COMPUTERS & STRUCTURES INC. ®

STEELER®

Stress Check of Steel Frames

A Postprocessor for ETABS®

by

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Developed and written in U.S.A.

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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 STRUCTURES. PREVIOUS VERSIONS OF THIS PROGRAM HAVE BEEN VERY SUCCESSFULLY USED ON A WIDE 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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I.

INTRODUCTION

STEELER is a steel stress check/design postprocessor for the three-dimensional static and dynamic building analysis computer program ETABS [1].

The program is intended for the automated code check/design based upon either the Uniform Building Code (UBC) [2], or the American Institute of Steel Construction (AISC) Allowable Stress Design (ASD) Specifications [3], or the AISC Plastic Design Specifications [3], or the AISC Load and Resistance Factor Design (LRFD) Specifications [4], or the Canadian Standards Association Limit States Design of Steel Structures (CISC) Specifications [5].

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

1. Ordinary Moment Resisting Space Frames
2. Special Moment Resisting Space Frames
3. Braced Frames
4. Eccentrically 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 resisting frame-type structures are automated in the algorithm.

The program will perform a joint shear analysis to determine if doubler plates are required in any of the joint panel zones. Requirements for continuity plates at the beam to column connections are evaluated.

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 space frames.

All capacity requirements associated with seismic framing systems that require ductility are satisfied.

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 postprocessor PLOTTER [7].

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

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

II.

SYSTEM PREPARATION AND EXECUTION PROCEDURE

This chapter deals with the installation and execution of STEELER on an MS-DOS based computer system.

User familiarity with MS-DOS is assumed.

The complete STEELER package includes:

a. This manual

b. Floppy disk, containing the following:
   1. Program Executable, STEELER.EXE
   2. Sample Test Data and Results

Note: the characters <CR> appear repeatedly in the text of this chapter. These characters mean "press the carriage return key." Do not type the characters <, C, R and >.

A. INSTALLING, CONFIGURING AND TESTING

The program provided must first be copied to the hard disk. The program and computer must then be configured before the program can be used. Follow the instructions in the SAP90/ETABS/SAFE Installation Guide (included with the ETABS package) for the 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 postprocessing file.

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

The user must then also prepare a STEELER input data file using a 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 of the 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.

Say that ETABS has been run using an input data file named EXSTL, to create the postprocessing file EXSTL.PST, and a corresponding STEELER input data file named CHKSTL has been prepared. In order to execute the STEELER program, proceed as follows:

From the directory where the STEELER input data file is resident, enter the command:

STEELER <CR>

Note: the STEELER input data file and the ETABS postprocessing file, EXSTL.PST, must always exist in the same directory. The STEELER executable and the file of AISC properties, AISC.DAT must also reside in the same directory unless a path to the STEELER executable has been activated using the MS-DOS **PATH** command and the AISC.DAT file is in the same directory as the executable.

After a few seconds the following banner will appear on the screen:

```
                    STEEL STRESS CHECK OF BUILDING SYSTEMS
                               VERSION  5.xx
                                    BY
                             ASHRAF HABIBULLAH

                          Copyright (c) 1984-1992
                       COMPUTERS AND STRUCTURES, INC.
                            All rights reserved

                                                    hit it baby...<CR>
```

Enter **<CR>**

The program will then display a copyright notice followed by a prompt for the STEELER input data filename as follows:

```
             THE USE OF THIS PROGRAM IS GOVERNED BY THE TERMS
             OF A LICENSE AGREEMENT AND THE PROGRAM IS TO BE
                   USED ONLY BY AUTHORIZED LICENSEES.

              UNAUTHORIZED USE IS UNETHICAL, UNPROFESSIONAL
               AND IN VIOLATION OF FEDERAL COPYRIGHT LAWS.

                ADDITIONAL INFORMATION MAY BE OBTAINED FROM

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                       BERKELEY, CALIFORNIA   94704

                           TEL:  (510) 845-2177
                           FAX:  (510) 845-4096

              IT IS THE RESPONSIBILITY OF THE USER TO VERIFY
                  ALL RESULTS PRODUCED BY THIS PROGRAM.


 ENTER "STEELER" INPUT FILE NAME :
```

Enter **CHKSTL <CR>**

Note: If program defaults are acceptable and no STEELER input is required, simply enter a **<CR>** above.

The program will then ask for the ETABS postprocessing file as follows:

```
ENTER "ETABS"   .PST FILENAME:
```

Enter **EXSTL.PST <CR>**

Note: the postprocessing file name must have a .PST extension. If a user enters a filename with no extension, the program will append the .PST extension to the filename.

The program will then display the input and output filenames as follows:

```
STEELER INPUT FILE---------- CHKSTL
ETABS POSTPROCESSING FILE--- EXSTL.PST
INTERACTION STRESS CHECKS--- CHKSTL.STL
DETAILS OF INTERACTION------ CHKSTL.DTL
SHEAR STRESS CHECKS--------- CHKSTL.SHR
OUTPUT DISPLAY MAPS--------- CHKSTL.MAP
STEELER PLOT FILE----------- CHKSTL.PLO

              <CR> TO CONTINUE
```

If all of the filenames are appropriate,

Enter **<CR>**

The program will go into execution mode and a series of progress messages will be flashed to the screen until the job has been completed. The job completion message will read

```
JOB COMPLETED ....... NO CHARGE !!!
```

The output files created by the program are explained in Chapter V. To print an output file the MS-DOS **PRINT** command may be used. Appropriate line counts and page ejects are built into the files.

III.

STRESS CHECK AND DESIGN ALGORITHMS

(AISC-ASD 1989)

This chapter describes in detail the various aspects of the stress check procedures that are used by the program STEELER.

This chapter is based on the requirements of the American Institute of Steel Construction's Allowable Stress Design Specifications (AISC-ASD89) [3]. The additional seismic requirements in the Uniform Building Code (UBC91) [2] checked by the program are detailed in Appendix A. Equivalent chapters for the AISC Plastic 89 [3], for the AISC-LRFD86 [4] and the CISC89 [5] specifications are presented in other appendices.

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 is assumed.

References to pertinent sections and equations of the AISC-ASD89 are indicated with the "AISC" prefix.

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 postprocessing file brings across forces and moments associated with the eight independent load conditions (I, II, III, A, B...) 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.

If a structure is subjected to dead load (DL) and live load (LL) only, the stress check may need only one load combination, namely DL + LL (AISC A4.1).

However, if in addition to the dead load 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.

It is possible in the postprocessing 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.

B. THE UNSUPPORTED LENGTHS OF THE ELEMENTS

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

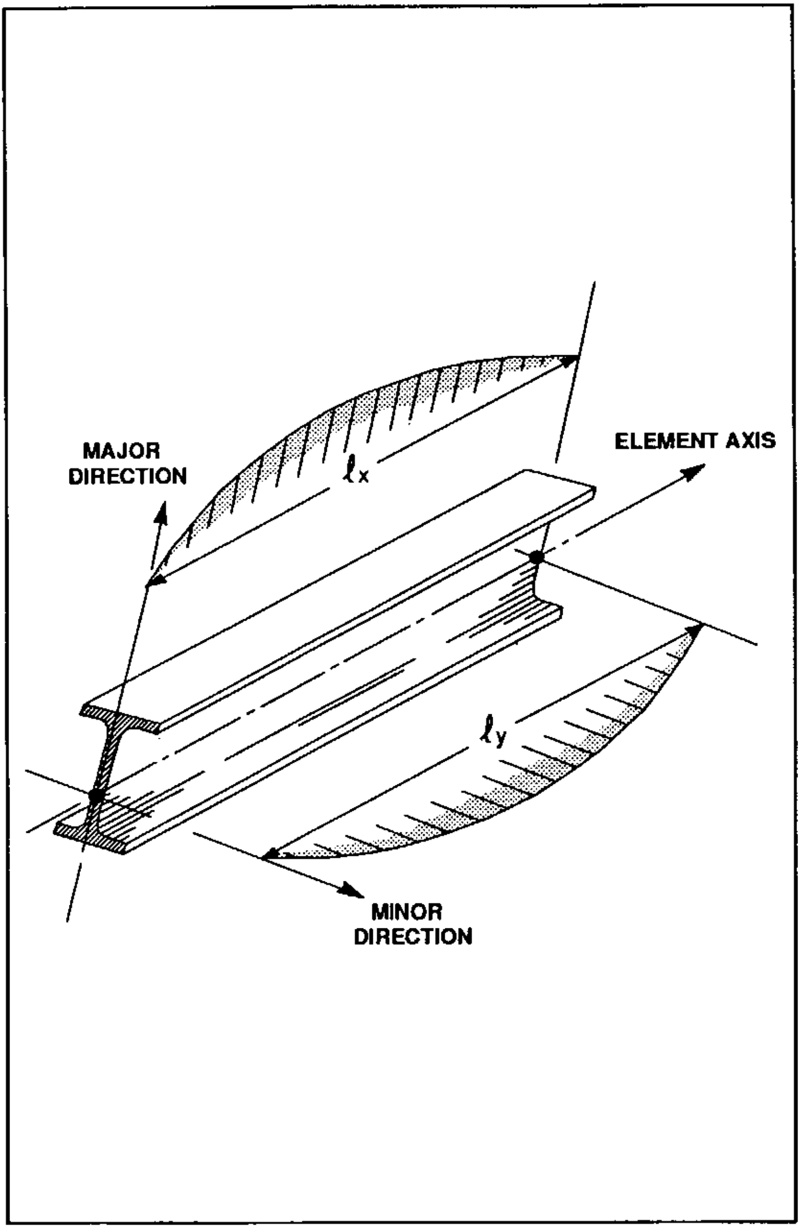
The two unsupported lengths are l_x and l_y, corresponding to instability in the major and minor directions of the element, respectively. See Figure III-1. These are the lengths between the support points of the element in the corresponding directions.

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.

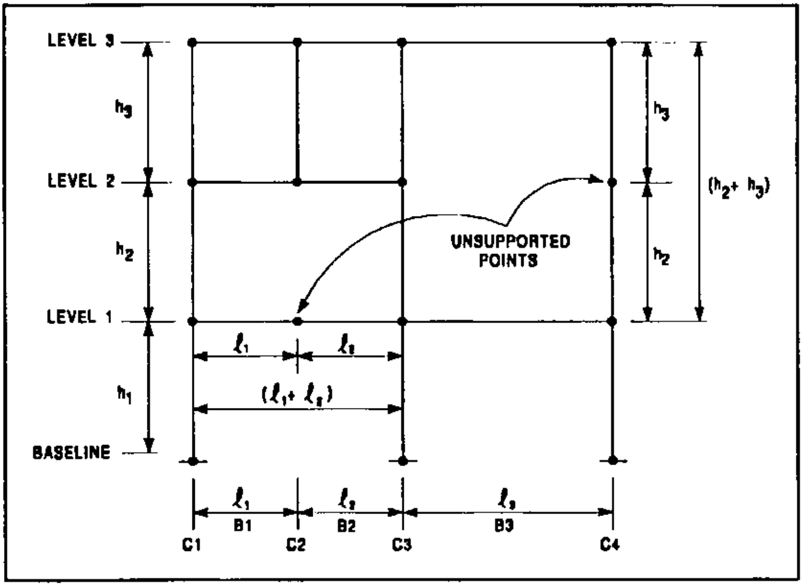
Therefore, the unsupported length of a column may actually be evaluated as being greater than the corresponding story height, and the unsupported length of a beam may actually be evaluated as being greater than the bay length.

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



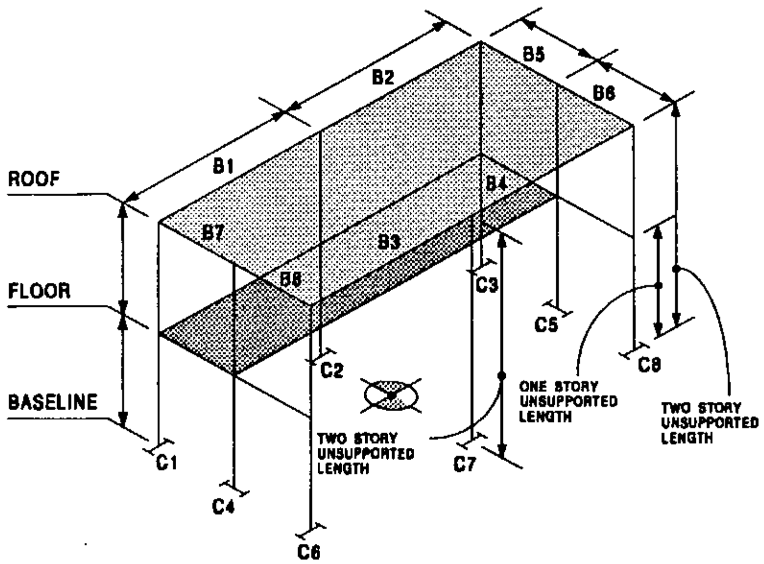
Element Unsupported Length
Figure III-1



Tracing Unsupported Element Lengths
Figure III-2

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



Conditions Affecting Unsupported Lengths
Figure III-3

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. THE 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:

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

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.

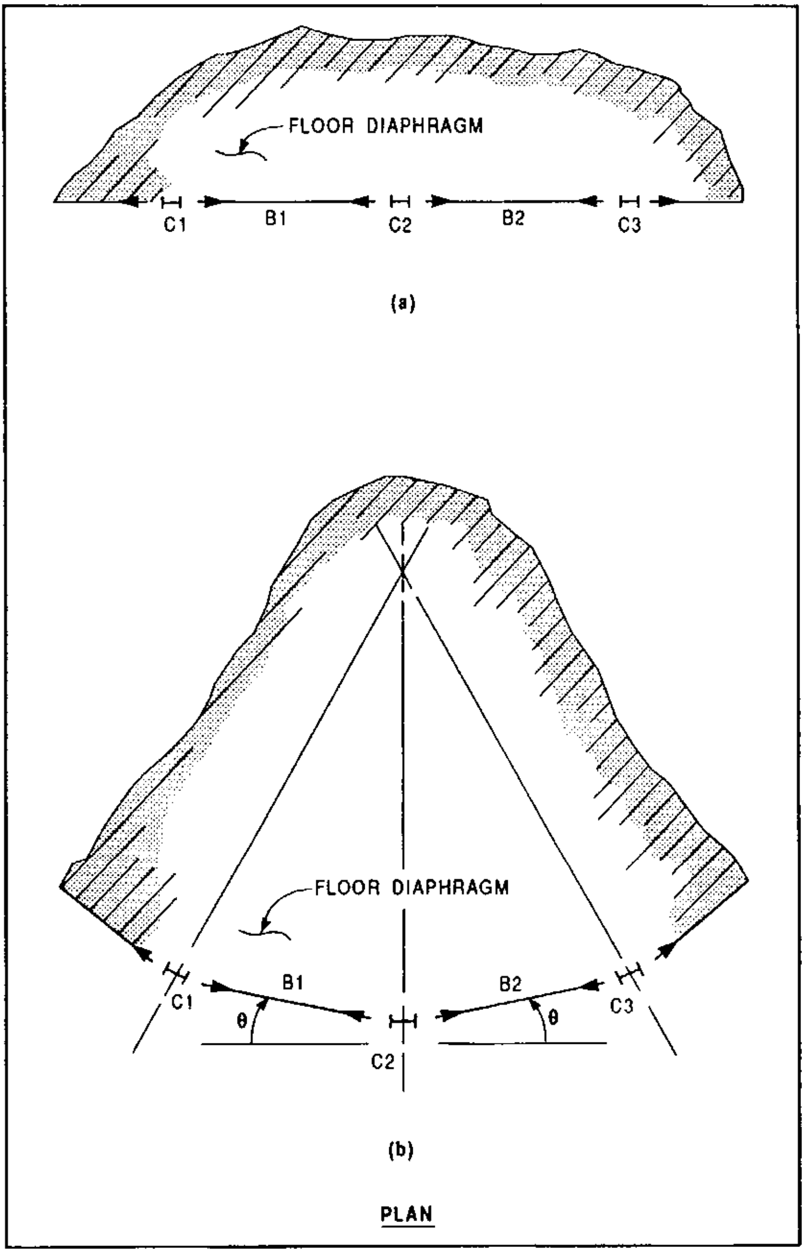
Cantilever beams and beams and columns having pin ends at the joint under consideration are excluded in the calculation of the stiffness EI/L summations.

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 in both directions.

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.



K-Factor Ambiguity
Figure III-4

If rotational releases exist at both ends of a column, the corresponding K-factors are set to unity.

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

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

For example, in Figure III-4(a) 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 Figure III-4(b) for column line C2. The small components of the beam stiffnesses 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.

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 option if the user wants to use inelastic K-factors as opposed to the elastic K-factors calculated by the program.

D. THE 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 diaphragm at the level of the beam.

E. THE STRESS CHECK AND DESIGN PROCEDURE

This section defines the stress 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-ASD89 [3]. Various parameters referenced 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.

A	=	Cross-sectional area, in^2
$\mathbf{A_f}$	=	Area of compression flange , in^2
$\mathbf{A_{vx}, A_{vy}}$	=	Major and minor shear areas, in^2
$\mathbf{C_b}$	=	Bending Coefficient
$\mathbf{C_m}$	=	Moment Coefficient
D	=	Outside diameter of pipes, in
E	=	Modulus of elasticity, ksi
$\mathbf{F_a}$	=	Allowable axial compressive stress, ksi
$\mathbf{F_{bx}, F_{by}}$	=	Allowable major and minor bending stresses, ksi
$\mathbf{F'_{ex}}$	=	$\dfrac{12\,\pi^2 E}{23\,(K_x l_x / r_x)^2}$
$\mathbf{F'_{ey}}$	=	$\dfrac{12\,\pi^2 E}{23\,(K_y l_y / r_y)^2}$
$\mathbf{F_t}$	=	Allowable axial tensile stress, ksi
$\mathbf{F_v}$	=	Allowable shear 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
$\mathbf{M_x, M_y}$	=	Factored major and minor moments in member, kip-in
P	=	Factored axial force in member, kips
$\mathbf{S_x, S_y}$	=	Major and minor section moduli, in^3
$\mathbf{V_x, V_y}$	=	Factored major and minor shear forces, kips
b	=	Nominal dimension of longer leg of angles, in
	=	$\mathbf{b_f - 2t_w}$ for welded BOX sections, in

Stress Check Parameters
Figure III-5

	=	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

Stress Check Parameters
Figure III-5 (continued)

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:

$$f_a = P/A$$

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

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

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

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

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

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

Compact Section Criteria
Figure III-6

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

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

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

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

otherwise, if $Kl/r > C_c$

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

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

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

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

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

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

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

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

$$F_{bx} = \frac{12 x 10^3\, C_b}{l_y\left(\frac{d}{A_f}\right)} \leq 0.6\, F_y \qquad \text{(AISC Eqn.F1-8)}$$

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:

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

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

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

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

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

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

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

$$\text{or} \quad 1200 \frac{b}{F_y}$$

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:

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

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

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

provided the section is COMPACT, otherwise

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

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 0.40 F_y (AISC Eqn. 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

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

(AISC Eqn. H1-1)

and

$$CR1b = \frac{f_a}{0.60\, F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

(AISC Eqn. H1-2)

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

$$CR2 = \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

(AISC Eqn. 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

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

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.

