the properties of which are to be repeated at bays **NB1** through **NB2**.
SD1	5	(4,5)	Identification of the first story level associated with the beam being reassigned.
SD2	6		Identification of the last story level associated with the beam being reassigned.
P1	7	(6)	Parameter 1: = Beam property ID (**NT** = I) = Live load factor (**NT** = R) = Major K-factor (**NT** = K) = Major length (**NT** = L) = C_{mx} (**NT** = C) = Major moment magnification factor (sidesway moments) (**NT**= M) = Link beam status 0 Active 1 Inactive (**NT** = E)

Variable	Field	Note	Entry
P2	8		Parameter 2: = Minor K-factor (**NT** = K) = Minor length (**NT** = L) = C_{my} (**NT** = C) = Minor moment magnification factor (sidesway moments) (**NT** = M) = Link beam clear length (**NT** = E)
P3	9		Parameter 3: = C_b (**NT** = C) = Major moment magnification factor (non-sidesway moments) (**NT** = M)
P4	10		Parameter 4: = Minor moment magnification factor (non-sidesway moments) (**NT** = M)

Notes

1. This entry identifies the type of data that is being defined by this data line. For example, if **NT** = L, the parameters **P1** and **P2** on this data line are major and minor direction unsupported lengths, respectively.

2. This entry is a bay number. The number must be positive and not greater than **NB**.

3. If **NSAME** is nonzero, it is a bay number, the properties of which are already defined by default or by user specifications in preceding data lines of this data section. The nonzero entry for **NSAME** puts the program into a duplication mode. In this mode, the member properties (as identified by the **NT** entry) for the beam elements at all levels on bays **NB1** through **NB2** are set identical to that of bay **NSAME** as it stands defined at the time of this entry.

   Subsequent reassignment of properties to bay **NSAME** in later data lines will not result in automatic corresponding reassignment of the properties on bays **NB1** through **NB2**, or vice versa.

   In this duplication mode (i.e. when **NSAME** is non zero), the entries for **SD1, SD2, P1, P2, P3** and **P4** are meaningless and must not be entered. These entries are only needed if **NSAME** is 0.

4. This entry is an alphanumeric story identifier that must correspond to one of the story level identifiers previously defined in the ETABS data.

5. All beam elements existing on bays **NB1** through **NB2** associated with levels **SD1** through **SD2** will be assigned the properties identified by the entries **NT**, **P1**, **P2**, **P3** and **P4** on this data line.

6. For **NT** = I, R, K, L, C, and M see Note 6 under the column reassignment data.

   If **NT** = E, the data line is for defining the clear length and status of link beams in eccentrically braced frames. This allows users to overwrite clear lengths of link beams calculated by program. It also allows the user to inactivate link beams detected by program or declare (activate) beams to be link beams.

5.ii.c. Brace Reassignment Data

If **IRBRP** is 0, none of the brace element parameters are to be reassigned, therefore, skip this data section (including the blank termination line defined below).

Also, if **MBR** for this frame is 0 in the ETABS data then there are no braces in this frame. Therefore, skip this data section completely (including the blank termination line defined below).

Otherwise, provide as many data lines as needed to define the required parameters. The order of input is immaterial, and parameter assignments for any brace element at any level may be repeated. The last values read (or generated) will be used. **End this data section with a blank line.**

Format

Prepare the data in the following form:

NT NBR1 NBR2 NBRINC P1 P2 P3 P4

Description

Variable	Field	Note	Entry
NT	1	(1)	Data line type: = I Property type = R Live load factor = K K-factors = L Unsupported lengths = C C_b and C_m factors = M Moment Magnification factors
NBR1	2	(2)	Brace ID of first brace being reassigned.
NBR2	3		Brace ID of last brace being reassigned.
NBRINC	4		Brace ID increment.
P1	5	(3)	Parameter 1: = Brace property ID (**NT** = I) = Live load factor (**NT** = R) = Major K-factor (**NT** = K) = Major length (**NT** = L) = C_{mx} (**NT** = C) = Major moment magnification factor (sidesway moments) (**NT**= M)
P2	6		Parameter 2: = Minor K-factor (**NT** = K) = Minor length (**NT** = L) = C_{my} (**NT** = C) = Minor moment magnification factor (sidesway moments) (**NT** = M)
P3	7		Parameter 3: = C_b (**NT** = C) = Major moment magnification factor (non-sidesway moments) (**NT** = M)
P4	8		Parameter 4: = Minor moment magnification factor (non-sidesway moments) (**NT** = M)

Notes

1. This entry identifies the type of data that is being defined by this data line. For example, if **NT** = L, the parameters **P1** and **P2** on this data line are major and minor direction unsupported lengths, respectively.

2. The parameters **NBR1, NBR2** and **NBRINC** define the following series of brace element identifications:

   NBR1, NBR1 + NBRINC, NBR1 + 2 NBRINC . . .

   which continues until **NBR2** is reached.

   All generated identifications that do not correspond to actual brace element identifications are ignored by the program. All brace elements that exist and are defined by the above series will be assigned the properties identified by the entries **NT, P1, P2, P3** and **P4** on this data line.

3. For **NT** = I, R, K, L, C, and M see Note 6 under the column reassignment data.

Chapter V

Program Output

Depending upon the options activated by the STEELER input data, the program will produce up to five output files. The names of these output files are built from the input filename with unique filename extensions. Therefore, if the name of the STEELER input data file is CHKSTL, the five output files produced by STEELER will be CHKSTL.STL, CHKSTL.DTL, CHKSTL.SHR, CHKSTL.MAP and CHKSTL.PLO. However, if the program defaults are acceptable and the STEELER input data file is not provided, the five output files produced by STEELER will have the same extensions as above but will be derived from the ETABS input file name.

A. The *filename*.STL File

The output file *filename*.STL contains a tabulated echo of all the STEELER input data and other pertinent information recovered from the ETABS post processing file. Link beam information along with actual and allowable link beam rotations are also printed for eccentric braced frames.

The file contains the controlling interaction stress ratio information and the associated stress locations, combination numbers and code equation identifiers for all members. Load combinations which cause axial compression in the members are treated separately from load combinations which cause axial tension (or no axial force) in the members.