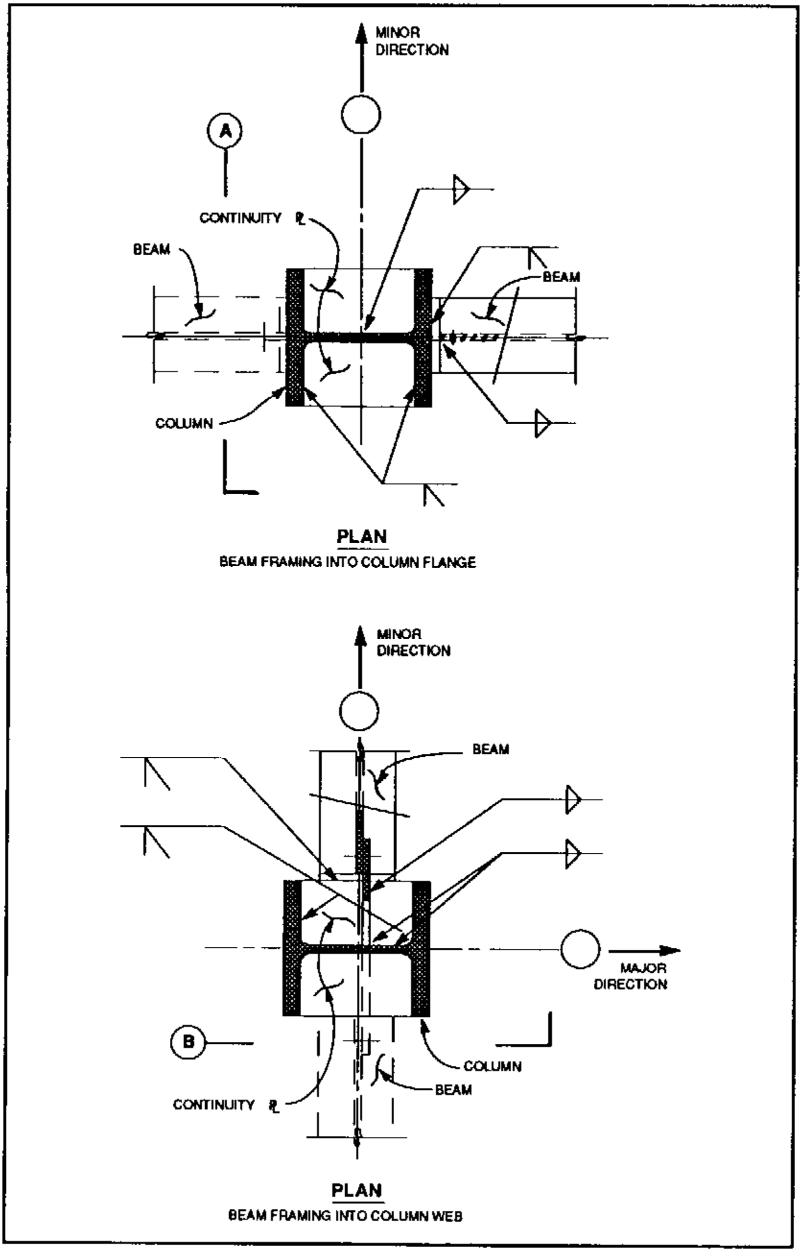
c) The steel beam frames in a direction that is at an angle to both of the principal axes of the column, i.e. the beam frames partially into the column web and partially into the column flange.

To achieve a beam/column moment connection, continuity plates such as shown in Figures III-8 and III-9 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.

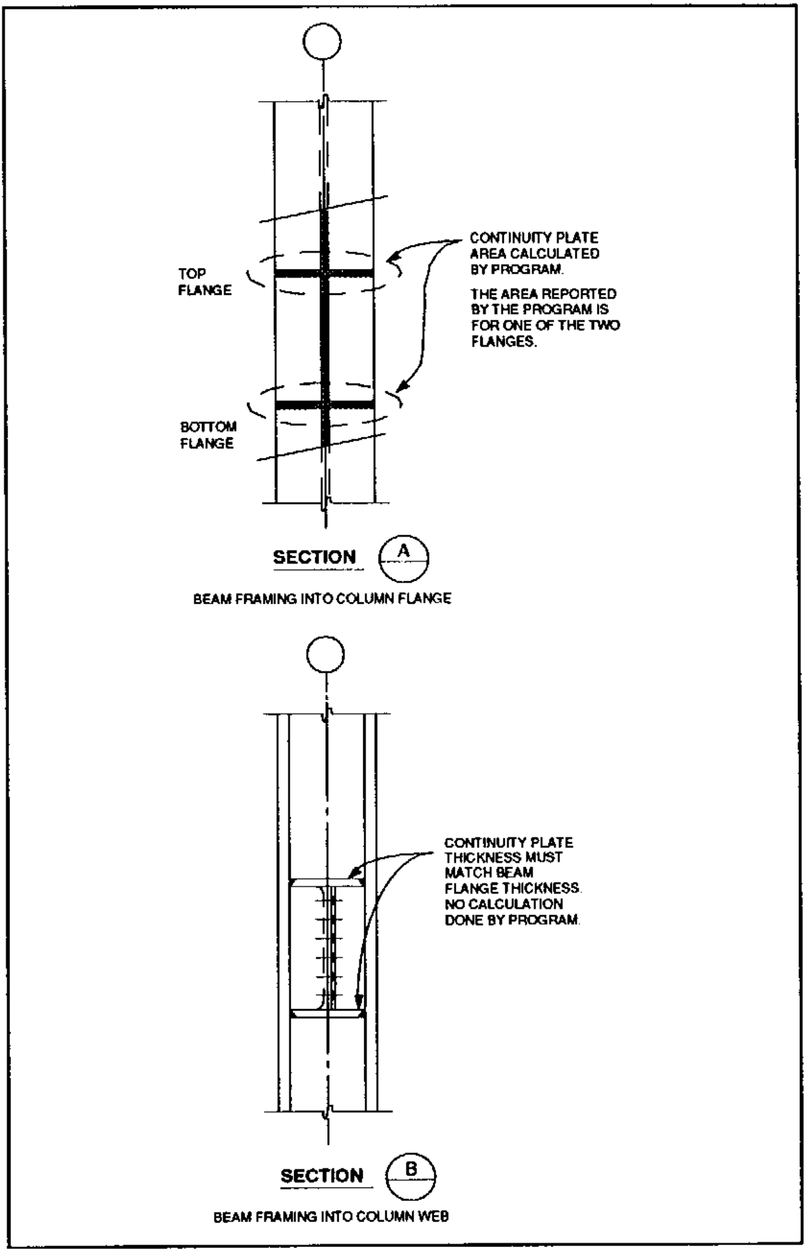
For connection conditions described by b) and c) above, the thickness of such plates is usually set equal to the flange thickness of the corresponding beam.

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 direction) and reports the maximum continuity plate area that is needed for each beam flange. See figure III-9. The continuity plate requirements are evaluated for moment frames only. No check is made for braced frames.



Plan Showing Continuity Plates
Figure III-8



Section Showing Continuity Plates
Figure III-9

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

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

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:

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

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

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

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 com-

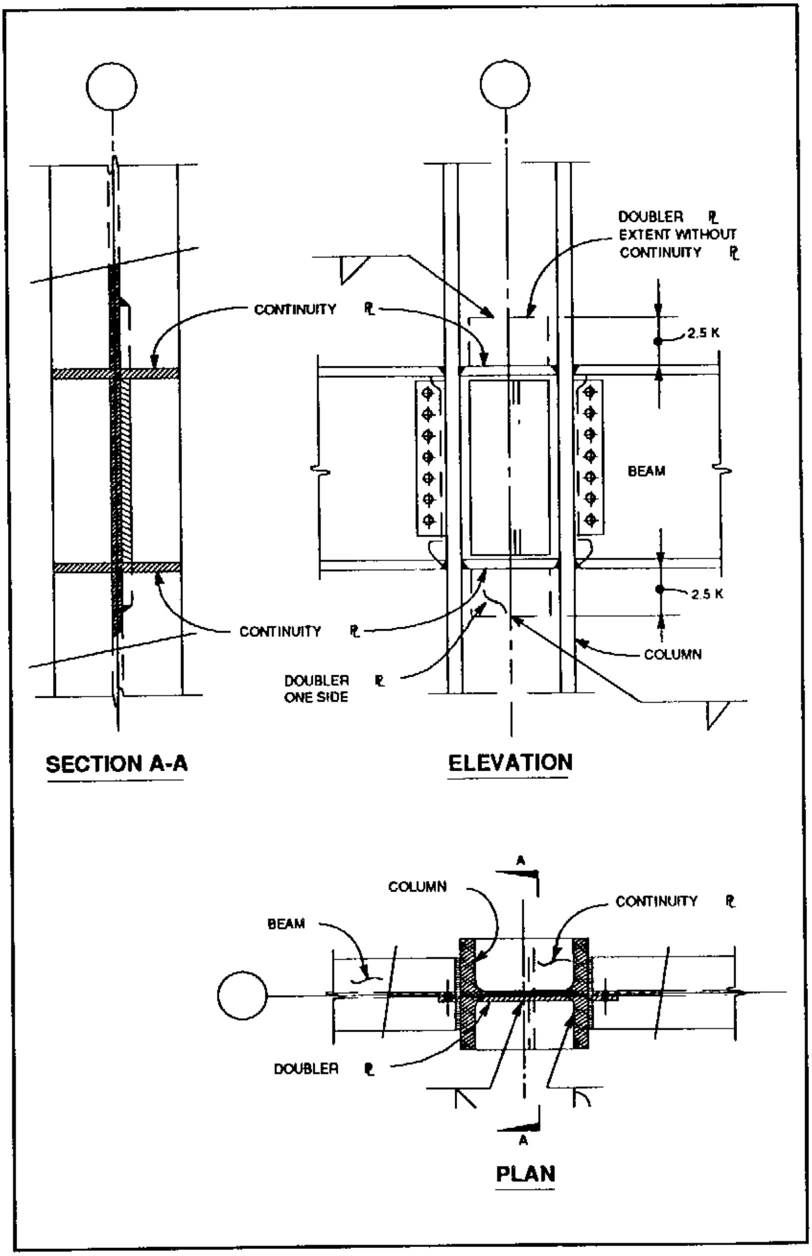
binations with lateral loads (AISC K1.2)
(The additional 4/3 in the above equation is included to account for the 0.75 factor that may be used by the user in the lateral load combination to account in turn for the 1/3rd increase in allowable stress for such conditions.)

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.

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-10. The program investigates such situations and reports the thicknesses of any required doubler plates. Only columns with I shapes are investigated for doubler plate requirements.



Doubler Plates
Figure III-10

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

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
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 Joint 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.

IV.

INPUT DATA FOR PROGRAM STEELER

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 UBC code and special moment resisting type frames.

The ETABS postprocessing 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 sequence of data lines described herein will establish the data file required 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.

A. FREE FORMAT

All input data for STEELER is prepared in free format. In other words, the data on a particular line does not have to correspond to prespecified column locations. The data is input as a string of numbers which are separated by one or more blanks. **It is important to enter all items even if they are zero; however, trailing zeros on any data line need not be input.** No data line may be more than eighty characters in length. Also, the data file should not contain spacing tab characters that are generated by some editors while preparing a data file.

B. COMMENT DATA

Any line having a $ (dollar) sign in Column 1 is treated as a comment line and ignored by the program.

The $ sign may also be used in any other column on any data line. In such cases entries to the right of the $ sign will be treated as comment data and ignored by the program. This option allows the user to effectively comment the data.

C. ARITHMETIC OPERATIONS

Simple arithmetic statements are possible when entering floating point real numbers in the free format fields. The following type of operators can be used:

+	for addition
–	for subtraction
/	for division
*	for multiplication
P	for raising to the power of

The following are typical data entries that are possible:

11.92*12
7.63/386.4
3P.5
150P1.5*33
6.66-1.11*7.66/12.2

The operators are applied as they are encountered in the scan from left to right, so that

11.92*12 is evaluated as 11.92×12

7.63/386.4 is evaluated as $\frac{7.63}{386.4}$

3P.5 is evaluated as $3^{.5}$

150P1.5*33 is evaluated as $(150^{1.5}) \times 33$

6.66-1.11*7.66/12.2 is evaluated as $\frac{(6.66 - 1.11) \times 7.66}{12.2}$

Decimal points for whole floating point numbers are not necessary. For example, the number (6.0) may be entered as (6). Scientific exponential notation is also allowed. For example, the number 1.5E10 is read as 1.5×10^{10}.

D. UNITS

All input data for STEELER and the corresponding ETABS data must be prepared using either English or MKS metric or SI metric units.

If the **English** option is used, the input must be prepared using **inch-kip-second** units.

If the **MKS** metric option is used, the input must be prepared using **meter-kilogram-second** units.

If the **SI** metric option is used, the input must be prepared using **meter-kiloNewton-second** units.

This is irrespective of the fact that all the numerical techniques described in Chapter III or in the appendices are presented in inch-kip-second units or millimeter-Newton-second units.

E. ETABS DATA PREPARATION HINTS

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 ETABS Load Condition II and the lateral loads are in Load Conditions A,B, DYN-1, DYN-2 and DYN-3. 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-LRFD86 and the CISC89 specifications the STEELER program assumes that a P-Delta analysis has been made in ETABS. That is, the bending moments causing side-sway 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.**

F. DETAILED DESCRIPTION OF THE STEELER INPUT DATA

There are basically five data sections associated with the STEELER input. A summary of the data setup is shown in Figure IV-1. This chapter details each section and the associated data lines. A sample data file is listed in the last section of this manual.

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

First, 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 table 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 after each 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 (or entry).

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

Typical Data Setup for STEELER
Figure IV-1

1. CONTROL INFORMATION

Prepare the following data as defined in Sections a, b and c below. This data is always needed. A total of 4 data lines is required.

a. Heading Data

Prepare **two** lines of data for output labeling, up to 80 characters per line. This information will appear on every page.

b. Execution Control Information

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

ICODE NFR NLC LLC NRMP NRCP NRBP NRDP IUNIT IEX PROPFILE

c. Output Control Information

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

RI RS IDC IDB IDD ISC ISB ISD 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 UBC91 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 **NRDP** all default to zero; **IUNIT** defaults to E; **IEX** defaults to zero;

PROPFILE defaults to blank; **RI** and **RS** default to 0.005; **IDC**, **IDB**, **IDD**, **ISC**, **ISB**, **ISD**, **MCI**, **MBI**, **MCC**, and **MCD** all default to 1.

1. CONTROL INFORMATION

First Data Line

Variable	Field	Note	Entry
ICODE	1	(1)	Code Identifier: = 1 UBC91 (includes seismic) = 2 AISC-ASD89 = 3 AISC-Plastic 89 = 4 AISC-LRFD86 = 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

1. CONTROL INFORMATION (continued)

First Data Line (continued)

Variable	Field	Note	Entry
NRBP	7	(5,8)	Number of redefined (or new) beam section property types
NRDP	8	(5,9)	Number of redefined (or new) brace section property types
IUNIT	9	(10)	Type of units: = E English units = M MKS metric units = S SI metric units
IEX	10	(11)	Execution mode: = 0 Normal execution mode = 1 Data check mode
PROPFILE	11	(12)	User section property filename

Second Data Line

Variable	Field	Note	Entry
RI	1	(13)	Interaction stress ratio cutoff
RS	2	(13)	Shear stress ratio cutoff

1. CONTROL INFORMATION (continued)

Second Data Line (continued)

Variable	Field	Note	Entry
IDC	3	(14)	Flag for column interaction detail information
IDB	4	(14)	Flag for beam interaction detail information
IDD	5	(14)	Flag for brace interaction detail information
ISC	6	(15)	Flag for column shear stress check
ISB	7	(15)	Flag for beam shear stress check
ISD	8	(15)	Flag for brace shear stress check
MCI	9	(16)	Flag for map of column interaction stress ratios
MBI	10	(16)	Flag for map of beam interaction stress ratios
MCC	11	(16)	Flag for map of column continuity plates

1. CONTROL INFORMATION (continued)

Second Data Line (continued)

Variable	Field	Note	Entry
MCD	12	(16)	Flag for map of column doubler plates

1 - 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 checks used for the AISC-ASD89 code. The details of the additional checks for the UBC91 code are given in Appendix A. The details of the checks for the AISC Plastic Design (1989) code are given in Appendix B; for the AISC-LRFD86 code in Appendix C; and for the CISC89 code in Appendix D. The loading combinations specified in Section 2 later must correspond to the design level at which check is made.

   It is re-iterated here that only the design checks listed in Chapter III or the appropriate appendices 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 separately addressed 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 given in Section 2 are used.

4. This entry defines the vertical load condition of ETABS that corresponds to the live load.

1 - NOTES: (continued)

The live load reduction factors are then applied to the member forces associated with this load condition, before the member forces 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 **NRDP** defines the number of brace section property sets that are defined in Section 4(iii) below.

10. If **IUNIT** is E all STEELER (and ETABS) input data **must** be prepared in inch-kip-second units and all output will be in the same units.

    If **IUNIT** is M all STEELER (and ETABS) input data **must** be prepared in meter-kilogram-second units. However, the

1 - NOTES: (continued)

STEELER output will be in millimeter or meter-ton-second units e.g., required doubler plate thickness and continuity plate areas will be in millimeter units, whereas the unbraced lengths of the members will be in meter units.

If **IUNIT** is S all STEELER (and ETABS) input data must be prepared in meter-kiloNewton-second units and the STEELER output will be in millimeter or meter-kiloNewton-second units.

The units are printed for each output quantity in all the output files e.g., {k} for kips, {in} for inches, {mm} for millimeters, etc.

The AISC data base is supplied in both inch (file AISC.INC) or meter (file AISC.MET) units.The data base with the correct units should be named AISC.DAT before using ETABS and STEELER.

If **IUNIT** is anything other than, E, M or S it is taken as E.

11. If **IEX** equals 1, the program will only read, print and check the data for consistency and will terminate execution after all the input has been read. All stress check operations will be bypassed. The normal execution mode will produce a complete echo of the input and results. If a data error is detected in a normal execution mode, the program will immediately switch to the data check mode and execution will be terminated after all the input has been read.

12. **PROPFILE** is a user section property file (maximum eight characters) with a .DAT extension which the program searches for if a section property name is not found in the

1 - NOTES: (continued)

AISC.DAT file provided as part of the program. An ETABS and STEELER compatible user section property file can be created by use of the program PROPER, available on request. This allows the user to conveniently use custom built and other non-AISC sections.

It is important that the **PROPFILE** and the AISC.DAT be unchanged between the ETABS and STEELER runs.

13. 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**.

14. 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.

1 - NOTES (continued)

15. 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.

16. 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:

a. The vertical static load conditions I, II and III.

b. The lateral static load conditions A and B.

c. The lateral dynamic load conditions 1, 2 and 3.

Each load combination is generated by factoring the load conditions by the corresponding load combination factors, and then summing up the eight products.

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

L LTYP XI XII XIII XA XB XD1 XD2 XD3

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

2. LOAD COMBINATION DATA

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 Dyn-1 and Dyn-2 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

2. LOAD COMBINATION DATA (continued)

Variable	Field	Note	Entry
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
XD1	8		Multiplier for lateral dynamic load condition 1
XD2	9		Multiplier for lateral dynamic load condition 2
XD3	10		Multiplier for lateral dynamic load condition 3

2 - 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 Dyn-1 and Dyn-2 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.

2 - NOTES: (continued)

Typically, structures are subjected to vertical loads due to dead and live loads which usually 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 UBC91, 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 (EX,EY), the user needs to specify the following load combinations..

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

The 0.75 factor is for the 1/3 rd increase in the allowable stresses, so that all stress ratios are compared against unity while checking for overstresses.

2 - NOTES: (continued)

These are the program defaults whenever the UBC91 code is used, assuming DL is gravity Load Conditions I, LL is gravity Load Condition II, EX is static lateral Load Condition A and EY is static lateral Load Condition B.

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

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.

Prepare the data in the following form:

MID MTYPE E U P FY FBMAJ FBMIN

3. MATERIAL PROPERTY REDEFINITION DATA

Variable	Field	Note	Entry
MID	1	(1)	Identification number of material type that is being defined
MTYPE	2	(2)	Material type: = S Steel = C Concrete (frames) = W Concrete (walls) = M Masonry (walls) = O Other
E	3	(3)	Modulus of elasticity
U	4		Unit weight (weight/volume)
P	5		Poisson's ratio
FY	6	(4)	Yield stress of structural steel
FBMAJ	7	(5)	Allowable bending stresses for structural steel sections in the major direction
FBMIN	8	(5)	Allowable bending stresses for structural steel sections in the minor direction

3 - 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/stress check postprocessors operating off the ETABS postprocessing data base are available. The material type designation is basically an indicator for the postprocessors.

   The steel checking postprocessor STEELER, for example, will only check those members that have a material type S. The concrete frame design postprocessor CONKER will only design the members having a material designation of C. The shear wall design postprocessor WALLER will only process those members that have a material type W or M. Materials having a designation type O will not be processed by any of the postprocessors.

3. This is the modulus of elasticity of the material. Remember consistent units.

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

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     4
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
MAP OF CONTROLLING BEAM INTERACTION STRESS RATIOS

           BEAM       1      2      3      4      5      6      7     8
LEVEL

ROOF                .712   .712   .210   .210   .029   .029   .029  .029

FLOOR               .692   .692                 .046   .061   .046  .061

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

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
MAP OF COLUMN DOUBLER PLATE THICKNESS REQUIREMENTS (in)

        COLUMN      1      2      3      4      5      6      7      8
LEVEL

ROOF              .00    .00    .00    .00    .00    .00    .00    .00

FLOOR             .00    .00    .00    .00    .00    .00    .00    .00

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

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     2
                                ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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

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

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.MAP
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
MAP OF CONTROLLING COLUMN INTERACTION STRESS RATIOS

          COLUMN       1       2       3       4       5       6       7      8
LEVEL

ROOF                 .252    .098    .252    .045    .045    .149    .159   .149

FLOOR                .185    .200    .185    .094    .094    .158    .177   .158

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

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                                   ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
BEAM   SHEAR STRESS CHECK

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

   1       .247 END-J  < 1>      .000 END-J  < 9>

   2       .247 END-I  < 1>      .000 END-J  < 9>

   5       .017 END-J  < 5>      .000 END-J  < 9>

   6       .017 END-I  < 4>      .000 END-J  < 1>

   7       .017 END-J  < 5>      .000 END-J  < 9>

   8       .017 END-I  < 4>      .000 END-J  < 1>
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991  (CHAPTER 27)
BEAM   SHEAR STRESS CHECK

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

   1       .251 END-J  < 1>     .000 END-J  < 9>

   2       .251 END-I  < 1>     .000 END-J  < 9>

   3       .106 END-J  < 1>     .000 END-J  < 9>

   4       .106 END-I  < 1>     .000 END-J  < 9>

   5       .011 END-J  < 5>     .000 END-J  < 9>

   6       .011 END-I  < 4>     .000 END-J  < 9>

   7       .011 END-J  < 5>     .000 END-J  < 9>

   8       .011 END-I  < 4>     .000 END-J  < 9>
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    2
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN SHEAR STRESS CHECK

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

   1       .059 BOTTOM < 3>      .008 BOTTOM < 5>

   2       .035 BOTTOM < 7>      .003 BOTTOM < 4>

   3       .059 BOTTOM < 2>      .008 BOTTOM < 5>

   4       .046 BOTTOM < 5>      .003 BOTTOM < 2>

   5       .046 BOTTOM < 5>      .003 BOTTOM < 3>

   6       .037 BOTTOM < 3>      .008 BOTTOM < 4>

   7       .027 BOTTOM < 7>      .000 BOTTOM < 4>

   8       .037 BOTTOM < 2>      .008 BOTTOM < 4>
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                        ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.SHR
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN SHEAR STRESS CHECK

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

   1       .144 BOTTOM < 1>      .007 BOTTOM < 5>

   2       .028 BOTTOM < 7>      .001 BOTTOM < 5>

   3       .144 BOTTOM < 1>      .007 BOTTOM < 5>

   4       .031 BOTTOM < 4>      .001 BOTTOM < 2>

   5       .031 BOTTOM < 4>      .001 BOTTOM < 3>

   6       .038 BOTTOM < 3>      .007 BOTTOM < 4>

   7       .027 BOTTOM < 7>      .000 BOTTOM < 4>

   8       .038 BOTTOM < 2>      .007 BOTTOM < 4>
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                              ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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)     .000  .692  .000
   2                            1.00  1.00   345.52  120.00
     (T)     .000  .692  .000
   5                            1.00  1.00   165.43  165.43
     (T)     .000  .046  .000
   6                            1.00  1.00   165.43  165.43
     (C)     .001  .027  .014
     (T)     .001  .045  .015
   7                            1.00  1.00   165.43  165.43
     (T)     .000  .046  .000
   8                            1.00  1.00   165.43  165.43
     (C)     .001  .027  .014
     (T)     .001  .045  .015
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                        ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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)     .000  .712  .000
   2                            1.00  1.00   345.52  120.00
     (T)     .000  .712  .000
   3                            1.00  1.00   345.52  120.00
     (T)     .000  .210  .000
   4                            1.00  1.00   345.52  120.00
     (T)     .000  .210  .000
   5                            1.00  1.00   165.43  165.43
     (T)     .000  .029  .000
   6                            1.00  1.00   165.43  165.43
     (T)     .000  .029  .000
   7                            1.00  1.00   165.43  165.43
     (T)     .000  .029  .000
   8                            1.00  1.00   165.43  165.43
     (T)     .000  .029  .000
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     2
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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     .25     2.10
    (C)      .060  .066  .060
   2                           1.68  1.00   126.01  144.00     .51      .00
    (C)      .108  .092  .001
   3                           1.89  1.20   126.01  111.14     .25     2.10
    (C)      .060  .066  .060
   4                           1.21  1.00   111.14  144.00    1.97      .00
    (C)      .010  .084  .000
   5                           1.21  1.00   111.14  144.00    1.97      .00
    (C)      .010  .084  .000
   6                           1.21  1.20   231.14  111.14     .00     2.08
    (C)      .033  .034  .091
   7                           1.18  1.00   231.14  264.00     .00      .00
    (C)      .077  .099  .001
   8                           1.21  1.20   231.14  111.14     .00     2.08
    (C)      .033  .034  .091
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.DTL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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     .49    4.15
     (C)     .035  .208  .009
  2                            1.94  1.00   102.01   120.00     .99     .00
     (C)     .053  .045  .000
  3                            2.55  1.06   102.01    87.14     .49    4.15
     (C)     .035  .208  .009
  4                            1.08  1.00    87.14   120.00    3.93     .00
     (C)     .005  .040  .000
  5                            1.08  1.00    87.14   120.00    3.93     .00
     (C)     .005  .040  .000
  6                            1.21  1.06   231.14    87.14    1.99    4.14
     (C)     .026  .119  .005
  7                            1.18  1.00   231.14   264.00    4.10     .00
     (C)     .075  .083  .000
  8                            1.21  1.06   231.14    87.14    1.99    4.14
     (C)     .026  .119  .005
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   11
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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                                               43.0    43.8 SEISMIC
              (T)    .692 END-J  < 1> (BENDING)      .0
   2  W18X50                                               43.8    43.0 SEISMIC
              (T)    .692 END-I  < 1> (BENDING)      .0
   5  W33X118                                             180.8   182.2 SEISMIC
              (T)    .046 END-J  < 5> (BENDING)      .0
   6  W33X118                                             182.2   180.8 SEISMIC
              (C)    .043 END-I  < 5> (H1-3)         .8
              (T)    .061 END-I  < 4> (H2-1)         .5
   7  W33X118                                             180.8   182.2 SEISMIC
              (T)    .046 END-J  < 5> (BENDING)      .0
   8  W33X118                                             182.2   180.8 SEISMIC
              (C)    .043 END-I  < 5> (H1-3)         .8
              (T)    .061 END-I  < 4> (H2-1)         .5
```

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   10
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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                                               42.6    44.1 SEISMIC
                (T)    .712 END-J  < 1> (BENDING)     .0
   2  W18X50                                               44.1    42.6 SEISMIC
                (T)    .712 END-I  < 1> (BENDING)     .0
   3  W33X118                                             105.5   114.1 SEISMIC
                (T)    .210 END-J  < 1> (BENDING)     .0
   4  W33X118                                             114.1   105.5 SEISMIC
                (T)    .210 END-I  < 1> (BENDING)     .0
   5  W33X118                                             180.8   182.4 SEISMIC
                (T)    .029 END-J  < 5> (BENDING)     .0
   6  W33X118                                             182.4   180.7 SEISMIC
                (T)    .029 END-I  < 4> (BENDING)     .0
   7  W33X118                                             180.8   182.4 SEISMIC
                (T)    .029 END-J  < 5> (BENDING)     .0
   8  W33X118                                             182.4   180.7 SEISMIC
                (T)    .029 END-I  < 4> (BENDING)     .0
```

Sample STEELER Output (continued)

CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS PAGE 9
ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME
LEVEL ID .... FLOOR

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

COL ID	SECTION TYPE	CHECK TYPE	STRESS RATIO	STRESS POINT	<LC>	AISC/UBC EQUATION	MAXIMUM AXIAL (K)	CONT-PL AREA (sqin)	DBLR-PL THICK (in)	SECTION TYPE
1	W14X120	(C)	.185	TOP	< 4>	(H1-3)	47.8	1.3	.00	SEISMIC
2	W14X120	(C)	.200	BOTTOM	< 3>	(H1-3)	97.9	1.3	.00	SEISMIC
3	W14X120	(C)	.185	TOP	< 4>	(H1-3)	47.8	1.3	.00	SEISMIC
4	W14X120	(C)	.094	BOTTOM	< 4>	(H1-3)	9.3	3.3	.00	SEISMIC
5	W14X120	(C)	.094	BOTTOM	< 4>	(H1-3)	9.3	3.3	.00	SEISMIC
6	W14X120	(C)	.158	BOTTOM	< 5>	(H1-3)	22.5	.0	.00	SEISMIC
7	W14X120	(C)	.177	BOTTOM	< 3>	(H1-3)	59.6	.0	.00	SEISMIC
8	W14X120	(C)	.158	BOTTOM	< 5>	(H1-3)	22.5	.0	.00	SEISMIC

Sample STEELER Output (continued)

CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS PAGE 8
ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

COL ID	SECTION TYPE	CHECK TYPE	STRESS RATIO	STRESS POINT	<LC>	AISC/UBC EQUATION	MAXIMUM AXIAL (K)	CONT-PL AREA (sqin)	DBLR-PL THICK (in)	SECTION TYPE
1	W14X120	(C)	.252	TOP	< 1>	(H1-3)	23.5	1.3	.00	SEISMIC
2	W14X120	(C)	.098	TOP	< 3>	(H1-3)	49.2	1.3	.00	SEISMIC
3	W14X120	(C)	.252	TOP	< 1>	(H1-3)	23.5	1.3	.00	SEISMIC
4	W14X120	(C)	.045	BOTTOM	< 4>	(H1-3)	4.7	3.3	.00	SEISMIC
5	W14X120	(C)	.045	BOTTOM	< 4>	(H1-3)	4.7	3.3	.00	SEISMIC
6	W14X120	(C)	.149	TOP	< 3>	(H1-3)	21.0	3.3	.00	SEISMIC
7	W14X120	(C)	.159	TOP	< 3>	(H1-3)	58.2	3.3	.00	SEISMIC
8	W14X120	(C)	.149	TOP	< 2>	(H1-3)	21.0	3.3	.00	SEISMIC

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     7
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME


SPECIFIED BEAM MAJOR DIRECTION Cm-FACTORS

ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS        .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   .000   .000   .000   .000
FLOOR      1.000  1.000                 .000   .000   .000   .000


SPECIFIED BEAM Cb-FACTORS

LEVEL          1      2      3      4      5      6      7      8

ROOF       1.000  1.000  1.000  1.000   .000   .000   .000   .000
FLOOR      1.000  1.000                 .000   .000   .000   .000
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    6
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME


FRAME NUMBER------------------------------      1
FRAMING TYPE------------------------------      2 (SPECIAL)

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 STORY LEVELS---------------------     2
NUMBER OF COLUMN LINES---------------------     8
NUMBER OF BAYS-----------------------------     8
NUMBER OF BRACING ELEMENTS-----------------     0
NUMBER OF PANEL ELEMENTS-------------------     0
NUMBER OF COLUMN LATERAL LOAD PATTERNS-----     0
NUMBER OF BEAM SPAN LOAD PATTERNS----------     2
MAXIMUM NUMBER OF LOADS PER BEAM SPAN------     0


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       .000


SPECIFIED BEAM MINOR DIRECTION K-FACTORS

ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       .000


SPECIFIED BEAM MAJOR DIRECTION LENGTHS (in)

ALL ELEMENTS HAVE THIS OPTION SPECIFIED AS       .000


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
FLOOR      120.0  120.0                   .0     .0     .0     .0
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE     5
                                    ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME
```

SECTION PROPERTIES FOR BEAMS

ID	MAT ID	SECTION TYPE	DEPTH BELOW (in)	DEPTH ABOVE (in)	BEAM WIDTH (in)	FLANGE THICK (in)	WEB THICK (in)
1	1	W18X50	17.990	.000	7.495	.570	.355
2	1	W33X118	32.860	.000	11.480	.740	.550

ANALYSIS SECTION PROPERTIES FOR BEAMS

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	14.700	6.390	7.120	.1240E+01	.8000E+03	.4010E+02
2	34.700	18.070	14.160	.5300E+01	.5900E+04	.1870E+03

AISC STRESS CHECK SECTION PROPERTIES FOR BEAMS

ID	MAJOR S (in_3)	MINOR S (in_3)	MAJOR Z (in_3)	MINOR Z (in_3)	MAJOR R (in)	MINOR R (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 STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    4
                              ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME
```

SECTION PROPERTIES FOR COLUMNS

ID	MAT ID	SECTION TYPE	MAJOR DIM (in)	MINOR DIM (in)	FLANGE THICK (in)	WEB THICK (in)
1	1	W14X120	14.480	14.670	.940	.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	8.540	22.980	.9370E+01	.1380E+04	.4950E+03

AISC 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 STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    3
                                 ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME


MATERIAL PROPERTIES

MAT  MAT    ELASTIC       UNIT  POISSONS      YIELD      MAJOR      MINOR
 ID TYPE    MODULUS     WEIGHT     RATIO         FY         FB         FB
              (Ksi)   (K/cuin)                (Ksi)      (Ksi)      (Ksi)

  1    S  .295E+05   .284E-03      .250   .360E+02  .000E+00  .000E+00
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    2
                              ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME


DESIGN LOADING COMBINATION DATA

LOAD TYPE       I       II      III       A        B    DYN-1    DYN-2  DYN-3
   1    0   1.000    1.000     .000     .000     .000     .000     .000   .000
   2    0    .750     .750     .000     .750     .000     .000     .000   .000
   3    0    .750     .750     .000    -.750     .000     .000     .000   .000
   4    0    .750     .750     .000     .000     .750     .000     .000   .000
   5    0    .750     .750     .000     .000    -.750     .000     .000   .000
   6    0    .750     .000     .000     .750     .000     .000     .000   .000
   7    0    .750     .000     .000    -.750     .000     .000     .000   .000
   8    0    .750     .000     .000     .000     .750     .000     .000   .000
   9    0    .750     .000     .000     .000    -.750     .000     .000   .000
```

Sample STEELER Output (continued)

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                  ETABS_FILE:EXSTL.PST/STEELER_FILE:CHKSTL.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME


CODE CHECK IDENTIFIER---------------------------      1 (UBC 1991 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-----------------  .0000
SHEAR STRESS RATIO CUTOFF-----------------------  .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
```

```
$HEADING DATA
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME
$
$CONTROL DATA
$ ICODE NFR NLC LLC NRMP NRCP NRBP NRDP IUNIT IEX
     1   1   9   2    0    0    0    0    E    0
$OUTPUT CONTROL INFORMATION
$   RI    RS  IDC   IDB   IDD  ISC  ISB   ISD  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    XD1    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    P   FY   FBMAJ    FBMIN
$
$COLUMN SECTION PROPERTY REPLACEMENT DATA
$ID IMAT ITYPE  DMAJ  DMIN  TF  TW
$
$BEAM SECTION PROPERTY REPLACEMENT DATA
$ID IMAT ITYPE  DBMAJ  DAMAJ  DMIN  TF  TW
$
$BRACE SECTION PROPERTY REPLACEMENT DATA
$ID IMAT ITYPE  DMAJ  DMIN  TF  TW
$
$FRAME STRESS CHECK ACTIVATION DATA
$ I  ITYP  IRCP  IRBP  IRDP   RW   IDBL
  1     2     0     1     0    0      0
$
$COLUMN ELEMENT REDEFINITION DATA
$ NT NSAME MC1 MC2 SD1  SD2   P1 P2
$
$BEAM ELEMENT REDEFINITION DATA
$ NT NSAME MB1 MB2 SD1  SD2   P1 P2
  L      0   1   2 ROOF FLOOR  0 30*12/3
  L      0   3   4 ROOF ROOF   0 30*12/3
```

STEELER Input Data File: CHKSTL

```
$BEAM SPAN LOADING ASSIGNMENT DATA
1 0 ROOF 1 2 0 1
2 1
3 1
4 1

$
$FRAME LOCATION DATA
1 0 0 0 0 /MAIN FRAME
$
$STRUCTURAL STATIC LATERAL LOAD DATA
20 0 0 0 0 20
10 0 0 7.5*12 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
```

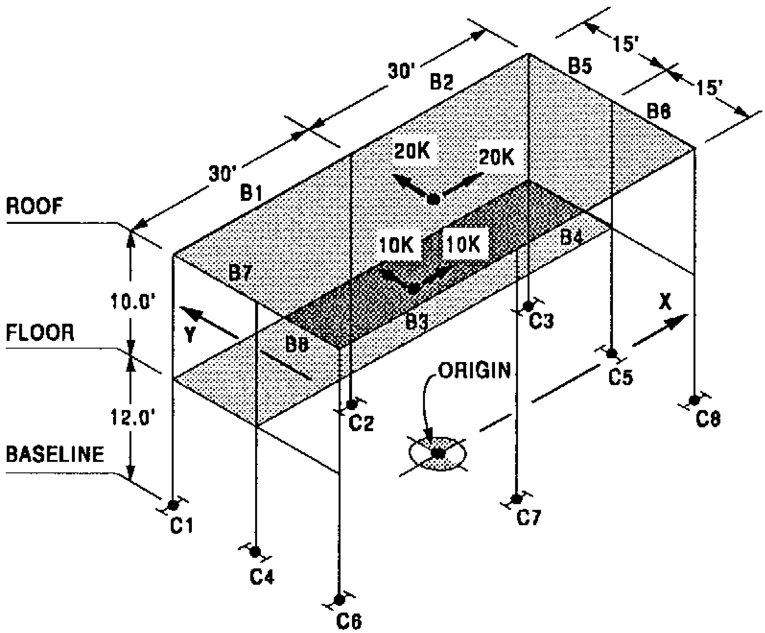
ETABS Input Data File: EXSTL (continued)

```
$HEADING DATA
/SAMPLE EXAMPLE FOR STEELER MANUAL    UNITS:KIP-INCH-SECOND
/DUCTILE MOMENT RESISTING STEEL FRAME
$
$CONTROL DATA
2 1 1 0 4 0 1 1 2 0 0 1 0 0 0 2 0 1 0
386.4
$
$STORY DATA
ROOF 120 0
FLOOR 144 0
$
$MATERIAL PROPERTY DATA
1 S 29500 .490/1728 .25 36
$
$COLUMN SECTION PROPERTY DATA
1 1 W14X120
$
$BEAM SECTION PROPERTY DATA
1 1 W18X50
2 1 W33X118
$
$FRAME HEADING
/MAIN FRAME
$
$FRAME CONTROL DATA
1 2 8 8 0 0 0 2
$
$COLUMN LINE COORDINATES AND ORIENTATION
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
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 0.5/12
$
$COLUMN ASSIGNMENT DATA
1 0 ROOF 1 1
2 1
3 1
4 1
5 1
6 0 ROOF 1 0 0 0
6 0 FLOOR 1 0 0 1
7 6
8 6

$BEAM ASSIGNMENT DATA
1 0 ROOF 1 1
2 1
3 0 ROOF 2
4 3
5 0 ROOF 2 1
6 5
7 5
8 5

$
```

ETABS Input Data File: EXSTL



- TYPICAL COLUMN W14 x 120
- TYPICAL BEAM W33 x 118
  EXCEPT BEAMS 1 & 2 W18 x 50
- LOADING ON BEAMS 1 THRU 4
  1K/ft DL
  .5K/ft LL
- SPECIAL MOMENT RESISTING FRAME
- UBC 91

Sample Example

VII.

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 file EXSTL, along with the ETABS postprocessing file EXSTL.PST, all exist on the STEELER diskette, which comes with the complete STEELER package.

6. Structural Engineers Association of California

"Recommended Lateral Force Requirements and Commentary," Sacramento, California, 1990.

7. Habibullah, A.

"PLOTTER - An Interactive Input and Output Display Postprocessor for ETABS," Computers and Structures, Inc., Berkeley, California, 1991.

8. 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.

VI.

REFERENCES

1. **Habibullah, A.**

   "ETABS - Three-Dimensional Analysis of Building Systems," Computers and Structures, Inc., Berkeley, California, 1991.

2. **International Conference of Building Officials**

   "Uniform Building Code," Whittier, California, 1991.

3. **American Institute of Steel Construction**

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

4. **American Institute of Steel Construction**

   "Manual of Steel Construction, Load & Resistance Factor Design," First Edition, Chicago, Illinois, 1986.

5. **Canadian Standards Association**

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

```
CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                              ETABS_FILE:exst1.PST/STEELER_FILE:sample.MAP
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

FRAME ID .... /MAIN FRAME

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
MAP OF CONTROLLING COLUMN INTERACTION STRESS RATIOS

        COLUMN       1     2     3     4     5     6     7     8
LEVEL

ROOF               .252  .098  .252  .045  .045  .149  .159  .149

FLOOR              .185  .200  .185  .094  .094  .158  .177  .158

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

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

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 Figures V-6.

```
 CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    1
                                         ETABS_FILE:exstl.PST/STEELER_FILE:sample.SHR
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN SHEAR STRESS CHECK

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

    1            .144 BOTTOM < 1>       .007 BOTTOM < 5>

NOTE:   (1)        (2)   (3)    (4)      (2)    (3)    (4)
```

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

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 Figures V-5.

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 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:extl.PST/STEELER_FILE:sample.DTL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
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     .49     4.15
     (C)      .035  .208  .009

NOTE:  (1)  (2)        (3)            (4)           (5)              (6)
```

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

2. The program calculates and displays controlling interaction stress ratios separately from 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 UBC 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-LRFD and CISC codes this same location is used to report moment magnificaion factors.

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 overstresses.

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 UBC 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 AISC-LRFD and CISC 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.

```
 CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   13
                                        ETABS_FILE:ebf.PST/STEELER_FILE:sample.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/ECCENTRIC BRACED FRAME

FRAME ID .... /NORTH STEEL BRACED FRAME
LEVEL ID .... ROOF

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
BRACE  AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

 BRC SECTION CHECK STRESS STRESS        UBC      MAXIMUM CONNECT          SECTION
  ID  TYPE    TYPE  RATIO POINT  <LC> EQUATION    AXIAL  FORCE             TYPE
                                                   (K)    (K)

   1  W14X43                                              293.4           SEISMIC
               (C)    .701 BOTTOM < 1> (27101)    205.8
               (T)    .023 BOTTOM < 6> (H2-1)       6.2

NOTE:   (1)  (2)   (3)    (4) (5)    (6)    (7)        (8)   (13)            (11)
```

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

```
 CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE   10
                                    ETABS_FILE:exstl.PST/STEELER_FILE:sample.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
BEAM   AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

BEAM SECTION CHECK STRESS STRESS       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                                              42.6    44.1 SEISMIC
              (T)    .712 END-J  < 1> (BENDING)     .0

NOTE:  (1)  (2)   (3)    (4)  (5     (6)    (7)     (8)   (12)    (12)  (11)
```

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

```
 CSI/ETABS - EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS  PAGE    8
                                  ETABS_FILE:exst1.PST/STEELER_FILE:sample.STL
/SAMPLE EXAMPLE FOR STEELER MANUAL
/SPECIAL MOMENT RESISTING STEEL FRAME

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

UNIFORM BUILDING CODE 1991 (CHAPTER 27)
COLUMN AXIAL FORCE AND BIAXIAL MOMENT INTERACTION STRESS CHECK

 COL SECTION CHECK STRESS STRESS       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     .00 SEISMIC
               (C)   .252 TOP   < 1> (H1-3)       23.5

NOTE:  (1)  (2)    (3)    (4) (5)    (6)    (7)       (8)    (9)     (10)  (11)
```

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

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 or the appropriate appendix for code references.

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 grovern.

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 and Appendix B, Section E.

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 AISC 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 AISC data base).

11. This is the section classification based on the design code and framing type. e.g., for Special Moment Resisting Space frame and eccentrically braced frame in the UBC Code the

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 AISC data base, will display the section size, e.g., W14X68; other section types are identified as USER, I-SECT etc.

3. The program calculates and displays controlling interaction stress ratios separately from 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.

V.

OUTPUT FILES OF PROGRAM STEELER

Depending upon the options activated by the STEELER input data, the program will produce up to four 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 four output files produced by STEELER will be CHKSTL.STL, CHKSTL.DTL, CHKSTL.SHR and CHKSTL.MAP. However, if the program defaults are acceptable and the STEELER input data file is not provided, the four output files produced by STEELER are EXSTL.STL, EXSTL.DTL, EXSTL.SHR and EXSTL.MAP

Sample output files are presented in Chapter VII.

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 postprocessing file. Link beam information along with actual and allowable link beam rotations are also printed out 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

5(ii) - NOTES: (continued)

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 algorithm defined in Chapter III or the appropriate appendix.

If **NT**=M and the AISC-LRFD and CISC 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-LRFD and U_2 for CISC) (from Load conditions A,B, Dyn-1, Dyn-2 and Dyn-3) in the major and minor directions, respectively. The default values for these is 1.0 as the program assumes a P-Delta analysis has been made. For the AISC-LRFD 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 Appendix C. For the CISC 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 Appendix D.

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) - NOTES: (continued)

If **NT** = R, the data line is for redefining 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 of the column to calculate the live load reduction factor.

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 redefining 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 or the appropriate appendix.

If **NT** = L, the data line is for redefining 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 algorithms defined in Chapter III or the appropriate appendix.

5(ii) - NOTES: (continued)

7. All column elements existing on column lines **MC1** through **MC2** associated with levels **SD1** through **SD2** will be assigned the properties identified by the entries **NT**, **P1** and **P2, P3** and **P4** on this data line.

   As a reminder, column elements associated with a particular level exist below the level.

8. All beam elements existing in bays **MB1** through **MB2** between levels **SD1** through **SD2** (inclusive) will be assigned the properties identified by the entries **NT**, **P1** and **P2, P3** and **P4** on this data line.

9. The parameters **MD1, MD2** and **MDINC** define the following series of brace element identifications:

   MD1, MD1+MDINC, MD1+2MDINC . . .

   which continues until **MD2** is reached.

   All generated identifications that do not correspond to actual brace element identifications are ignored by the program.

10. If **NT** = I, the data line is for redefining 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 values for the section properties are as originally defined in the ETABS data.

5(ii) - NOTES: (continued)

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 in bays **MB1** through **MB2** are set identical to the properties of the beam elements, at the corresponding levels, in bay **NSAME** as it stands defined at the time of this entry.

Redefinitions of beam properties in bay **NSAME** in subsequent data lines will not result in automatic corresponding redefinitions of the member properties in the duplicated bays **MB1** through **MB2**.

In the duplication mode, the entries for **SD1,SD2, P1**, and **P2, P3** and **P4** are meaningless and must not be entered. These entries are only used if **NSAME** is 0.

4. This entry is a column line number. The number must not be greater than **NC**. Also, **MC2** may never be less than **MC1**.

5. This entry is a bay number. The number must not be greater than **NB**. Also, **MB2** may never be less than **MB1**.

6. This entry is an alphanumeric story identifier that must correspond to one of the story level identifiers previously defined in Section 3 of the ETABS data.

   The level associated with **SD2** may never be higher than the level associated with **SD1**.

5(ii) - 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. 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 **MC1** through **MC2** are set identical to the properties of the column elements, at the corresponding levels, of column line **NSAME** as it stands defined at the time of this entry.

   Redefinitions of column properties on column line **NSAME** in subsequent data lines will not result in automatic corresponding redefinitions of the member properties on the duplicated column lines **MC1** through **MC2**.

   In the duplication mode, 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.

3. If **NSAME** is nonzero, it is a bay number, the properties of which are already defined by default or by user specification in preceding data lines of this data section.

5(ii)-c. Brace Element Reassignment Data (Continued)

Variable	Field	Note	Entry
P2	6	(10)	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	(10)	Parameter 3: = C_b (NT= C) = Major moment magnification factor (non-sidesway moments) (NT = M)
P4	8	(10)	Parameter 4: = Minor moment magnification factor (non-sidesway moments) (NT = M)

5(ii)-c. Brace Element Reassignment Data

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 (For AISC-LRFD and CISC only)
MD1	2	(9)	Brace ID number of first brace being reassigned
MD2	3	(9)	Brace ID number of last brace being reassigned
MDINC	4	(9)	Brace ID increment
P1	5	(10)	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)

5 (ii)-c. Brace Element Reassignment Data

If **IRDP** 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 **NTRU** is 0, 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.** Prepare the data in the following form:

NT MD1 MD2 MDINC P1 P2 P3 P4

5(ii)-b. Beam Element Reassignment Data (Continued)

Variable	Field	Note	Entry
P1	7	(10)	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)
P2	8	(10)	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	(10)	Parameter 3: = C_b (NT = C) = Major moment magnification factor (non-sidesway moments) (NT = M)
P4	10	(10)	Parameter 4: = Minor moment magnification factor (non-sidesway moments) (NT = M)

5(ii)-b. Beam Element Reassignment Data

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 (For AISC-LRFD and CISC only) = E Link Beam length and status
NSAME	2	(3)	Bay number, the properties of which are to be repeated at bays **MB1** through **MB2**
MB1	3	(5,8)	Bay number of first bay being reassigned
MB2	4	(5,8)	Bay number of last bay being reassigned
SD1	5	(6,8)	Identification of the story level associated with topmost beam being reassigned
SD2	6	(6,8)	Identification of the story level associated with bottommost beam being reassigned

5 (ii)-b. Beam Element Reassignment Data

If **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.** Prepare the data in the following form:

NT NSAME MB1 MB2 SD1 SD2 P1 P2 P3 P4

5(ii)-a. Column Element Reassignment Data (Continued)

Variable	Field	Note	Entry
P1	7	(10)	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)
P2	8	(10)	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	(10)	Parameter 3: = C_b (NT = C) = Major moment magnification factor (non-sidesway moments) (NT = M)
P4	10	(10)	Parameter 4: = Minor moment magnification factor (non-sidesway moments) (NT = M)

5(ii)-a. Column Element Reassignment Data

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 (For AISC-LRFD and CISC only)
NSAME	2	(2)	Column line number, the properties of which are to be repeated at column lines **MC1** through **MC2**
MC1	3	(4,7)	Column line number of first column line being reassigned
MC2	4	(4,7)	Column line number of last column line being reassigned
SD1	5	(6,7)	Identification of the story level associated with topmost column being reassigned
SD2	6	(6,7)	Identification of the story level associated with bottommost column being reassigned

5 (ii) - a. Column Element 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.** Prepare the data in the following form:

NT NSAME MC1 MC2 SD1 SD2 P1 P2 P3 P4

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(i) - NOTES:

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 **IRDP** 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 UBC91 code.

5. This entry selects the criterion used in computing doubler plates under the UBC91 code. A value of 0 forces the program to select doubler plates based upon a force which is lesser of gravity loads plus 1.85 times the prescribed seismic forces or $0.8\Sigma M_S$ of the girders framing into the Column flanges at the joint. (UBC 2710(g)2). A value of 1 only uses $0.8\Sigma M_S$ of the girders to compute doubler plates. The default value is 0.

5(i) - 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 data. See Chapter V, Section D-7 of the ETABS manual (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 UBC 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 Appendix A. The default value of **ITYP** for the UBC91 code is 2 (special moment resisting frame).

   For all other codes (AISC-ASD89, AISC-Plastic Design, AISC-LRFD86 and the CISC89) **ITYP** of 1 and 3 are the only valid frame types. **ITYP** of 2 and 4 will result in an error message and termination of the program.

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.