Sections are classified as SEISMIC, COMPACT, NON-COMPACT, CLASS 1, CLASS 2, CLASS 3 or CLASS 4 depending upon the code and type of frame specified for stress check.

Doubler and continuity plate requirements for the columns, controlling beam joint shears for beam connection design and maximum compression and tension axial force values in all members are included in this file. See Figures V-1, V-2 and V-3.

The notes in this section correspond to the numbers shown in these figures.

1. This is the column line ID number of the column, or the bay ID number of the beam, or the brace ID number of the brace.

2. This is the element section type being used. Sections specified off the data base, will display the section label, e.g., W14X68; other section types are identified as USER, I-SECT etc.

3. The program calculates and displays controlling interaction stress ratios separately for load combinations which produce axial compression (C) and load combinations which produce axial tension (T), or no axial force, in the member.

4. This is the value of the controlling interaction stress ratio.

5. This is the location where the controlling interaction stress ratio exists.

6. This number identifies the design load combination associated with the controlling interaction stress ratio. It is reported as zero if the special built-in loading combinations govern.

7. This identifies the code equation or subsection number that produced the controlling interaction stress ratio. Non-code identifications (BENDING) and (TENSION) correspond to special equations defined in Chapter III, Section E, G and I.

8. This is the maximum axial force (in kips {K}for English units, tons{T} for MKS units and kiloNewtons {kN} for SI units) generated in the member from any of the specified design load combinations including special built-in loading combinations for some codes. This value is not necessarily from the load combination identified in Note 6 above.

9. This is the area of each of the continuity plates (in square inches {sqin}for English units and square millimeters {sqmm} for MKS and SI units) required at the top of the column at this level. The continuity plate material is assumed to be the same as the column material. This is only calculated for I shapes (including W, M, S or HP sections from the data base).

10. This is the thickness of the doubler plate (in inches {in}for English units and millimeters {mm} for MKS and SI units) required at the top of the column at this level. The doubler plate material is assumed to be the same as the column material. This is only calculated for I shapes (including W, M, S or HP sections from the data base).

11. This is the section classification based on the design code and framing type. e.g., for special moment resisting frame and eccentrically braced frame in the UBC94 code the section may be classified as SEISMIC, COMPACT or NON-COMPACT.

12. These are the maximum beam connection shears (in kips{K}for English units, tons {T} for MKS units and kiloNewtons {kN}for SI units) for the design of the beam connections.

13. This is the brace connection design force. This force is not necessarily equal to the maximum axial force reported adjacent to it.

See Chapter III for code references.

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    8
                                        ETABS_FILE:exstl.PST/STEELER_FILE:chkstl.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

  COL SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CONT-PL DBLR-PL SECTION
   ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION   AXIAL    AREA  THICK    TYPE
                                                   (K)   (sqin)   (in)

   1  W14X120                                               1.3    0.00 SEISMIC
               (C)  0.232 BOTTOM < 1> H1-3        22.9
   |   |        |     |     |      |    |           |         |      |       |
  (1) (2)      (3)   (4)   (5)    (6)  (7)         (8)       (9)   (10)    (11)
   |   |        |     |     |      |    |           |         |      |       |
   --------------------Note Reference Number------------------------------
```

Column Interaction Stress Check
File: filename.STL
Figure V-1

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   10
                                        ETABS_FILE:exstl.PST/STEELER_FILE:chkstl.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

 BEAM SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CON-SHR CON-SHR SECTION
   ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION   AXIAL   END-I   END-J    TYPE
                                                   (K)     (K)     (K)

   1  W18X50                                              42.5    44.3 SEISMIC
               (T)  0.690 END-J  < 1> (BENDING)    0.0
   |   |        |     |     |      |      |         |       |       |       |
  (1) (2)      (3)   (4)   (5)    (6)    (7)       (8)    (12)    (12)    (11)
   |   |        |     |     |      |      |         |       |       |       |
   ------------------------Note Reference Number-------------------------
```

Beam Interaction Stress Check
File: filename.STL
Figure V-2

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS   PAGE   12
                                     ETABS_FILE:sample.PST/STEELER_FILE:samstl.STL
 FILE : SAMSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... / 3D FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BRACE  AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

  BRC SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CONNECT        SECTION
   ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION   AXIAL   FORCE           TYPE
                                                   {K}     {K}

    1  W14X43
    |     |        (C)  0.115 BOTTOM < 7> (H1-3)     12.1      |        SEISMIC
    |     |        (T)  0.004 BOTTOM < 6> (H2-1)      1.0      |              |
    |     |         |     |     |     |     |          |       |              |
   (1)   (2)       (3)   (4)   (5)   (6)   (7)        (8)    (13)           (11)
    |     |         |     |     |     |     |          |       |              |
    -------------------------Note Reference Number---------------------------
```

Brace Interaction Stress Check
File: filename.STL
Figure V-3

B. The *filename*.DTL File

The output file *filename*.DTL contains backup information associated with the calculation of the interaction stress ratios tabulated in the *filename*.STL file. The file contains a breakdown of the controlling stress ratios in terms of the contributions from major bending, minor bending and axial effects. The breakdown of the stress ratios into their various components assists the user in taking effective corrective measures, in the event of indicated overstress.

The file also contains the effective length (K-) factors and the unsupported lengths in the major and minor directions of the members.

For special moment resisting frames in the UBC94 code the ratios of the beam plastic capacities, with respect to the column plastic capacities at the top joint of any column for the column major and minor directions are also tabulated for each column element.

For the AISC-LRFD93 and the CISC89 code major and minor moment magnification factors are reported.

For link beams in eccentric braced frames rotation ratios (actual rotation divided by allowable rotation) and link beam failure mode are also reported.

The notes in this section correspond to the numbers shown in Figure V-4.

1. This is the column line ID number of the column, or the bay ID number of the beam, or the brace ID number of the brace.

2. The program calculates and displays controlling interaction stress ratios separately for load combinations which produce axial compression (C) and load combinations which produce axial tension (T), or no axial force, in the member.

3. This is the breakdown of the stress ratio according to contributions from the axial, major bending and minor bending of the controlling design combination. The sum of these three values will equal the stress ratio, except for circular sections the axial component is added to the SRSS of the two bending components. Beams connected to the diaphragm at both ends will only get major bending components.

4. These are the effective length (K-) factors of the member in the major and minor directions, respectively.

5. These are the member unsupported lengths in the major and minor directions, respectively.

6. These are the ratios of the beam to column plastic moment capacity summations in the column major and minor directions, respectively, at the top of the column at this level for special moment resisting frames in the UBC94 code only. For link beams in eccentrically braced frames this same location is used to report rotation ratios and beam failure modes. For the AISC-LRFD93 and the CISC89 codes this same location is used to report moment magnification factors.

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                         ETABS_FILE:exstl.PST/STEELER FILE:chkstl.DTL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN INTERACTION DETAILS

  COL CHECK  <---BREAK DOWN---><-K-FACTORS-><-UNSUP LENGTHS-><--B/C  RATIOS-->
   ID TYPE   AXIAL MAJOR MINOR  MAJOR MINOR    MAJOR   MINOR   MAJOR   MINOR
                                               {in}    {in}

    1                           2.55  1.06   102.01   87.14    0.49    4.15
     (C)     0.034 0.191 0.007

   (1)(2)           (3)              (4)            (5)              (6)
    |  |             |                |              |                |
    -----------------------Note Reference Number-----------------------
```