5(i). FRAME STRESS CHECK ACTIVATION DATA

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 resisting space frame = 2 Special moment resisting space frame = 3 Braced frame = 4 Eccentrically braced frame
IRCP	3	(3)	Column reassignment flag: = 0 No column reassignment provided = 1 Column reassignment provided
IRBP	4	(3)	Beam reassignment flag: = 0 No beam reassignment provided = 1 Beam reassignment provided
IRDP	5	(3)	Brace reassignment flag: = 0 No brace reassignment provided = 1 Brace reassignment provided
RW	6	(4)	Seismic design coefficient (UBC91 code only)
IDBL	7	(5)	Doubler Plate flag = 0 see notes = 1 see notes

5. FRAME STRESS CHECK ACTIVATION DATA SETS

Provide **NFR** data sets, one for each of the ETABS frames that are to be stress checked.

(i). Frame Stress Check Control Data

Prepare one line of data in the following form:

I ITYP IRCP IRBP IRDP RW IDBL

4 - NOTES: (continued)

direction. Similarly for other sections shown in Figure IV-2.

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-4.

7. 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. Beam depths recovered from the AISC data base are assigned to **DBMAJ** (**DAMAJ** is set to 0). See Figure IV-5.

   In calculating the section properties of shapes in Figure IV-2 for beams, **DMAJ** = **DBMAJ** + **DAMAJ**.

8. 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-6.

9. These dimensions are only needed if **ITYPE** is **BOX**, **I-SECT**, **C-SECT**, **T-SECT** or **L-SECT**.

4 - NOTES: (continued)

4. 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.

5. 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). If the material associated with this section property set is of type S, i.e. steel, all 12 properties must be nonzero.

   If **ITYPE** is **RECT**, **PIPE**, **CIRCLE**, **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**.

   The program has a built-in data base of steel shapes conforming to the standards of the American Institute of Steel Construction (AISC). The available section properties are listed in Figure IV-3.

   If **ITYPE** is an AISC identification label (with no embedded blanks), 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.

   When the section properties are recovered from the AISC data base of wide flange section options, it should be noted that the strong moment of inertia is assigned to correspond to the bending about the major axis, and the area of the web is assigned to correspond to the shear forces along the major

4 - 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. 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.

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

   If the identification number is less than or equal to **NDP**, 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 **NDP**, the brace property table is expanded and a new brace property set corresponding to this identification number is created.

4(iii). Brace Property Redefinition Data (Continued)

Second Data Line (continued)

Variable	Field	Note	Entry
S1	7		Section modulus, about major axis
S2	8		Section modulus, about minor axis
Z1	9		Plastic modulus, about major axis
Z2	10		Plastic modulus, about minor axis
R1	11		Radius of gyration, about major axis
R2	12		Radius of gyration, about minor axis

4(iii). Brace Property Redefinition Data (Continued)

First Data Line (continued)

Variable	Field	Note	Entry
TF	6	(9)	Flange thickness
TW	7	(9)	Web thickness

Second Data Line

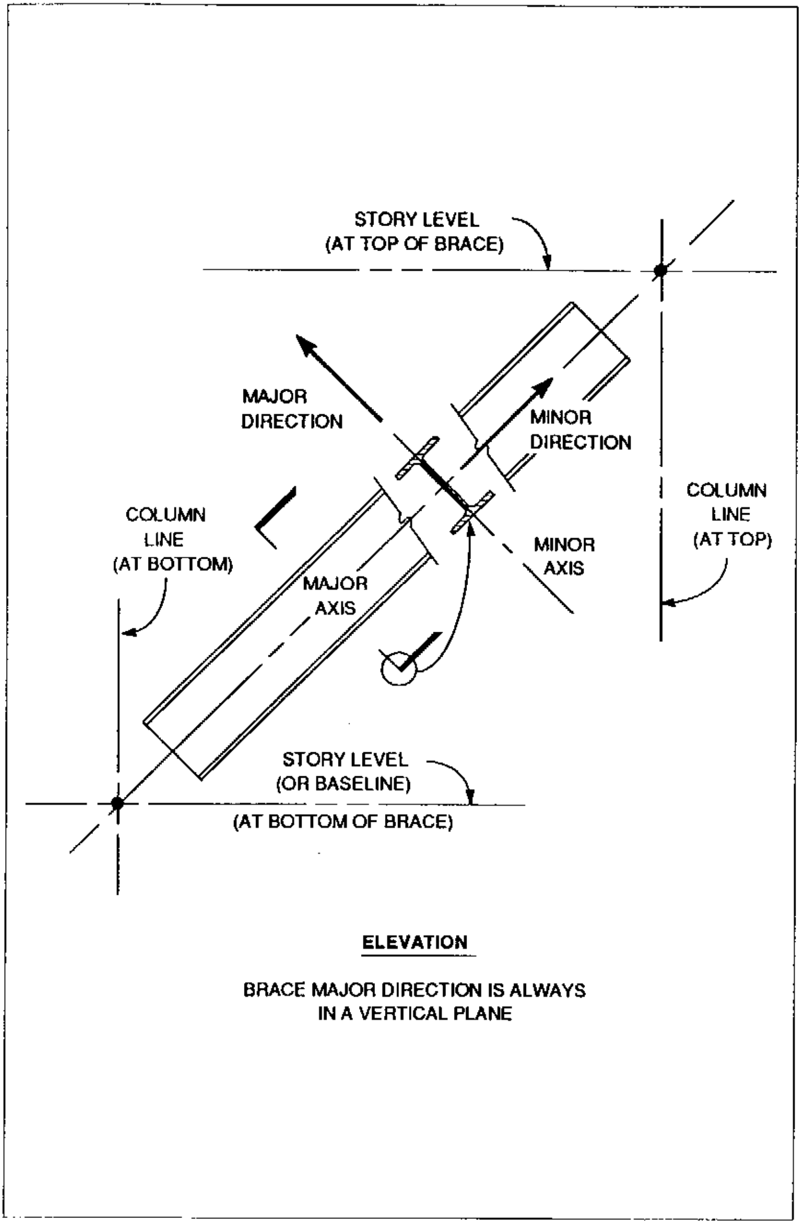
Variable	Field	Note	Entry
A	1		Cross-sectional axial area
A1	2		Shear area corresponding to major direction shear forces
A2	3		Shear area corresponding to minor direction shear forces
J	4		Torsional constant
I1	5		Moment of inertia, about major axis
I2	6		Moment of inertia, about minor axis

4(iii). Brace Property Redefinition Data

First Data Line

Variable	Field	Note	Entry
ID	1	(3)	Identification number of brace section property that is being replaced or added
IMAT	2	(4)	Material identification type for this section property
ITYPE	3	(5)	Section type: = USER = RECT = PIPE = CIRCLE = BOX = I-SECT = C-SECT = T-SECT = L-SECT = W14X233*
DMAJ	4	(8)	Section dimension in major direction
DMIN	5	(8)	Section dimension in minor direction

*or any other of the AISC designations listed in Figure IV-3 or present in the user defined **PROPFILE.**



Brace Section Orientation
Figure IV-6

4(iii). Brace Property Redefinition Data

If **NRDP** is 0, skip this data section. Otherwise, provide **NRDP** 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.

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 IMAT ITYPE 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 A1 A2 J I1 I2 S1 S2 Z1 Z2 R1 R2

4(ii). Beam Property Redefinition Data (Continued)

Second Data Line (continued)

Variable	Field	Note	Entry
R1	11		Radius of gyration, about major axis
R2	12		Radius of gyration, about minor axis

4(ii). Beam Property Redefinition Data (Continued)

Second Data Line

Variable	Field	Note	Entry
A	1		Cross-sectional axial area
A1	2		Shear area corresponding to major direction shear forces
A2	3		Shear area corresponding to minor direction shear forces
J	4		Torsional constant
I1	5		Moment of inertia, about major axis
I2	6		Moment of inertia, about minor axis
S1	7		Section modulus, about major axis
S2	8		Section modulus, about minor axis
Z1	9		Plastic modulus, about major axis
Z2	10		Plastic modulus, about minor axis

4(ii). Beam Property Redefinition Data (Continued)

First Data Line (continued)

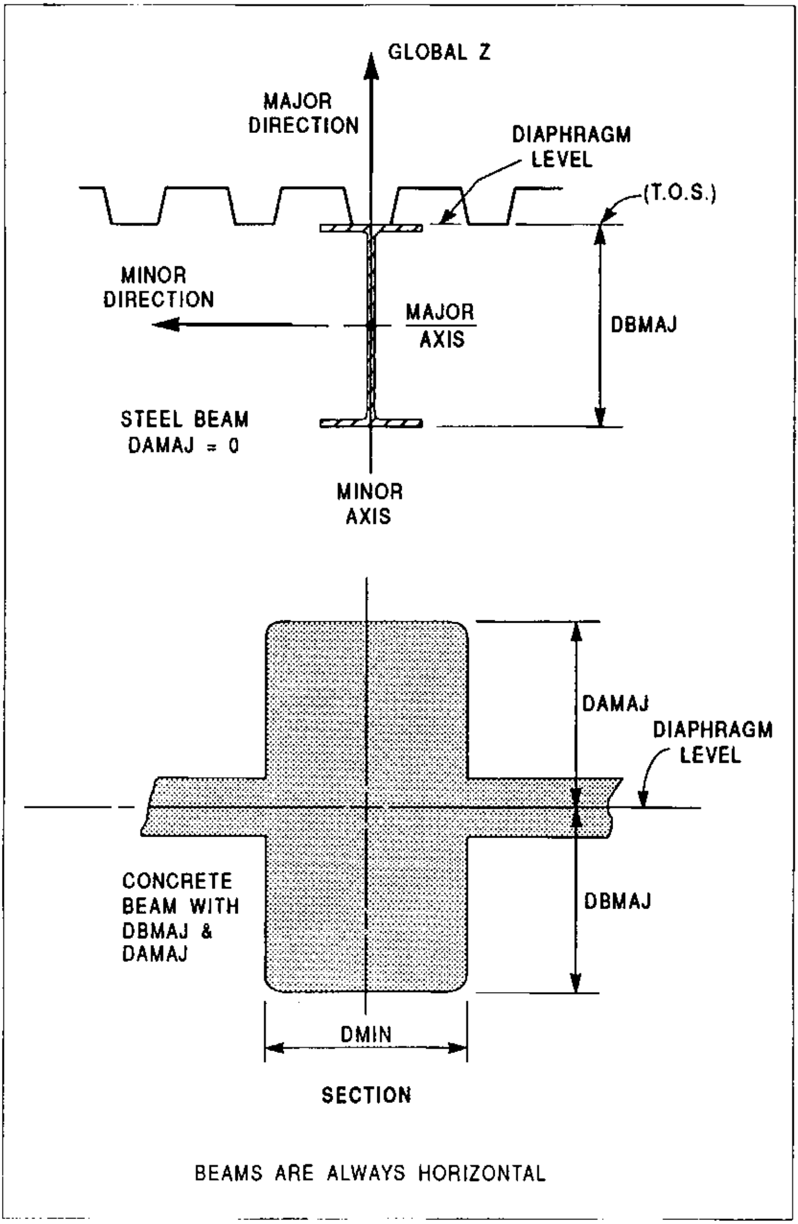
Variable	Field	Note	Entry
DMIN	6	(7)	Section dimension in minor direction, width
TF	7	(9)	Flange thickness
TW	8	(9)	Web thickness

4(ii). Beam Property Redefinition Data

First Data Line

Variable	**Field**	**Note**	**Entry**
ID	1	(2)	Identification number of beam section property that is being replaced or added
IMAT	2	(4)	Material identification type for this section property
ITYPE	3	(5)	Section type: = USER = RECT = PIPE = CIRCLE = BOX = I-SECT = C-SECT = T-SECT = L-SECT = W14X233*
DBMAJ	4	(7)	Section dimension in major direction, depth below diaphragm
DAMAJ	5	(7)	Section dimension in major direction, depth above diaphragm

*or any other of the AISC designations listed in Figure IV-3 or present in the user defined **PROPFILE.**



Beam Section Orientation
Figure IV-5

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.

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 IMAT ITYPE 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 the line is needed, it should be prepared in the following form:

A A1 A2 J I1 I2 S1 S2 Z1 Z2 R1 R2

4(i). Column Property Redefinition Data (continued)

Second Data Line (continued)

Variable	**Field**	**Note**	**Entry**
S1	7		Section modulus, about major axis
S2	8		Section modulus, about minor axis
Z1	9		Plastic modulus, about major axis
Z2	10		Plastic modulus, about minor axis
R1	11		Radius of gyration, about major axis
R2	12		Radius of gyration, about minor axis

4(i). Column Property Redefinition Data (continued)

First Data Line (continued)

Variable	Field	Note	Entry
TF	6	(9)	Flange thickness
TW	7	(9)	Web thickness

Second Data Line

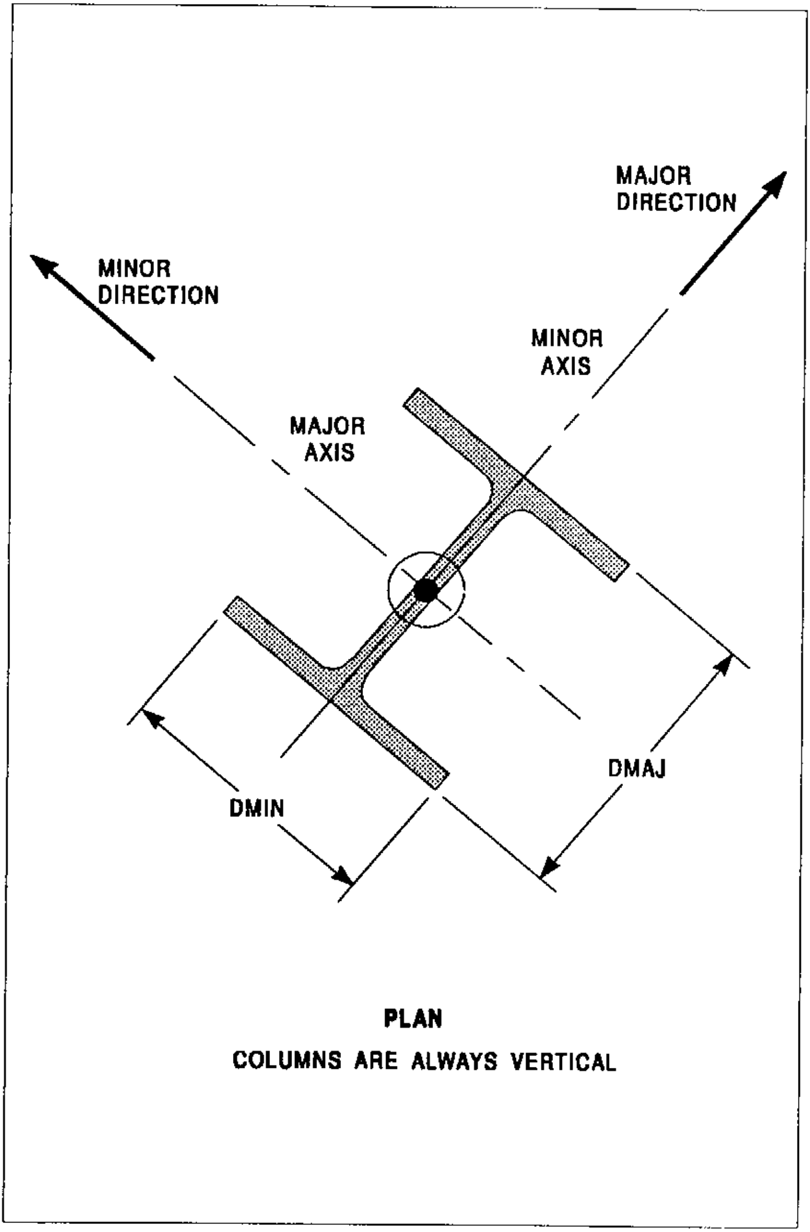
Variable	Field	Note	Entry
A	1		Cross-sectional axial area
A1	2		Shear area corresponding to major direction shear forces
A2	3		Shear area corresponding to minor direction shear forces
J	4		Torsional constant
I1	5		Moment of inertia, about major axis
I2	6		Moment of inertia, about minor axis

4(i). Column Property Redefinition Data

First Data Line

Variable	**Field**	**Note**	**Entry**
ID	1	(1)	Identification number of column section property that is being replaced or added
IMAT	2	(4)	Material identification type for this section property
ITYPE	3	(5)	Section type: = USER = RECT = PIPE = CIRCLE = BOX = I-SECT = C-SECT = T-SECT = L-SECT = W14X233*
DMAJ	4	(6)	Section dimension in major direction
DMIN	5	(6)	Section dimension in minor direction

*or any other of the AISC designations listed in Figure IV-3 or present in the user defined **PROPFILE.**



Column Section Orientation
Figure IV-4

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.

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 IMAT ITYPE 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 A1 A2 J I1 I2 S1 S2 Z1 Z2 R1 R2

TS6X4X1/4	TS6X4X3/16	TS6X3X3/8	TS6X3X5/16
TS6X3X1/4	TS6X3X3/16	TS6X2X3/8	TS6X2X5/16
TS6X2X1/4	TS6X2X3/16	TS5X4X3/8	TS5X4X5/16
TS5X4X1/4	TS5X4X3/16	TS5X3X1/2	TS5X3X3/8
TS5X3X5/16	TS5X3X1/4	TS5X3X3/16	TS5X2X5/16
TS5X2X1/4	TS5X2X3/16	TS4X3X5/16	TS4X3X1/4
TS4X3X3/16	TS4X2X5/16	TS4X2X1/4	TS4X2X3/16
TS3X2X1/4	TS3X2X3/16	PS.5	PS.75
PS1	PS1.25	PS1.5	PS2
PS2.5	PS3	PS3.5	PS4
PS5	PS6	PS8	PS10
PS12	PE.5	PE.75	PE1
PE1.25	PE1.5	PE2	PE2.5
PE3	PE3.5	PE4	PE5
PE6	PE8	PE10	PE12
PD2	PD2.5	PD3	PD4
PD5	PD6	PD8	

Built-in AISC Property Designations (continued)
Figure IV-3

2L3.5X4X1/2-3	2L3.5X4X1/2-6	2L3.5X4X3/8	2L3.5X4X3/8-3
2L3.5X4X3/8-6	2L3.5X4X5/16	2L3.5X4X5/16-3	2L3.5X4X5/16-6
2L3.5X4X1/4	2L3.5X4X1/4-3	2L3.5X4X1/4-6	2L3X4X1/2
2L3X4X1/2-3	2L3X4X1/2-6	2L3X4X3/8	2L3X4X3/8-3
2L3X4X3/8-6	2L3X4X5/16	2L3X4X5/16-3	2L3X4X5/16-6
2L3X4X1/4	2L3X4X1/4-3	2L3X4X1/4-6	2L3X3.5X3/8
2L3X3.5X3/8-3	2L3X3.5X3/8-6	2L3X3.5X5/16	2L3X3.5X5/16-3
2L3X3.5X5/16-6	2L3X3.5X1/4	2L3X3.5X1/4-3	2L3X3.5X1/4-6
2L2.5X3.5X3/8	2L2.5X3.5X3/8-3	2L2.5X3.5X3/8-6	2L2.5X3.5X5/16
2L2.5X3.5X5/16-3	2L2.5X3.5X5/16-6	2L2.5X3.5X1/4	2L2.5X3.5X1/4-3
2L2.5X3.5X1/4-6	2L2.5X3X3/8	2L2.5X3X3/8-3	2L2.5X3X3/8-6
2L2.5X3X1/4	2L2.5X3X1/4-3	2L2.5X3X1/4-6	2L2.5X3X3/16
2L2.5X3X3/16-3	2L2.5X3X3/16-6	2L2X3X3/8	2L2X3X3/8-3
2L2X3X3/8-6	2L2X3X5/16	2L2X3X5/16-3	2L2X3X5/16-6
2L2X3X1/4	2L2X3X1/4-3	2L2X3X1/4-6	2L2X3X3/16
2L2X3X3/16-3	2L2X3X3/16-6	2L2X2.5X3/8	2L2X2.5X3/8-3
2L2X2.5X3/8-6	2L2X2.5X5/16	2L2X2.5X5/16-3	2L2X2.5X5/16-6
2L2X2.5X1/4	2L2X2.5X1/4-3	2L2X2.5X1/4-6	2L2X2.5X3/16
2L2X2.5X3/16-3	2L2X2.5X3/16-6	TS16X16X1/2	TS16X16X3/8
TS16X16X5/16	TS14X14X1/2	TS14X14X3/8	TS14X14X5/16
TS12X12X1/2	TS12X12X3/8	TS12X12X5/16	TS12X12X1/4
TS10X10X5/8	TS10X10X1/2	TS10X10X3/8	TS10X10X5/16
TS10X10X1/4	TS8X8X5/8	TS8X8X1/2	TS8X8X3/8
TS8X8X5/16	TS8X8X1/4	TS8X8X3/16	TS7X7X1/2
TS7X7X3/8	TS7X7X5/16	TS7X7X1/4	TS7X7X3/16
TS6X6X1/2	TS6X6X3/8	TS6X6X5/16	TS6X6X1/4
TS6X6X3/16	TS5X5X1/2	TS5X5X3/8	TS5X5X5/16
TS5X5X1/4	TS5X5X3/16	TS4X4X1/2	TS4X4X3/8
TS4X4X5/16	TS4X4X1/4	TS4X4X3/16	TS3.5X3.5X5/16
TS3.5X3.5X1/4	TS3.5X3.5X3/16	TS3X3X5/16	TS3X3X1/4
TS3X3X3/16	TS2.5X2.5X1/4	TS2.5X2.5X3/16	TS2X2X1/4
TS2X2X3/16	TS20X12X1/2	TS20X12X3/8	TS20X12X5/16
TS20X8X1/2	TS20X8X3/8	TS20X8X5/16	TS20X4X1/2
TS20X4X3/8	TS20X4X5/16	TS18X6X1/2	TS18X6X3/8
TS18X6X5/16	TS16X12X1/2	TS16X12X3/8	TS16X12X5/16
TS16X8X1/2	TS16X8X3/8	TS16X8X5/16	TS16X4X1/2
TS16X4X3/8	TS16X4X5/16	TS14X10X1/2	TS14X10X3/8
TS14X10X5/16	TS14X6X1/2	TS14X6X3/8	TS14X6X5/16
TS14X6X1/4	TS14X4X1/2	TS14X4X3/8	TS14X4X5/16
TS14X4X1/4	TS12X8X5/8	TS12X8X1/2	TS12X8X3/8
TS12X8X5/16	TS12X8X1/4	TS12X6X5/8	TS12X6X1/2
TS12X6X3/8	TS12X6X5/16	TS12X6X1/4	TS12X6X3/16
TS12X4X1/2	TS12X4X3/8	TS12X4X5/16	TS12X4X1/4
TS12X4X3/16	TS12X2X1/4	TS12X2X3/16	TS10X6X5/8
TS10X6X1/2	TS10X6X3/8	TS10X6X5/16	TS10X6X1/4
TS10X6X3/16	TS10X4X1/2	TS10X4X3/8	TS10X4X5/16
TS10X4X1/4	TS10X4X3/16	TS10X2X3/8	TS10X2X5/16
TS10X2X1/4	TS10X2X3/16	TS8X6X1/2	TS8X6X3/8
TS8X6X5/16	TS8X6X1/4	TS8X6X3/16	TS8X4X1/2
TS8X4X3/8	TS8X4X5/16	TS8X4X1/4	TS8X4X3/16
TS8X3X3/8	TS8X3X5/16	TS8X3X1/4	TS8X3X3/16
TS8X2X3/8	TS8X2X5/16	TS8X2X1/4	TS8X2X3/16
TS7X5X1/2	TS7X5X3/8	TS7X5X5/16	TS7X5X1/4
TS7X5X3/16	TS7X4X3/8	TS7X4X5/16	TS7X4X1/4
TS7X4X3/16	TS7X3X3/8	TS7X3X5/16	TS7X3X1/4
TS7X3X3/16	TS6X4X1/2	TS6X4X3/8	TS6X4X5/16

Built-in AISC Property Designations (continued)
Figure IV-3

2L2X2X5/16-6	2L2X2X1/4	2L2X2X1/4-3	2L2X2X1/4-6
2L2X2X3/16	2L2X2X3/16-3	2L2X2X3/16-6	2L2X2X1/8
2L2X2X1/8-3	2L2X2X1/8-6	2L8X6X1	2L8X6X1-3
2L8X6X1-6	2L8X6X3/4	2L8X6X3/4-3	2L8X6X3/4-6
2L8X6X1/2	2L8X6X1/2-3	2L8X6X1/2-6	2L8X4X1
2L8X4X1-3	2L8X4X1-6	2L8X4X3/4	2L8X4X3/4-3
2L8X4X3/4-6	2L8X4X1/2	2L8X4X1/2-3	2L8X4X1/2-6
2L7X4X3/4	2L7X4X3/4-3	2L7X4X3/4-6	2L7X4X1/2
2L7X4X1/2-3	2L7X4X1/2-6	2L7X4X3/8	2L7X4X3/8-3
2L7X4X3/8-6	2L6X4X3/4	2L6X4X3/4-3	2L6X4X3/4-6
2L6X4X5/8	2L6X4X5/8-3	2L6X4X5/8-6	2L6X4X1/2
2L6X4X1/2-3	2L6X4X1/2-6	2L6X4X3/8	2L6X4X3/8-3
2L6X4X3/8-6	2L6X3.5X3/8	2L6X3.5X3/8-3	2L6X3.5X3/8-6
2L6X3.5X5/16	2L6X3.5X5/16-3	2L6X3.5X5/16-6	2L5X3.5X3/4
2L5X3.5X3/4-3	2L5X3.5X3/4-6	2L5X3.5X1/2	2L5X3.5X1/2-3
2L5X3.5X1/2-6	2L5X3.5X3/8	2L5X3.5X3/8-3	2L5X3.5X3/8-6
2L5X3.5X5/16	2L5X3.5X5/16-3	2L5X3.5X5/16-6	2L5X3X1/2
2L5X3X1/2-3	2L5X3X1/2-6	2L5X3X3/8	2L5X3X3/8-3
2L5X3X3/8-6	2L5X3X5/16	2L5X3X5/16-3	2L5X3X5/16-6
2L5X3X1/4	2L5X3X1/4-3	2L5X3X1/4-6	2L4X3.5X1/2
2L4X3.5X1/2-3	2L4X3.5X1/2-6	2L4X3.5X3/8	2L4X3.5X3/8-3
2L4X3.5X3/8-6	2L4X3.5X5/16	2L4X3.5X5/16-3	2L4X3.5X5/16-6
2L4X3.5X1/4	2L4X3.5X1/4-3	2L4X3.5X1/4-6	2L4X3X1/2
2L4X3X1/2-3	2L4X3X1/2-6	2L4X3X3/8	2L4X3X3/8-3
2L4X3X3/8-6	2L4X3X5/16	2L4X3X5/16-3	2L4X3X5/16-6
2L4X3X1/4	2L4X3X1/4-3	2L4X3X1/4-6	2L3.5X3X3/8
2L3.5X3X3/8-3	2L3.5X3X3/8-6	2L3.5X3X5/16	2L3.5X3X5/16-3
2L3.5X3X5/16-6	2L3.5X3X1/4	2L3.5X3X1/4-3	2L3.5X3X1/4-6
2L3.5X2.5X3/8	2L3.5X2.5X3/8-3	2L3.5X2.5X3/8-6	2L3.5X2.5X5/16
2L3.5X2.5X5/16-3	2L3.5X2.5X5/16-6	2L3.5X2.5X1/4	2L3.5X2.5X1/4-3
2L3.5X2.5X1/4-6	2L3X2.5X3/8	2L3X2.5X3/8-3	2L3X2.5X3/8-6
2L3X2.5X1/4	2L3X2.5X1/4-3	2L3X2.5X1/4-6	2L3X2.5X3/16
2L3X2.5X3/16-3	2L3X2.5X3/16-6	2L3X2X3/8	2L3X2X3/8-3
2L3X2X3/8-6	2L3X2X5/16	2L3X2X5/16-3	2L3X2X5/16-6
2L3X2X1/4	2L3X2X1/4-3	2L3X2X1/4-6	2L3X2X3/16
2L3X2X3/16-3	2L3X2X3/16-6	2L2.5X2X3/8	2L2.5X2X3/8-3
2L2.5X2X3/8-6	2L2.5X2X5/16	2L2.5X2X5/16-3	2L2.5X2X5/16-6
2L2.5X2X1/4	2L2.5X2X1/4-3	2L2.5X2X1/4-6	2L2.5X2X3/16
2L2.5X2X3/16-3	2L2.5X2X3/16-6	2L6X8X1	2L6X8X1-3
2L6X8X1-6	2L6X8X3/4	2L6X8X3/4-3	2L6X8X3/4-6
2L6X8X1/2	2L6X8X1/2-3	2L6X8X1/2-6	2L4X8X1
2L4X8X1-3	2L4X8X1-6	2L4X8X3/4	2L4X8X3/4-3
2L4X8X3/4-6	2L4X8X1/2	2L4X8X1/2-3	2L4X8X1/2-6
2L4X7X3/4	2L4X7X3/4-3	2L4X7X3/4-6	2L4X7X1/2
2L4X7X1/2-3	2L4X7X1/2-6	2L4X7X3/8	2L4X7X3/8-3
2L4X7X3/8-6	2L4X6X3/4	2L4X6X3/4-3	2L4X6X3/4-6
2L4X6X5/8	2L4X6X5/8-3	2L4X6X5/8-6	2L4X6X1/2
2L4X6X1/2-3	2L4X6X1/2-6	2L4X6X3/8	2L4X6X3/8-3
2L4X6X3/8-6	2L3.5X6X3/8	2L3.5X6X3/8-3	2L3.5X6X3/8-6
2L3.5X6X5/16	2L3.5X6X5/16-3	2L3.5X6X5/16-6	2L3.5X5X3/4
2L3.5X5X3/4-3	2L3.5X5X3/4-6	2L3.5X5X1/2	2L3.5X5X1/2-3
2L3.5X5X1/2-6	2L3.5X5X3/8	2L3.5X5X3/8-3	2L3.5X5X3/8-6
2L3.5X5X5/16	2L3.5X5X5/16-3	2L3.5X5X5/16-6	2L3X5X1/2
2L3X5X1/2-3	2L3X5X1/2-6	2L3X5X3/8	2L3X5X3/8-3
2L3X5X3/8-6	2L3X5X5/16	2L3X5X5/16-3	2L3X5X5/16-6
2L3X5X1/4	2L3X5X1/4-3	2L3X5X1/4-6	2L3.5X4X1/2

Built-in AISC Property Designations (continued)
Figure IV-3

L8X4X1	L8X4X3/4	L8X4X9/16	L8X4X1/2
L7X4X3	L7X4X5/8	L7X4X1/2	L7X4X3/8
L6X6X1	L6X6X7/8	L6X6X3/4	L6X6X5/8
L6X6X9/16	L6X6X1/2	L6X6X7/16	L6X6X3/8
L6X6X5/16	L6X4X7/8	L6X4X3/4	L6X4X5/8
L6X4X9/16	L6X4X1/2	L6X4X7/16	L6X4X3/8
L6X4X5/16	L6X3.5X1/2	L6X3.5X3/8	L6X3.5X5/16
L5X5X7/8	L5X5X3/4	L5X5X5/8	L5X5X1/2
L5X5X7/16	L5X5X3/8	L5X5X5/16	L5X3.5X3/4
L5X3.5X5/8	L5X3.5X1/2	L5X3.5X7/16	L5X3.5X3/8
L5X3.5X5/16	L5X3.5X1/4	L5X3X5/8	L5X3X1/2
L5X3X7/16	L5X3X3/8	L5X3X5/16	L5X3X1/4
L4X4X3/4	L4X4X5/8	L4X4X1/2	L4X4X7/16
L4X4X3/8	L4X4X5/16	L4X4X1/4	L4X3.5X5/8
L4X3.5X1/2	L4X3.5X7/16	L4X3.5X3/8	L4X3.5X5/16
L4X3.5X1/4	L4X3X5/8	L4X3X1/2	L4X3X7/16
L4X3X3/8	L4X3X5/16	L4X3X1/4	L3.5X3.5X1/2
L3.5X3.5X7/16	L3.5X3.5X3/8	L3.5X3.5X5/16	L3.5X3.5X1/4
L3.5X3X1/2	L3.5X3X7/16	L3.5X3X3/8	L3.5X3X5/16
L3.5X3X1/4	L3.5X2.5X1/2	L3.5X2.5X7/16	L3.5X2.5X3/8
L3.5X2.5X5/16	L3.5X2.5X1/4	L3X3X1/2	L3X3X7/16
L3X3X3/8	L3X3X5/16	L3X3X1/4	L3X3X3/16
L3X2.5X1/2	L3X2.5X7/16	L3X2.5X3/8	L3X2.5X5/16
L3X2.5X1/4	L3X2.5X3/16	L3X2X1/2	L3X2X7/16
L3X2X3/8	L3X2X5/16	L3X2X1/4	L3X2X3/16
L2.5X2.5X1/2	L2.5X2.5X3/8	L2.5X2.5X5/16	L2.5X2.5X1/4
L2.5X2.5X3/16	L2.5X2X3/8	L2.5X2X5/16	L2.5X2X1/4
L2.5X2X3/16	L2X2X3/8	L2X2X5/16	L2X2X1/4
L2X2X3/16	L2X2X1/8	2L8X8X9/8	2L8X8X9/8-3
2L8X8X9/8-6	2L8X8X1	2L8X8X1-3	2L8X8X1-6
2L8X8X7/8	2L8X8X7/8-3	2L8X8X7/8-6	2L8X8X3/4
2L8X8X3/4-3	2L8X8X3/4-6	2L8X8X5/8	2L8X8X5/8-3
2L8X8X5/8-6	2L8X8X1/2	2L8X8X1/2-3	2L8X8X1/2-6
2L6X6X1	2L6X6X1-3	2L6X6X1-6	2L6X6X7/8
2L6X6X7/8-3	2L6X6X7/8-6	2L6X6X3/4	2L6X6X3/4-3
2L6X6X3/4-6	2L6X6X5/8	2L6X6X5/8-3	2L6X6X5/8-6
2L6X6X1/2	2L6X6X1/2-3	2L6X6X1/2-6	2L6X6X3/8
2L6X6X3/8-3	2L6X6X3/8-6	2L5X5X7/8	2L5X5X7/8-3
2L5X5X7/8-6	2L5X5X3/4	2L5X5X3/4-3	2L5X5X3/4-6
2L5X5X1/2	2L5X5X1/2-3	2L5X5X1/2-6	2L5X5X3/8
2L5X5X3/8-3	2L5X5X3/8-6	2L5X5X5/16	2L5X5X5/16-3
2L5X5X5/16-6	2L4X4X3/4	2L4X4X3/4-3	2L4X4X3/4-6
2L4X4X5/8	2L4X4X5/8-3	2L4X4X5/8-6	2L4X4X1/2
2L4X4X1/2-3	2L4X4X1/2-6	2L4X4X3/8	2L4X4X3/8-3
2L4X4X3/8-6	2L4X4X5/16	2L4X4X5/16-3	2L4X4X5/16-6
2L4X4X1/4	2L4X4X1/4-3	2L4X4X1/4-6	2L3.5X3.5X3/8
2L3.5X3.5X3/8-3	2L3.5X3.5X3/8-6	2L3.5X3.5X5/16	2L3.5X3.5X5/16-3
2L3.5X3.5X5/16-6	2L3.5X3.5X1/4	2L3.5X3.5X1/4-3	2L3.5X3.5X1/4-6
2L3X3X1/2	2L3X3X1/2-3	2L3X3X1/2-6	2L3X3X3/8
2L3X3X3/8-3	2L3X3X3/8-6	2L3X3X5/16	2L3X3X5/16-3
2L3X3X5/16-6	2L3X3X1/4	2L3X3X1/4-3	2L3X3X1/4-6
2L3X3X3/16	2L3X3X3/16-3	2L3X3X3/16-6	2L2.5X2.5X3/8
2L2.5X2.5X3/8-3	2L2.5X2.5X3/8-6	2L2.5X2.5X5/16	2L2.5X2.5X5/16-3
2L2.5X2.5X5/16-6	2L2.5X2.5X1/4	2L2.5X2.5X1/4-3	2L2.5X2.5X1/4-6
2L2.5X2.5X3/16	2L2.5X2.5X3/16-3	2L2.5X2.5X3/16-6	2L2X2X3/8
2L2X2X3/8-3	2L2X2X3/8-6	2L2X2X5/16	2L2X2X5/16-3