Column Interaction Output Details
File: filename.DTL
Figure V-4

C. The *filename*.SHR File

The output file *filename*.SHR contains controlling shear stress ratios and the associated stress locations and combination numbers for the major and minor directions of all members. See Figure V-5.

The notes in this section correspond to the numbers shown in this figure.

1. This is the column line ID number of the column, or the bay ID number of the beam, or the brace ID number of the brace.

2. This is the value of the controlling shear stress ratio for the shear force in the specified direction.

3. This is the location where the controlling stress ratio exists.

4. This number identifies the design load combination associated with the controlling shear stress ratio.

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS   PAGE    1
                                      ETABS_FILE:exstl.PST/STEELER FILE:chkstl.SHR
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN SHEAR STRESS CHECK

        <---MAJOR  SHEAR---> <---MINOR  SHEAR--->
  COL     STRESS STRESS         STRESS STRESS
   ID      RATIO POINT  <LC>     RATIO POINT  <LC>

    1      0.138 BOTTOM < 1>     0.006 BOTTOM < 5>
    |        |      |     |        |      |      |
   (1)      (2)    (3)   (4)      (2)    (3)    (4)
    |        |      |     |        |      |      |
    ----------Note Reference Number--------------
```

Column Shear Stress Check
File: filename.SHR
Figure V-5

D. The *filename*.MAP File

The output file *filename*.MAP contains numerical maps of design parameters, such as controlling interaction stress ratios, doubler plates, continuity plates, link beam shear stress ratios, etc. These maps display the parameters in a spatial manner that corresponds to an elevation view of the frame being stress checked.

This form of output is concise and gives the engineer a good overall view of the stress and design state of the frame.

The map output is only available for selected parameters of column and beam elements. See Figure V-6.

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                      ETABS FILE:exstl.PST/STEELER_FILE:chkstl.MAP
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME

 UNIFORM BUILDING CODE 1994
 MAP OF CONTROLLING COLUMN INTERACTION STRESS RATIOS

          COLUMN        1       2       3       4       5       6       7     8
 LEVEL

 ROOF              0.232  0.096  0.232  0.053  0.053  0.130  0.147 0.130

 FLOOR             0.188  0.204  0.188  0.103  0.103  0.159  0.176 0.159

 LEVEL
          COLUMN        1       2       3       4       5       6       7     8
```

Map Of Column Interaction Stress Ratios
File: filename.MAP
Figure V-6

E. The *filename*.PLO File

This is a binary file containing data used by the display post processor program ETABSOUT.

References

1. American Institute of Steel Construction

Manual of Steel Construction, Allowable Stress Design, 9th Edition, Chicago, Illinois, 1989.

2. International Conference of Building Officials

Uniform Building Code, Whittier, California, 1994.

3. American Institute of Steel Construction

Manual of Steel Construction, Load & Resistance Factor Design, Second Edition, Chicago, Illinois, 1994.

4. Canadian Standards Association

Limit States Design of Steel Structures - CAN/CSA-S16.1-M89, Rexdale, Ontario, 1989.

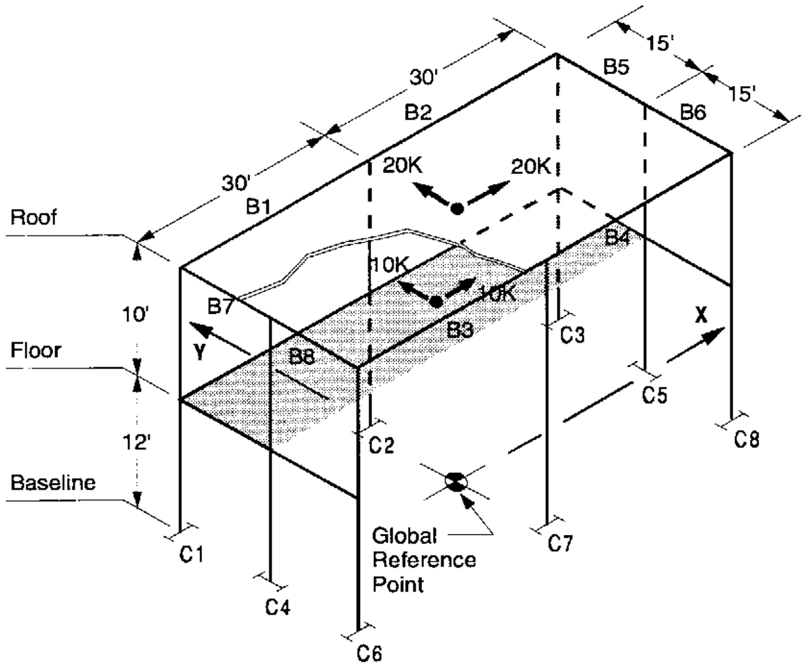
5. White, D. W. and Hajjar, J. F.