Built-in AISC Property Designations (continued)
Figure IV-3

WT13.5X108.5	WT13.5X97	WT13.5X89	WT13.5X80.5
WT13.5X73	WT13.5X64.5	WT13.5X57	WT13.5X51
WT13.5X47	WT13.5X42	WT12X88	WT12X81
WT12X73	WT12X65.5	WT12X58.5	WT12X52
WT12X51.5	WT12X47	WT12X42	WT12X38
WT12X34	WT12X31	WT12X27.5	WT10.5X83
WT10.5X73.5	WT10.5X66	WT10.5X61	WT10.5X55.5
WT10.5X50.5	WT10.5X46.5	WT10.5X41.5	WT10.5X36.5
WT10.5X34	WT10.5X31	WT10.5X28.5	WT10.5X25
WT10.5X22	WT9X71.5	WT9X65	WT9X59.5
WT9X53	WT9X48.5	WT9X43	WT9X38
WT9X35.5	WT9X32.5	WT9X30	WT9X27.5
WT9X25	WT9X23	WT9X20	WT9X17.5
WT8X50	WT8X44.5	WT8X38.5	WT8X33.5
WT8X28.5	WT8X25	WT8X22.5	WT8X20
WT8X18	WT8X15.5	WT8X13	WT7X365
WT7X332.5	WT7X302.5	WT7X275	WT7X250
WT7X227.5	WT7X213	WT7X199	WT7X185
WT7X171	WT7X155.5	WT7X141.5	WT7X128.5
WT7X116.5	WT7X105.5	WT7X96.5	WT7X88
WT7X79.5	WT7X72.5	WT7X66	WT7X60
WT7X54.5	WT7X49.5	WT7X45	WT7X41
WT7X37	WT7X34	WT7X30.5	WT7X26.5
WT7X24	WT7X21.5	WT7X19	WT7X17
WT7X15	WT7X13	WT7X11	WT6X168
WT6X152.5	WT6X139.5	WT6X126	WT6X115
WT6X105	WT6X95	WT6X85	WT6X76
WT6X68	WT6X60	WT6X53	WT6X48
WT6X43.5	WT6X39.5	WT6X36	WT6X32.5
WT6X29	WT6X26.5	WT6X25	WT6X22.5
WT6X20	WT6X17.5	WT6X15	WT6X13
WT6X11	WT6X9.5	WT6X8	WT6X7
WT5X56	WT5X50	WT5X44	WT5X38.5
WT5X34	WT5X30	WT5X27	WT5X24.5
WT5X22.5	WT5X19.5	WT5X16.5	WT5X15
WT5X13	WT5X11	WT5X9.5	WT5X8.5
WT5X7.5	WT5X6	WT4X33.5	WT4X29
WT4X24	WT4X20	WT4X17.5	WT4X15.5
WT4X14	WT4X12	WT4X10.5	WT4X9
WT4X7.5	WT4X6.5	WT4X5	WT3X12.5
WT3X10	WT3X7.5	WT3X8	WT3X6
WT3X4.5	WT2.5X9.5	WT2.5X8	WT2X6.5
MT7X9	MT6X5.9	MT5X4.5	MT4X3.25
MT3X10	MT3X2.2	MT2.5X9.45	MT2X6.5
ST12X60.5	ST12X53	ST12X50	ST12X45
ST12X40	ST10X48	ST10X43	ST10X37.5
ST10X33	ST9X35	ST9X27.35	ST7.5X25
ST7.5X21.45	ST6X25	ST6X20.4	ST6X17.5
ST6X15.9	ST5X17.5	ST5X12.7	ST4X11.5
ST4X9.2	ST3.5X10	ST3.5X7.65	ST3X8.625
ST3X6.25	ST2.5X7.375	ST2.5X5	ST2X4.75
ST2X3.85	ST1.5X3.75	ST1.5X2.85	L9X4X5/8
L9X4X9/16	L9X4X1/2	L8X8X9/8	L8X8X1
L8X8X7/8	L8X8X3/4	L8X8X5/8	L8X8X9/16
L8X8X1/2	L8X6X1	L8X6X7/8	L8X6X3/4
L8X6X5/8	L8X6X9/16	L8X6X1/2	L8X6X7/16

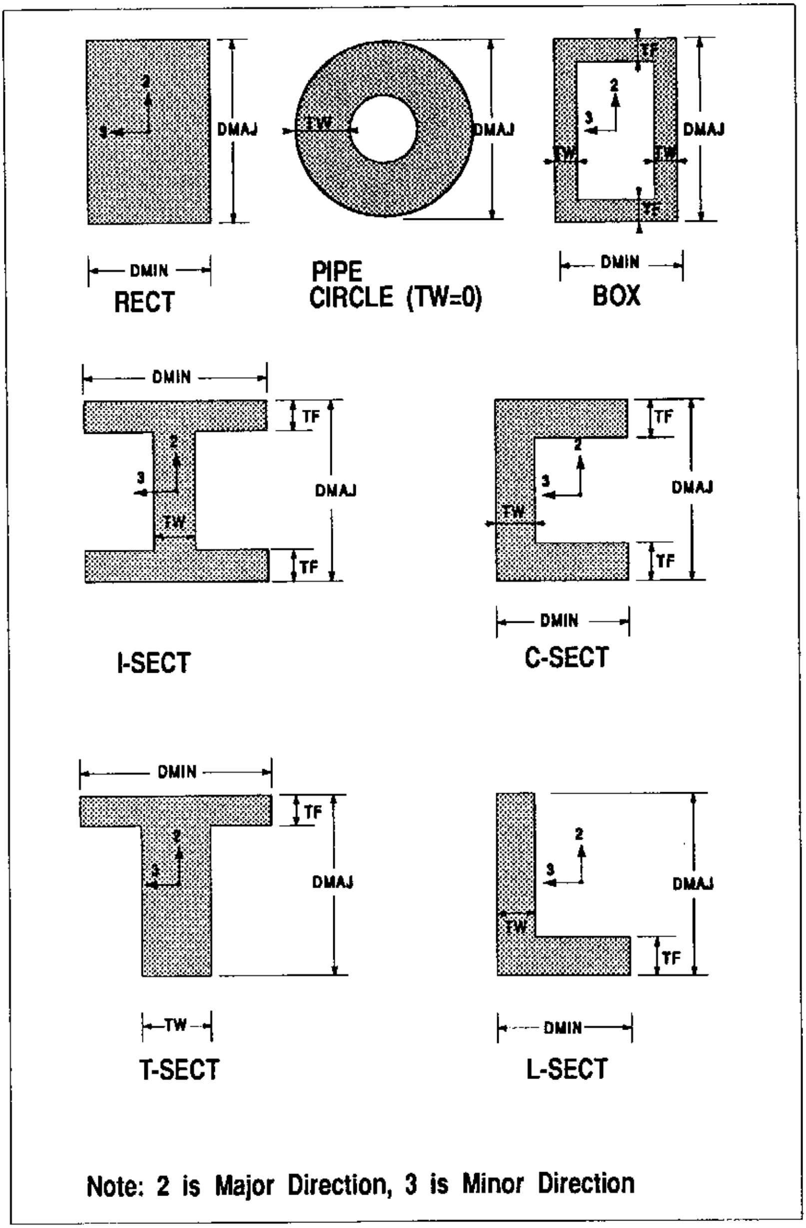
Built-in AISC Property Designations (continued)
Figure IV-3

W12X279	W12X252	W12X230	W12X210
W12X190	W12X170	W12X152	W12X136
W12X120	W12X106	W12X96	W12X87
W12X79	W12X72	W12X65	W12X58
W12X53	W12X50	W12X45	W12X40
W12X35	W12X30	W12X26	W12X22
W12X19	W12X16	W12X14	W10X112
W10X100	W10X88	W10X77	W10X68
W10X60	W10X54	W10X49	W10X45
W10X39	W10X33	W10X30	W10X26
W10X22	W10X19	W10X17	W10X15
W10X12	W8X67	W8X58	W8X48
W8X40	W8X35	W8X31	W8X28
W8X24	W8X21	W8X18	W8X15
W8X13	W8X10	W6X25	W6X20
W6X15	W6X16	W6X12	W6X9
W5X19	W5X16	W4X13	M14X18
M12X11.8	M10X9	M8X6.5	M6X20
M6X4.4	M5X18.9	M4X13	S24X121
S24X106	S24X100	S24X90	S24X80
S20X96	S20X86	S20X75	S20X66
S18X70	S18X54.7	S15X50	S15X42.9
S12X50	S12X40.8	S12X35	S12X31.8
S10X35	S10X25.4	S8X23	S8X18.4
S7X20	S7X15.3	S6X17.25	S6X12.5
S5X14.75	S5X10	S4X9.5	S4X7.7
S3X7.5	S3X5.7	HP14X117	HP14X102
HP14X89	HP14X73	HP13X100	HP13X87
HP13X73	HP13X60	HP12X84	HP12X74
HP12X63	HP12X53	HP10X57	HP10X42
HP8X36	C15X50	C15X40	C15X33.9
C12X30	C12X25	C12X20.7	C10X30
C10X25	C10X20	C10X15.3	C9X20
C9X15	C9X13.4	C8X18.75	C8X13.75
C8X11.5	C7X14.75	C7X12.25	C7X9.8
C6X13	C6X10.5	C6X8.2	C5X9
C5X6.7	C4X7.25	C4X5.4	C3X6
C3X5	C3X4.1	MC18X58	MC18X51.9
MC18X45.8	MC18X42.7	MC13X50	MC13X40
MC13X35	MC13X31.8	MC12X50	MC12X45
MC12X40	MC12X35	MC12X31	MC12X10.6
MC10X41.1	MC10X33.6	MC10X28.5	MC10X25
MC10X22	MC10X8.4	MC10X6.5	MC9X25.4
MC9X23.9	MC8X22.8	MC8X21.4	MC8X20
MC8X18.7	MC8X8.5	MC7X22.7	MC7X19.1
MC7X17.6	MC6X18	MC6X15.3	MC6X16.3
MC6X15.1	MC6X12	WT18X179.5	WT18X164
WT18X150	WT18X140	WT18X130	WT18X122.5
WT18X115	WT18X128	WT18X116	WT18X105
WT18X97	WT18X91	WT18X85	WT18X80
WT18X75	WT18X67.5	WT16.5X177	WT16.5X159
WT16.5X145.5	WT16.5X131.5	WT16.5X120.5	WT16.5X110.5
WT16.5X100.5	WT16.5X84.5	WT16.5X76	WT16.5X70.5
WT16.5X65	WT16.5X59	WT15X117.5	WT15X105.5
WT15X95.5	WT15X86.5	WT15X74	WT15X66
WT15X62	WT15X58	WT15X54	WT15X49.5

Built-in AISC Property Designations (continued)
Figure IV-3

W40X328	W40X298	W40X268	W40X244
W40X221	W40X192	W40X655	W40X593
W40X531	W40X480	W40X436	W40X397
W40X362	W40X324	W40X297	W40X277
W40X249	W40X215	W40X199	W40X183
W40X167	W40X149	W36X848	W36X798
W36X720	W36X650	W36X588	W36X527
W36X485	W36X439	W36X393	W36X359
W36X328	W36X300	W36X280	W36X260
W36X245	W36X230	W36X256	W36X232
W36X210	W36X194	W36X182	W36X170
W36X160	W36X150	W36X135	W33X619
W33X567	W33X515	W33X468	W33X424
W33X387	W33X354	W33X318	W33X291
W33X263	W33X241	W33X221	W33X201
W33X169	W33X152	W33X141	W33X130
W33X118	W30X581	W30X526	W30X477
W30X433	W30X391	W30X357	W30X326
W30X292	W30X261	W30X235	W30X211
W30X191	W30X173	W30X148	W30X132
W30X124	W30X116	W30X108	W30X99
W30X90	W27X539	W27X494	W27X448
W27X407	W27X368	W27X336	W27X307
W27X281	W27X258	W27X235	W27X217
W27X194	W27X178	W27X161	W27X146
W27X129	W27X114	W27X102	W27X94
W27X84	W24X492	W24X450	W24X408
W24X370	W24X335	W24X306	W24X279
W24X250	W24X229	W24X207	W24X192
W24X176	W24X162	W24X146	W24X131
W24X117	W24X104	W24X103	W24X94
W24X84	W24X76	W24X68	W24X62
W24X55	W21X402	W21X364	W21X333
W21X300	W21X275	W21X248	W21X223
W21X201	W21X182	W21X166	W21X147
W21X132	W21X122	W21X111	W21X101
W21X93	W21X83	W21X73	W21X68
W21X62	W21X57	W21X50	W21X44
W18X311	W18X283	W18X258	W18X234
W18X211	W18X192	W18X175	W18X158
W18X143	W18X130	W18X119	W18X106
W18X97	W18X86	W18X76	W18X71
W18X65	W18X60	W18X55	W18X50
W18X46	W18X40	W18X35	W16X100
W16X89	W16X77	W16X67	W16X57
W16X50	W16X45	W16X40	W16X36
W16X31	W16X26	W14X730	W14X665
W14X605	W14X550	W14X500	W14X455
W14X426	W14X398	W14X370	W14X342
W14X311	W14X283	W14X257	W14X233
W14X211	W14X193	W14X176	W14X159
W14X145	W14X132	W14X120	W14X109
W14X99	W14X90	W14X82	W14X74
W14X68	W14X61	W14X53	W14X48
W14X43	W14X38	W14X34	W14X30
W14X26	W14X22	W12X336	W12X305

Built-in AISC Property Designations
Figure IV-3



Required Dimensions for Automatic Section Property Generation
Figure IV-2

4. SECTION PROPERTY REDEFINTION 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, (i) through (iii) below, to redefine the section property tables of the various element types that make up the frames in the structure.

3 - NOTES: (continued)

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 or the appropriate appendices. However, if these values are input by the user, they will override all program-calculated values.

Appendix A.

STRESS CHECK AND DESIGN ALGORITHMS

(Uniform Building Code 1991)

This appendix 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, 1991 (UBC91) [2].

References to pertinent sections and equations of the UBC91 are indicated with the "UBC" prefix.

A. DESIGN LOAD COMBINATIONS

Following are the default load combinations if UBC91 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 these load combinations are acceptable the user need not specify any load combinations in the STEELER input data file.

1. $DL + LL$
2. $0.75\,(DL + LL + \sqrt{EX^2 + EY^2})$
3. $0.75\,(DL + LL - \sqrt{EX^2 + EY^2})$

4. $0.75\,(DL \quad + \sqrt{EX^2 + EY^2})$
5. $0.75\,(DL \quad - \sqrt{EX^2 + EY^2})$

B. SPECIAL SEISMIC CONSIDERATIONS

For the UBC code analysis the program automatically considers the special seismic design requirements given in Section 2710 of the code. The special requirements checked by the program are dependent on the type of framing used and are described below for each type of framing.

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 and the two lateral load conditions A and B are associated with the two orthogonal seismic loadings **except if a dynamic analysis has been made.** For the case of a dynamic analysis, Load conditions Dyn-1 and Dyn-2 are considered active and correspond to the two orthogonal directions of seismic excitation. 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 condition.

1. Ordinary Moment Resisting 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 2710 (e) 1):

$$P_{DL} + 0.7P_{LL} + \frac{3}{8}R_w\sqrt{P_X^2 + P_Y^2}$$

$$P_{DL} + 0.7P_{LL} - \frac{3}{8}R_w\sqrt{P_X^2 + P_Y^2}$$

$$0.85P_{DL} + \frac{3}{8}R_w\sqrt{P_X^2 + P_Y^2}$$

$$0.85P_{DL} - \frac{3}{8}R_w\sqrt{P_X^2 + P_Y^2}$$

where P_{DL}, P_{LL}, P_X, and P_Y are axial loads due to dead load, live load, seismic loads in the X direction and seismic loads in the Y direction, 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 2710 (f)):

$$V_{DL} + V_{LL} + \frac{3}{8} R_w \sqrt{V_x^2 + V_y^2}$$

$$V_{DL} + V_{LL} - \frac{3}{8} R_w \sqrt{V_x^2 + V_y^2}$$

c) The continuity plates reported are for a beam flange force, $P_{bf} = A_{bf} F_{yb}$. See Chapter III, 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 a) above for ordinary moment resisting 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 2710 (g) 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 both ends of the beam are pinned, or cantilever beams.
	=	1 if one of the ends of the beam is pinned.
	=	2 if none of the ends of the beam are pinned.
M_{pb}	=	Plastic moment capacity of the beam.
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. Also, the capacity of the panel zone in resisting this shear is taken as (UBC 2710(g) 2A):

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

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 $52 / \sqrt{F_y}$. If this criteria is satisfied the section is reported as SEISMIC as described earlier in Section E.1 above. If this criteria is not satisfied the user must modify the section property (UBC 2710 (g) 3). Other members meeting this criteria are also reported as SEISMIC.

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 2710 (g) 4). For design of the continuity plate the beam flange force is taken as $A_{bf}\ F_{yb}$. See Chapter III, Section E.2 for details.

f) To facilitate the review of the strong column weak beam criterion (UBC 2710.(g).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

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

where

$R_{maj,min}$	=	Plastic moment capacity ratios, in the major and minor directions of the column, respectively.
M_{pb_n}	=	Plastic moment capacity of n-th beam connecting to column.
q_n	=	Angle between the n-th beam and the column major direction.
$M_{pca_{x,y}}$	=	Major and minor plastic moment capacities, reduced for axial force effects, of columns above story level.
$M_{pcb_{x,y}}$	=	Major and minor plastic moment capacities, reduced for axial force effects, of columns 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.)