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

Sample Example

The following is an example to illustrate the typical input and output associated with a STEELER run.

The STEELER input data file, CHKSTL, and the corresponding ETABS input data filc EXSTL, along with the ETABS post processing file EXSTL.PST, all exist on the STEELER diskette, which comes with the complete STEELER package.



Typical Column W14x120

Typical Beam W33x118
(except 1 and 2 W18x50)

Loading on Beams 1 Through 4:
1.0 k/ft DL
.5 k/ft LL

Special Moment Resisting Frame

(UBC 1994)

Sample Example

```
$ -------------------------------HEADING
ETABS 6.1
EXAMPLE EXSTL :  SAMPLE EXAMPLE FOR STEELER MANUAL UNITS: KIP-INCH-SEC
SPECIAL MOMENT RESISTING STEEL FRAME: USER DEFINED STATIC LATERAL LOAD
$ -------------------------------CONTROL DATA
 2 1 1 1 0 4 0 1 1 2 0 0 0 0 1 0 0 0 4 1 1
 386.4
$ -------------------------------STORY DATA
 ROOF  120
 FLOOR 144
$ -------------------------------MATERIAL PROPERTY DATA
 1  S  29500 .25 0.49/1728 0 0 36
$ -------------------------------COLUMN SECTION PROPERTIES
 1      W14X120 1
$ -------------------------------BEAM SECTION PROPERTIES
 1      W18X50  1
 2      W33X118 1
$ -------------------------------FRAME DATA
 MAIN FRAME
 1 8 8 0 0 2 0 1
$ -------------------------------COLUMN LINE LOCATIONS
 1  -360  180
 2     0  180
 3   360  180
 4  -360    0 90
 5   360    0 90
 6  -360 -180
 7     0 -180
 8   360 -180
$ -------------------------------BAY CONNECTIVITY
 1  1  2
 2  2  3
 3  6  7
 4  7  8
 5  3  5
 6  5  8
 7  1  4
 8  4  6
$ -------------------------------BEAM SPAN VERTICAL LOADING PATTERNS
 1 0 1/12
 2 0 .5/12
$ -------------------------------JOINT ASSIGNMENTS
 6 8 0 FLOOR FLOOR 0

$ -------------------------------COLUMN ASSIGNMENTS
 1 8 0 ROOF FLOOR 1

$ -------------------------------BEAM ASSIGNMENTS
 1 2 0 ROOF FLOOR 1
 3 4 0 ROOF ROOF  2
 5 8 0 ROOF FLOOR 2

$ -------------------------------ASSIGNMENT OF BEAM SPAN LOADINGS
 1 4 0 ROOF FLOOR 1 2
```

ETABS Input Data File : ***EXSTL***

```
$ ------------------------------FRAME LOCATION DATA
 1 0 0 0 /MAIN FRAME
$ ------------------------------USER-DEFINED LATERAL STATIC LOAD
 ROOF  1 A 20
 ROOF  1 B  0 20
 FLOOR 1 A 10 0  0 7.5*12
 FLOOR 1 B  0 10

$ ------------------------------LOAD CASE DEFINITION DATA
 1 0 1
 2 0 0 1
 3 0 0 0 0 1
 4 0 0 0 0 0 1
$ ------------------------------END OF INPUT DATA
```

*ETABS Input Data File : **EXSTL** (continued)*

```
$ --------------------------- HEADING DATA
  STEELER 6.1
  FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
  SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND
$
$ --------------------------- CONTROL DATA
$ ICODE NFR NLC LLC NRMP NRCP NRBP NRBRP
     1   1   9   2    0    0    0     0
$ --------------------------- OUTPUT CONTROL INFORMATION
$   RI    RS  IDC  IDB  IDBR  ISC  ISB  ISBR  MCI  MBI  MCC  MCD
    0     0    1    1     1    1    1     1    1    1    1    1
$
$ --------------------------- LOAD COMBINATION DEFINITION DATA
$ L  LTYP   XI     XII   XIII     XA     XB    XC   XD2    XD3
  1   0   1.0     1.0    0.0
  2   0   0.75    0.75   0.0    0.75
  3   0   0.75    0.75   0.0   -0.75
  4   0   0.75    0.75   0.0    0.0    0.75
  5   0   0.75    0.75   0.0    0.0   -0.75
  6   0   0.75    0.0    0.0    0.75
  7   0   0.75    0.0    0.0   -0.75
  8   0   0.75    0.0    0.0    0.0    0.75
  9   0   0.75    0.0    0.0    0.0   -0.75
$
$ --------------------------- MATERIAL PROPERTY REPLACEMENT DATA
$ MID MTYPE   E  U   W   M  ALPHA  FY  FBMAJ   FBMIN
$
$ --------------------------- COLUMN SECTION PROPERTY REPLACEMENT DATA
$ ID ITYPE  IMAT  DMAJ  DMIN  TF  TW
$
$ --------------------------- BEAM SECTION PROPERTY REPLACEMENT DATA
$ ID ITYPE  IMAT  DBMAJ  DAMAJ  DMIN  TF  TW
$
$ --------------------------- BRACE SECTION PROPERTY REPLACEMENT DATA
$ ID ITYPE  IMAT  DMAJ  DMIN  TF  TW
$
$ --------------------------- FRAME STRESS CHECK ACTIVATION DATA
$ I   ITYP  IRCP  IRBP  IRBRP  RW  IDBL
  1     2     0     1      0   0     0
$
$ --------------------------- COLUMN ELEMENT REDEFINITION DATA
$ NT NC1  NC2  NSAME  SD1  SD2  P1  P2  P3  P4
$
$ --------------------------- BEAM ELEMENT REDEFINITION DATA
$ NT NB1 NB2 NSAME    SD1  SD2    P1       P2     P3   P4
  L    1   2   0      ROOF FLOOR  0  30*12/3
  L    3   4   0      ROOF ROOF   0  30*12/3

$ --------------------------- BRACE ELEMENT REDEFINITION DATA
$ NT  NBR1  NBR2  NBRINC  P1  P2  P3  P4
$
$ END OF INPUT DATA
```

STEELER Input Data File : ***CHKSTL***

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                    ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 CODE CHECK IDENTIFIER---------------------------      1 (UBC 1994 STEEL)

 NUMBER OF FRAMES TO BE CHECKED------------------      1
 NUMBER OF STRESS CHECK LOAD COMBINATIONS--------      9
 ETABS LIVE LOAD CONDITION NUMBER----------------      2

 NUMBER OF REPLACED MATERIAL PROPERTIES----------      0
 NUMBER OF REPLACED COLUMN PROPERTIES------------      0
 NUMBER OF REPLACED BEAM PROPERTIES--------------      0
 NUMBER OF REPLACED BRACE PROPERTIES-------------      0

 TYPE OF UNITS (ENGLISH, MKS OR SI)--------------      E

 EXECUTION MODE----------------------------------      0

 INTERACTION STRESS RATIO CUTOFF----------------- 0.0000
 SHEAR STRESS RATIO CUTOFF----------------------- 0.0000

 COLUMN INTERACTION DETAIL FLAG------------------      1
 BEAM INTERACTION DETAIL FLAG--------------------      1
 BRACE INTERACTION DETAIL FLAG-------------------      1
 COLUMN SHEAR STRESS CHECK FLAG------------------      1
 BEAM SHEAR STRESS CHECK FLAG--------------------      1
 BRACE SHEAR STRESS CHECK FLAG-------------------      1

 MAP OF COLUMN INTERACTION STRESS RATIOS FLAG----      1
 MAP OF BEAM INTERACTION STRESS RATIOS FLAG------      1
 MAP OF COLUMN CONTINUITY PLATES FLAG------------      1
 MAP OF COLUMN DOUBLER PLATES FLAG---------------      1
```

*Sample Output from **STEELER***

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    2
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 DESIGN LOADING COMBINATION DATA

 LOAD TYPE       I       II      III       A       B       C   DYN-1  DYN-2
    1    0   1.000   1.000   0.000   0.000   0.000   0.000   0.000  0.000
    2    0   0.750   0.750   0.000   0.750   0.000   0.000   0.000  0.000
    3    0   0.750   0.750   0.000  -0.750   0.000   0.000   0.000  0.000
    4    0   0.750   0.750   0.000   0.000   0.750   0.000   0.000  0.000
    5    0   0.750   0.750   0.000   0.000  -0.750   0.000   0.000  0.000
    6    0   0.750   0.000   0.000   0.750   0.000   0.000   0.000  0.000
    7    0   0.750   0.000   0.000  -0.750   0.000   0.000   0.000  0.000
    8    0   0.750   0.000   0.000   0.000   0.750   0.000   0.000  0.000
    9    0   0.750   0.000   0.000   0.000  -0.750   0.000   0.000  0.000
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 MATERIAL PROPERTIES

  ID TYPE      ELASTIC    POISSONS        UNIT        UNIT    COEFF OF
               MODULUS       RATIO      WEIGHT        MASS   EXPANSION
                 (Ksi)                 (K/cuin)

   1    S  0.2950E+05      0.2500  0.2836E-03  0.0000E+00 0.0000E+00


 MATERIAL PROPERTIES FOR DESIGN

                 YIELD   STRENGTH       YIELD   STRENGTH        ALLOWABLES
  ID TYPE           FY      FC(FM)        FYS   FCS(FMS)       FBMAJ      FBMIN
                 (Ksi)       (Ksi)      (Ksi)      (Ksi)       (Ksi)      (Ksi)

   1    S  0.360E+02                                      0.000E+00 0.000E+00
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                                    ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND
```

SECTION PROPERTIES FOR COLUMNS

ID	SECTION TYPE	MAT ID	MAJOR DIM (in)	MINOR DIM (in)	FLANGE THICK (in)	WEB THICK (in)
1	W14X120	1	14.480	14.670	0.940	0.590

ANALYSIS SECTION PROPERTIES FOR COLUMNS

ID	AXIAL A (in_2)	MAJOR AV (in_2)	MINOR AV (in_2)	TORSION J (in_4)	MAJOR I (in_4)	MINOR I (in_4)
1	35.300	8.543	22.983	0.9370E+01	0.1380E+04	0.4950E+03

STRESS CHECK SECTION PROPERTIES FOR COLUMNS

ID	MAJOR S (in_3)	MINOR S (in_3)	MAJOR Z (in_3)	MINOR Z (in_3)	MAJOR R (in)	MINOR R (in)
1	190.608	67.485	212.000	102.000	6.252	3.745

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    5
                                    ETABS_FILE:EXSTL.PST/STEELER FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 SECTION PROPERTIES FOR BEAMS

       SECTION             MAT     DEPTH     DEPTH      BEAM   FLANGE       WEB
   ID  TYPE                 ID     BELOW     ABOVE     WIDTH    THICK     THICK
                                    (in)      (in)      (in)     (in)      (in)

    1  W18X50                1    17.990     0.000     7.495    0.570     0.355
    2  W33X118               1    32.860     0.000    11.480    0.740     0.550

 ANALYSIS SECTION PROPERTIES FOR   BEAMS

             AXIAL        MAJOR        MINOR      TORSION        MAJOR       MINOR
   ID            A           AV           AV            J            I           I
            (in_2)       (in_2)       (in 2)       (in_4)       (in_4)      (in_4)

    1       14.700        6.386        7.120  0.1240E+01  0.8000E+03 0.4010E+02
    2       34.700       18.073       14.159  0.5300E+01  0.5900E+04 0.1870E+03

 STRESS CHECK SECTION PROPERTIES FOR   BEAMS

             MAJOR        MINOR        MAJOR        MINOR        MAJOR       MINOR
   ID            S            S            Z            Z            R           R
            (in 3)       (in_3)       (in_3)       (in_3)         (in)        (in)

    1       88.938       10.700      101.000       16.600        7.377       1.652
    2      359.099       32.578      415.000       51.300       13.040       2.321
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    6
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 FRAME NUMBER--------------------------------      1
 FRAMING TYPE--------------------------------      2 (SPECIAL MOMENT)

 COLUMN PROPERTY REPLACEMENT CODE-----------       0
 BEAM PROPERTY REPLACEMENT CODE-------------       1
 BRACE PROPERTY REPLACEMENT CODE------------       0

 UBC STRUCTURE TYPE FACTOR, RW--------------      12

 DOUBLER PLATE CRITERION FLAG---------------       0


 FRAME ID NUMBER----------------------------       1

 NUMBER OF COLUMN LINES---------------------       8
 NUMBER OF BEAM BAYS------------------------       8
 NUMBER OF FLOOR BAYS-----------------------       0

 NUMBER OF JOINT LOAD PATTERNS--------------       0
 NUMBER OF BEAM SPAN LOAD PATTERNS----------       2
 NUMBER OF FLOOR SURFACE LOAD PATTERNS------       0

 MAXIMUM NUMBER OF BRACE ELEMENTS-----------       0
 MAXIMUM NUMBER OF PANEL ELEMENTS-----------       0
 MAXIMUM NUMBER OF LINK  ELEMENTS-----------       0

 MAXIMUM NUMBER OF LOADS PER BEAM SPAN------       4


 SPECIFIED BEAM PROPERTY ID*S

 LEVEL         1   2   3   4   5   6   7   8

 ROOF          1   1   2   2   2   2   2   2
 FLOOR         1   1   0   0   2   2   2   2


 SPECIFIED BEAM LIVE LOAD REDUCTION FACTORS

 ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       1.000


 SPECIFIED BEAM MAJOR DIRECTION K-FACTORS

 ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       0.000


 SPECIFIED BEAM MINOR DIRECTION K-FACTORS

 ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       0.000


 SPECIFIED BEAM MAJOR DIRECTION LENGTHS (in)

 ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       0.000
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    7
                                   ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND


 SPECIFIED BEAM MINOR DIRECTION LENGTHS (in)

 LEVEL          1     2     3     4     5     6     7     8

 ROOF       120.0 120.0 120.0 120.0   0.0   0.0   0.0   0.0
 FLOOR      120.0 120.0               0.0   0.0   0.0   0.0


 SPECIFIED BEAM MAJOR DIRECTION Cm-FACTORS

 ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS      0.000


 SPECIFIED BEAM MINOR DIRECTION Cm-FACTORS

 LEVEL          1     2     3     4     5     6     7     8

 ROOF       1.000 1.000 1.000 1.000 0.000 0.000 0.000 0.000
 FLOOR      1.000 1.000             0.000 0.000 0.000 0.000


 SPECIFIED BEAM Cb-FACTORS

 LEVEL          1     2     3     4     5     6     7     8

 ROOF       1.000 1.000 1.000 1.000 0.000 0.000 0.000 0.000
 FLOOR      1.000 1.000             0.000 0.000 0.000 0.000
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    8
                                 ETABS_FILE:EXSTL.PST/STEELER FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

  COL SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CONT-PL DBLR-PL SECTION
   ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION   AXIAL    AREA   THICK    TYPE
                                                    (K)  (sqin)   (in)

    1  W14X120                                              1.3    0.00 SEISMIC
               (C)   0.232 BOTTOM < 1> H1-3        22.9
    2  W14X120                                              1.3    0.00 SEISMIC
               (C)   0.096 TOP    < 3> H1-3        48.8
    3  W14X120                                              1.3    0.00 SEISMIC
               (C)   0.232 BOTTOM < 1> H1-3        22.9
    4  W14X120                                              3.3    0.00 SEISMIC
               (C)   0.053 BOTTOM < 4> H1-3         3.9
    5  W14X120                                              3.3    0.00 SEISMIC
               (C)   0.053 BOTTOM < 4> H1-3         3.9
    6  W14X120                                              3.3    0.00 SEISMIC
               (C)   0.130 TOP    < 3> H1-3        20.5
    7  W14X120                                              3.3    0.00 SEISMIC
               (C)   0.147 TOP    < 3> H1-3        57.7
    8  W14X120                                              3.3    0.00 SEISMIC
               (C)   0.130 TOP    < 2> H1-3        20.5
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    9
                                   ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

  COL SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CONT-PL DBLR-PL SECTION
   ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION  AXIAL    AREA   THICK    TYPE
                                                   (K)  (sqin)   (in)

    1  W14X120                                              1.3    0.00 SEISMIC
                (C)   0.188 BOTTOM < 4> H1-3        47.2
    2  W14X120                                              1.3    0.00 SEISMIC
                (C)   0.204 BOTTOM < 3> H1-3        97.4
    3  W14X120                                              1.3    0.00 SEISMIC
                (C)   0.188 BOTTOM < 4> H1-3        47.2
    4  W14X120                                              3.3    0.00 SEISMIC
                (C)   0.103 BOTTOM < 4> H1-3         8.1
    5  W14X120                                              3.3    0.00 SEISMIC
                (C)   0.103 BOTTOM < 4> H1-3         8.1
    6  W14X120                                              0.0    0.00 SEISMIC
                (C)   0.159 BOTTOM < 5> H1-3        22.0
    7  W14X120                                              0.0    0.00 SEISMIC
                (C)   0.176 BOTTOM < 3> H1-3        59.0
    8  W14X120                                              0.0    0.00 SEISMIC
                (C)   0.159 BOTTOM < 5> H1-3        22.0
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   10
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

 BEAM SECTION CHECK STRESS STRESS      AISC/UBC MAXIMUM CON-SHR CON-SHR SECTION
   ID  TYPE     TYPE  RATIO POINT  <LC> EQUATION   AXIAL   END-I  END-J    TYPE
                                                     (K)     (K)    (K)

    1  W18X50                                               42.5   44.3 SEISMIC
                 (T)   0.690 END-J  < 1> BENDING      0.0
    2  W18X50                                               44.3   42.5 SEISMIC
                 (T)   0.690 END-I  < 1> BENDING      0.0
    3  W33X118                                             105.4  114.1 SEISMIC
                 (T)   0.202 END-J  < 1> BENDING      0.0
    4  W33X118                                             114.1  105.4 SEISMIC
                 (T)   0.202 END-I  < 1> BENDING      0.0
    5  W33X118                                             180.8  182.3 SEISMIC
                 (T)   0.026 END-J  < 5> BENDING      0.0
    6  W33X118                                             182.3  180.8 SEISMIC
                 (T)   0.026 END-I  < 4> BENDING      0.0
    7  W33X118                                             180.8  182.3 SEISMIC
                 (T)   0.026 END-J  < 5> BENDING      0.0
    8  W33X118                                             182.3  180.8 SEISMIC
                 (T)   0.026 END-I  < 4> BENDING      0.0
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   11
                                        ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

 BEAM SECTION CHECK STRESS STRESS       AISC/UBC MAXIMUM CON-SHR CON-SHR SECTION
   ID  TYPE     TYPE  RATIO POINT  <LC> EQUATION   AXIAL   END-I   END-J    TYPE
                                                     (K)     (K)     (K)

    1  W18X50                                                42.9    43.8 SEISMIC
                (T)   0.664 END-J  < 1> BENDING      0.0
    2  W18X50                                                43.8    42.9 SEISMIC
                (T)   0.664 END-I  < 1> BENDING      0.0
    5  W33X118                                              180.9   182.1 SEISMIC
                (T)   0.045 END-J  < 5> BENDING      0.0
    6  W33X118                                              182.1   180.9 SEISMIC
                (C)   0.038 END-I  < 5> H1-3         0.7
                (T)   0.055 END-I  < 4> H2-1         0.5
    7  W33X118                                              180.9   182.1 SEISMIC
                (T)   0.045 END-J  < 5> BENDING      0.0
    8  W33X118                                              182.1   180.9 SEISMIC
                (C)   0.038 END-I  < 5> H1-3         0.7
                (T)   0.055 END-I  < 4> H2-1         0.5
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                   ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

FRAME ID .... /MAIN FRAME
LEVEL ID .... ROOF

UNIFORM BUILDING CODE 1994 (CHAPTER 22)
COLUMN INTERACTION DETAILS

 COL CHECK  <---BREAK DOWN---><-K-FACTORS-><-UNSUP LENGTHS-><--B/C  RATIOS-->
  ID TYPE   AXIAL MAJOR MINOR  MAJOR MINOR    MAJOR   MINOR   MAJOR   MINOR
                                               (in)    (in)

   1                            2.55  1.06   102.01   87.14    0.49    4.15
     (C)    0.034 0.191 0.007
   2                            1.94  1.00   102.01  120.00    0.99    0.00
     (C)    0.052 0.044 0.000
   3                            2.55  1.06   102.01   87.14    0.49    4.15
     (C)    0.034 0.191 0.007
   4                            1.08  1.00    87.14  120.00    3.93    0.00
     (C)    0.004 0.048 0.000
   5                            1.08  1.00    87.14  120.00    3.93    0.00
     (C)    0.004 0.048 0.000
   6                            1.21  1.06   231.14   87.14    1.99    4.14
     (C)    0.025 0.102 0.003
   7                            1.18  1.00   231.14  264.00    4.10    0.00
     (C)    0.075 0.072 0.000
   8                            1.21  1.06   231.14   87.14    1.99    4.14
     (C)    0.025 0.102 0.003
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS   PAGE    2
                                      ETABS FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN INTERACTION DETAILS

  COL CHECK  <---BREAK DOWN---><-K-FACTORS-><-UNSUP LENGTHS-><--B/C  RATIOS-->
   ID TYPE   AXIAL MAJOR MINOR  MAJOR MINOR    MAJOR    MINOR   MAJOR    MINOR
                                                (in)     (in)

    1                             1.89  1.20   126.01   111.14    0.25     2.10
      (C)    0.058 0.029 0.100
    2                             1.68  1.00   126.01   144.00    0.51     0.00
      (C)    0.107 0.097 0.000
    3                             1.89  1.20   126.01   111.14    0.25     2.10
      (C)    0.058 0.029 0.100
    4                             1.21  1.00   111.14   144.00    1.97     0.00
      (C)    0.009 0.094 0.000
    5                             1.21  1.00   111.14   144.00    1.97     0.00
      (C)    0.009 0.094 0.000
    6                             1.21  1.20   231.14   111.14    0.00     2.08
      (C)    0.032 0.029 0.099
    7                             1.18  1.00   231.14   264.00    0.00     0.00
      (C)    0.076 0.099 0.000
    8                             1.21  1.20   231.14   111.14    0.00     2.08
      (C)    0.032 0.029 0.099
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     3
                                        ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   INTERACTION DETAILS

 BEAM CHECK  <---BREAK DOWN---><-K-FACTORS-><-UNSUP LENGTHS->
   ID TYPE   AXIAL MAJOR MINOR  MAJOR MINOR    MAJOR   MINOR
                                                (in)    (in)

    1                            1.00  1.00   345.52  120.00
      (T)    0.000 0.690 0.000
    2                            1.00  1.00   345.52  120.00
      (T)    0.000 0.690 0.000
    3                            1.00  1.00   345.52  120.00
      (T)    0.000 0.202 0.000
    4                            1.00  1.00   345.52  120.00
      (T)    0.000 0.202 0.000
    5                            1.00  1.00   165.43  165.43
      (T)    0.000 0.026 0.000
    6                            1.00  1.00   165.43  165.43
      (T)    0.000 0.026 0.000
    7                            1.00  1.00   165.43  165.43
      (T)    0.000 0.026 0.000
    8                            1.00  1.00   165.43  165.43
      (T)    0.000 0.026 0.000
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                                    ETABS FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   INTERACTION DETAILS

 BEAM CHECK  <---BREAK DOWN---><-K-FACTORS-><-UNSUP LENGTHS->
   ID TYPE   AXIAL MAJOR MINOR  MAJOR MINOR     MAJOR    MINOR
                                                 (in)     (in)

    1                            1.00  1.00    345.52   120.00
     (T)     0.000 0.664 0.000
    2                            1.00  1.00    345.52   120.00
     (T)     0.000 0.664 0.000
    5                            1.00  1.00    165.43   165.43
     (T)     0.000 0.045 0.000
    6                            1.00  1.00    165.43   165.43
     (C)     0.001 0.027 0.010
     (T)     0.001 0.044 0.010
    7                            1.00  1.00    165.43   165.43
     (T)     0.000 0.045 0.000
    8                            1.00  1.00    165.43   165.43
     (C)     0.001 0.027 0.010
     (T)     0.001 0.044 0.010
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

FRAME ID .... /MAIN FRAME
LEVEL ID .... ROOF

UNIFORM BUILDING CODE 1994 (CHAPTER 22)
COLUMN SHEAR STRESS CHECK

       <---MAJOR  SHEAR---> <---MINOR  SHEAR--->
 COL     STRESS STRESS           STRESS STRESS
  ID      RATIO POINT  <LC>       RATIO POINT  <LC>

   1      0.138 BOTTOM < 1>      0.006 BOTTOM < 5>

   2      0.030 BOTTOM < 7>      0.001 BOTTOM < 5>

   3      0.138 BOTTOM < 1>      0.006 BOTTOM < 5>

   4      0.032 BOTTOM < 9>      0.001 BOTTOM < 2>

   5      0.032 BOTTOM < 9>      0.001 BOTTOM < 3>

   6      0.034 BOTTOM < 3>      0.006 BOTTOM < 4>

   7      0.025 BOTTOM < 7>      0.001 BOTTOM < 4>

   8      0.034 BOTTOM < 2>      0.006 BOTTOM < 4>
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     2
                                          ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 COLUMN SHEAR STRESS CHECK

        <---MAJOR  SHEAR---> <---MINOR  SHEAR--->
  COL     STRESS STRESS          STRESS STRESS
   ID      RATIO POINT  <LC>      RATIO POINT  <LC>

    1      0.056 BOTTOM < 3>      0.008 BOTTOM < 5>

    2      0.035 BOTTOM < 7>      0.004 BOTTOM < 4>

    3      0.056 BOTTOM < 2>      0.008 BOTTOM < 5>

    4      0.046 BOTTOM < 5>      0.004 BOTTOM < 2>

    5      0.046 BOTTOM < 5>      0.004 BOTTOM < 3>

    6      0.034 BOTTOM < 3>      0.008 BOTTOM < 4>

    7      0.025 BOTTOM < 7>      0.001 BOTTOM < 4>

    8      0.034 BOTTOM < 2>      0.008 BOTTOM < 4>
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... ROOF

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   SHEAR STRESS CHECK

        <---MAJOR  SHEAR---> <---MINOR  SHEAR--->
 BEAM     STRESS STRESS         STRESS STRESS
   ID      RATIO POINT  <LC>     RATIO POINT  <LC>

    1      0.252 END-J  < 1>     0.000 END-J  < 9>

    2      0.252 END-I  < 1>     0.000 END-J  < 9>

    3      0.106 END-J  < 1>     0.000 END-J  < 9>

    4      0.106 END-I  < 1>     0.000 END-J  < 9>

    5      0.010 END-J  < 5>     0.000 END-J  < 9>

    6      0.010 END-I  < 4>     0.000 END-J  < 9>

    7      0.010 END-J  < 5>     0.000 END-J  < 9>

    8      0.010 END-I  < 4>     0.000 END-J  < 9>
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                          ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME
 LEVEL ID .... FLOOR

 UNIFORM BUILDING CODE 1994 (CHAPTER 22)
 BEAM   SHEAR STRESS CHECK

        <---MAJOR  SHEAR---> <---MINOR  SHEAR--->
 BEAM     STRESS STRESS         STRESS STRESS
   ID      RATIO POINT  <LC>     RATIO POINT  <LC>

    1      0.247 END-J  < 1>     0.000 END-J  < 9>

    2      0.247 END-I  < 1>     0.000 END-J  < 9>

    5      0.017 END-J  < 5>     0.000 END-J  < 9>

    6      0.017 END-I  < 4>     0.000 END-J  < 1>

    7      0.017 END-J  < 5>     0.000 END-J  < 9>

    8      0.017 END-I  < 4>     0.000 END-J  < 1>
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS   PAGE     1
                                         ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME

 UNIFORM BUILDING CODE 1994
 MAP OF CONTROLLING COLUMN INTERACTION STRESS RATIOS

          COLUMN        1       2       3       4       5       6       7      8
 LEVEL

 ROOF                0.232   0.096   0.232   0.053   0.053   0.130   0.147 0.130

 FLOOR               0.188   0.204   0.188   0.103   0.103   0.159   0.176 0.159

 LEVEL
          COLUMN        1       2       3       4       5       6       7      8
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    2
                                   ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME

 UNIFORM BUILDING CODE 1994
 MAP OF COLUMN CONTINUITY PLATE AREA REQUIREMENTS (PER BEAM FLANGE)(sqin)

         COLUMN        1      2      3      4      5      6      7     8
 LEVEL

 ROOF                1.3    1.3    1.3    3.3    3.3    3.3    3.3   3.3

 FLOOR               1.3    1.3    1.3    3.3    3.3    0.0    0.0   0.0

 LEVEL
         COLUMN        1      2      3      4      5      6      7     8
```

Sample Output from ***STEELER*** *(continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                         ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME

 UNIFORM BUILDING CODE 1994
 MAP OF COLUMN DOUBLER PLATE THICKNESS REQUIREMENTS (in)

          COLUMN        1       2       3       4       5       6       7      8
 LEVEL

 ROOF                0.00    0.00    0.00    0.00    0.00    0.00    0.00   0.00

 FLOOR               0.00    0.00    0.00    0.00    0.00    0.00    0.00   0.00

 LEVEL
          COLUMN        1       2       3       4       5       6       7      8
```

*Sample Output from **STEELER** (continued)*

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS   PAGE    4
                                       ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
 FILE : CHKSTL  SAMPLE EXAMPLE FOR STEELER MANUAL
 SPECIAL MOMENT RESISTING STEEL FRAME UNITS : KIP-INCH-SECOND

 FRAME ID .... /MAIN FRAME

 UNIFORM BUILDING CODE 1994
 MAP OF CONTROLLING BEAM INTERACTION STRESS RATIOS

            BEAM         1        2        3        4        5        6        7      8
 LEVEL

 ROOF                0.690  0.690  0.202  0.202  0.026  0.026  0.026 0.026

 FLOOR               0.664  0.664                  0.045  0.055  0.045 0.055

 LEVEL
            BEAM         1        2        3        4        5        6        7      8
```

*Sample Output from **STEELER** (continued)*

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