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 2710 (g) 8).

3. Braced Frames

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

a) The additional requirement for column axial strength as noted in a) above for ordinary moment resisting 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 $\frac{720}{\sqrt{F_y}}$. If this check is not met, it is noted in the output (UBC 2710 (h) 2 A).

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

$$\beta = \frac{1}{1 + \frac{\frac{kl}{r}}{2C_c}} \qquad \text{(UBC 2710 (h) 2 B)}$$

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 (UBC 2710 (h) 3)

e) Chevron braces are designed for 1.5 times the specified loading combinations (UBC 2710 (h) 4 A (i)). (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 A-1. 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 Chapter III 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

2710 (i) 1). 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 $V_S \le 2.2\ M_S / e$ 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 2710 (i) 2).

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, Dyn-1 or Dyn-2) is checked to be less than the following values (UBC 2710 (i) 3):

$\Theta \le 0.060$ where link beam clear length $\le 1.6\ M_S / V_S$

$\Theta \le 0.015$ where link beam clear length $\ge 2.6\ M_S / V_S$

and $\Theta \le$ value interpolated between 0.060 and 0.015 as the link beam clear length varies from $1.6\ M_S / V_S$ to $2.6\ M_S / V_S$.

The value of Rw 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 2710

(i) (4). (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 2710 (i) 5).

f) The link beam connection shear is reported as equal to the link beam web shear capacity (UBC 2710 (i) 11).

g) The brace strength in compression (computed as above) is checked to be at least 1.5 times the axial force corresponding to the controlling link beam strength(UBC 2710 (i) 12). 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 2710 (i) 2.B). Where f_a is the lower of the axial stress in the link beam corresponding to yielding of the link beam web in shear or the link beam flanges in flexure. The correspondence between brace force and link beam forces is obtained from Load conditions A, B, Dyn-1, or Dyn-2 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 computed corresponding column forces at the strength of the eccentric braced frame bay (link beam strength) as computed in h) above (UBC 2710 (i) 13). The correspondence between column forces and link beam forces is obtained from Load conditions A, B, Dyn-1 or Dyn-2 whichever has the highest link beam shear force. 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

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

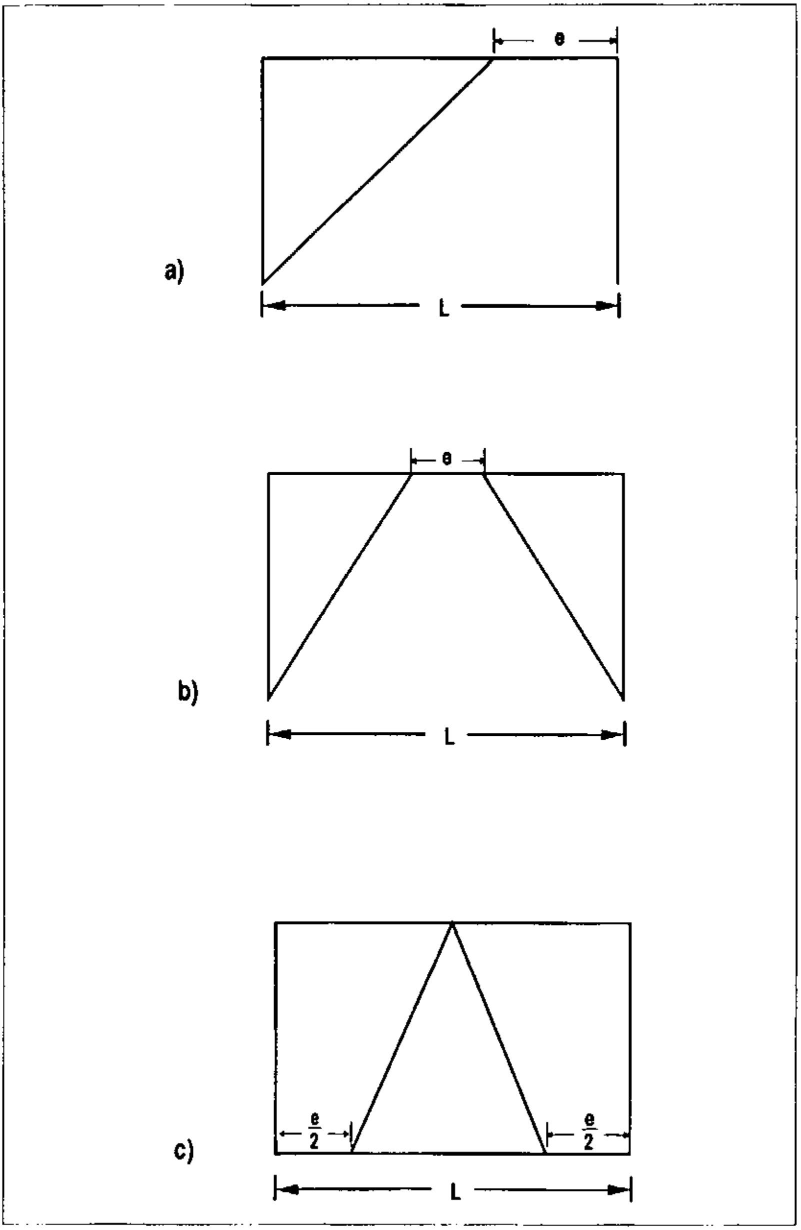
i) Axial forces in the beams are included in checking of the beams (UBC 2710 (i) 16).

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 2710 (i) 17).

k) The continuity plate requirements are checked for a beam flange force of $P_{bf} = A_{fb} F_{yb}$. See Chapter III, Section E.2 for details.

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



Eccentrically Braced Frame Configurations
Figure A-1

Appendix B.

CAPACITY CHECK AND DESIGN ALGORITHMS

(AISC Plastic 1989)

For the capacity check and design algorithms based on the American Institute of Steel Construction's Plastic Design Specifications (AISC Plastic 89) [3] this appendix replaces in its entirety Chapter III which is based on the AISC-ASD89. This appendix describes in detail the various aspects of the capacity check procedures that are used by the program STEELER.

Equivalent chapters for the UBC91 [2], for the AISC-LRFD86 [4] and for the CISC89 [5] specifications are presented in other appendices.

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 is assumed.

References to pertinent sections and equations of the AISC-Plastic 89 are indicated with the "AISC" prefix.

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 postprocessing file brings across forces and moments associated with the eight independent load conditions (I, II, III, A, B...) 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.

If a structure is subjected to dead load (DL) and live load (LL) only, the capacity check may need only one loading combination, namely 1.7 (DL + LL) (AISC N1).

However, if in addition to the dead load 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. 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 Plastic 1989 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 capacity ratios from the controlling load combinations are reported.

It is possible, in the postprocessing 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 capacity ratios.

B. THE UNSUPPORTED LENGTHS OF THE ELEMENTS

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

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 B-1. These are the lengths between the support points of the element in the corresponding directions.

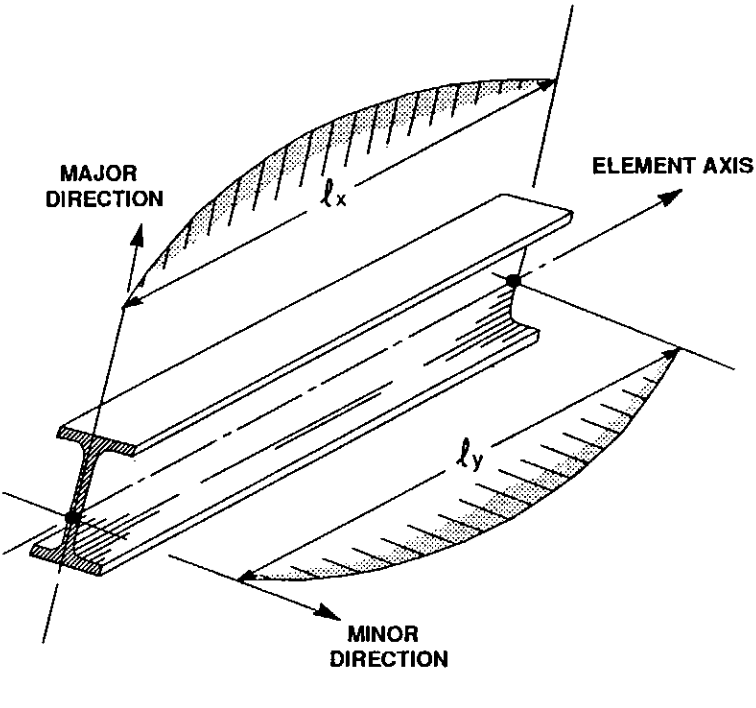
For beam- and column-type elements, the program will actually travel past any unsupported ends of the elements to automatically locate the element support point, and evaluate the corresponding unsupported element length.

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.

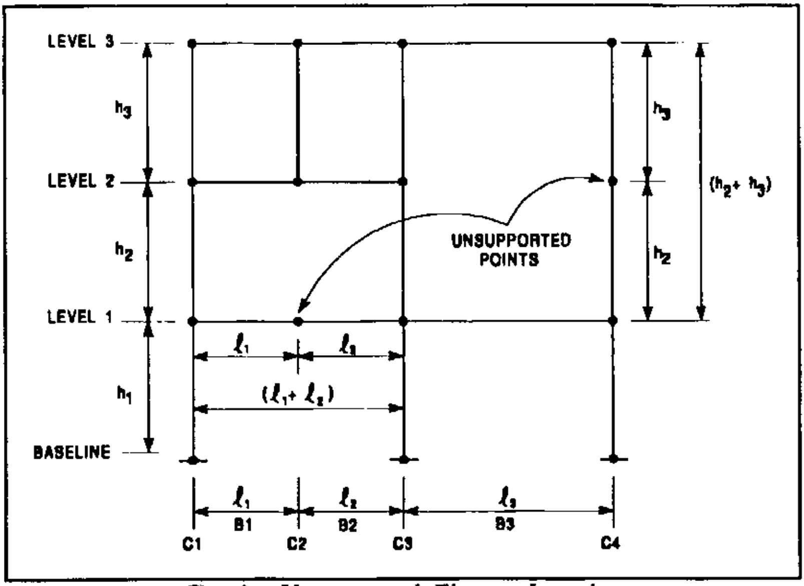
As shown in Figure B-2, the unsupported length l_x for the beam in Bay 1 at level 1 will not be l_1, but $(l_1 + l_2)$; and the unsupported length for the column at column line 4 at level 3 will not be h_3, but $(h_2 + h_3)$.

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 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.



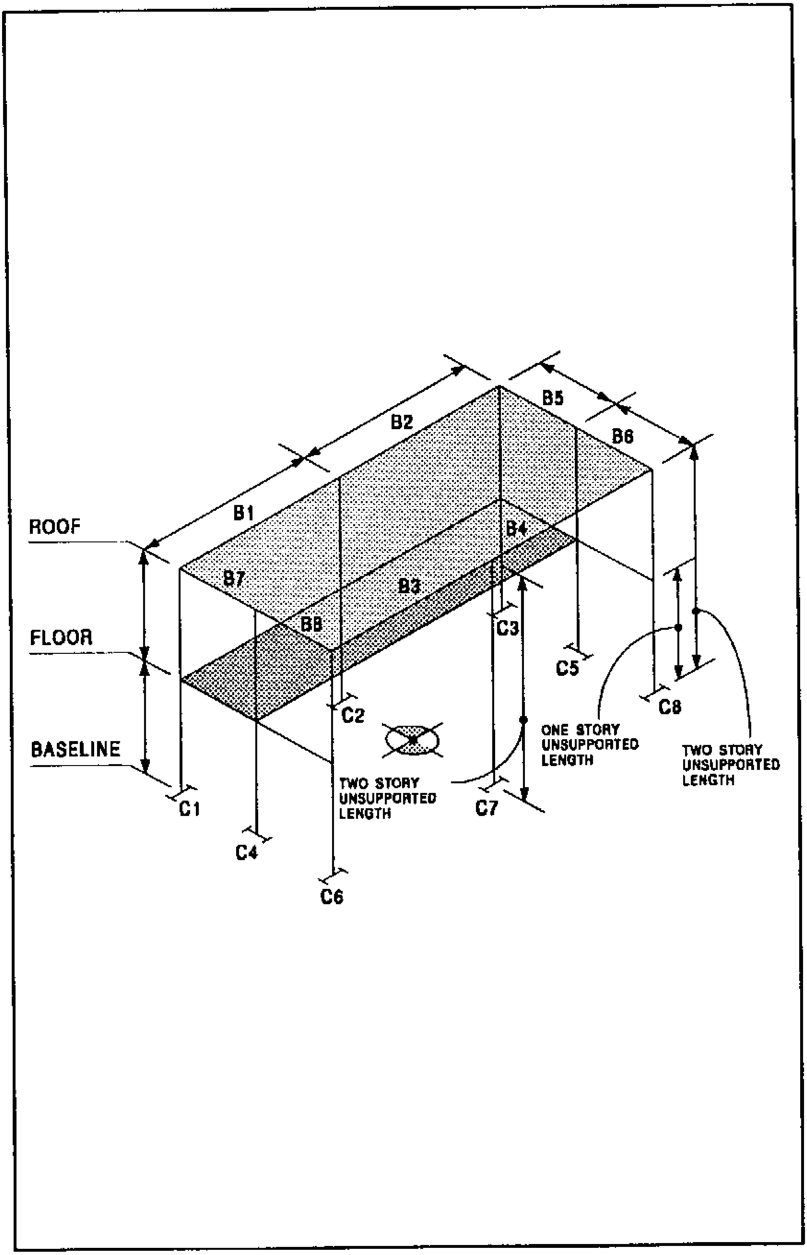
Element Unsupported Length
Figure B-1



Tracing Unsupported Element Lengths
Figure B-2

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 B-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.



Conditions Affecting Unsupported Lengths
Figure B-3

Similarly, the lateral unsupported lengths for beams are as shown in Figure B-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. THE 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:

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

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.

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 in both directions.

If no pin exists at the baseline of a column, the corresponding G-value is set to 1.0 in both directions.

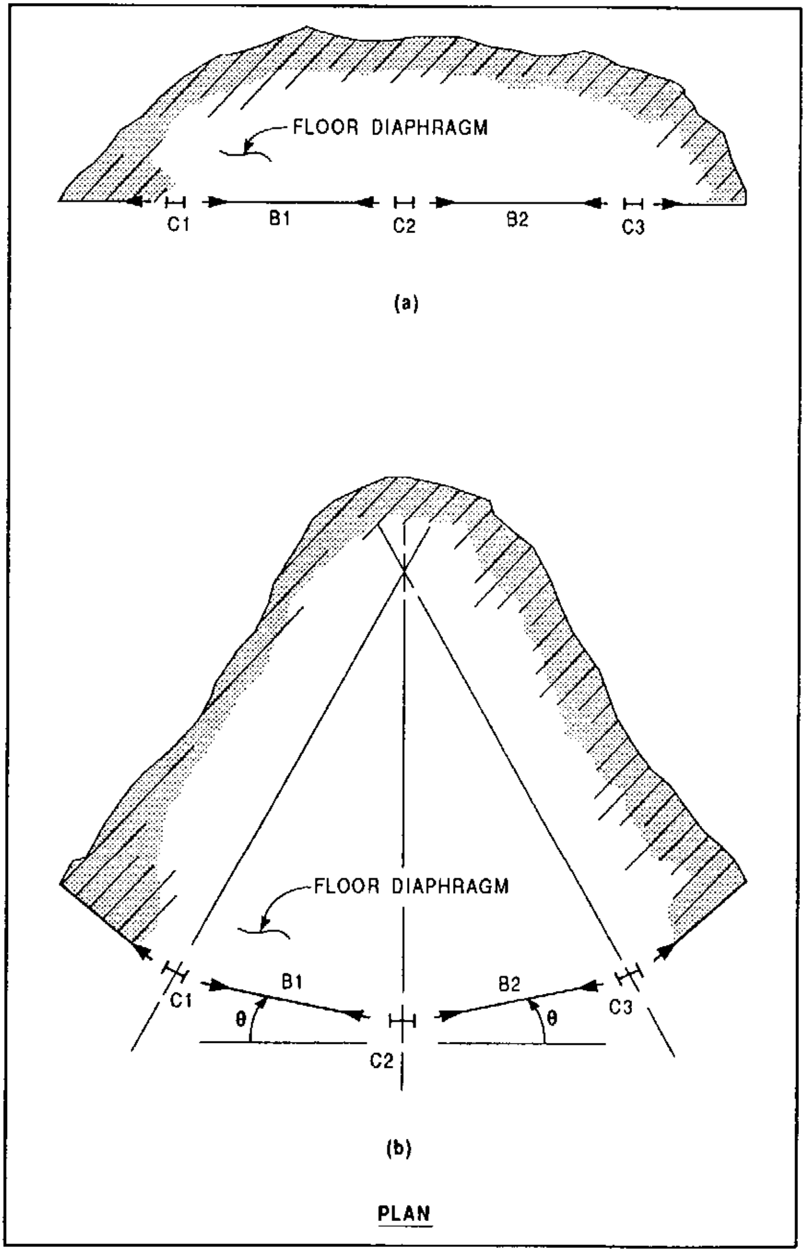
If there are no beams framing into a particular direction of a column, the associated G-value will be infinity. If the G-value at any one end of a column for a particular direction is infinity, the K-factor corresponding to that direction is set equal to unity.

If rotational releases exist at both ends of a column, the corresponding K-factors are set to unity.

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

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

For example, in Figure B-4(a) 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



K-Factor Ambiguity
Figure B-4

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 Figure B-4(b) for column line C2. The small components of the beam stiffnesses 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.

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 option if the user wants to use inelastic K-factors as opposed to the elastic K-factors calculated by the program.

D. THE ELEMENT STATIONS AND FORCE COMPONENTS

- The column, beam and brace elements are capacity 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 diaphragm at the level of the beam.

E. THE CAPACITY CHECK AND DESIGN PROCEDURE

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 Plastic Design Specifications [3]. The various parameters referenced in this section are tabulated in Figure B-5.

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

A	=	Cross-sectional area, in^2
A_{vx}, A_{vy}	=	Major and minor shear areas, in^2
C_{mx},C_{my}	=	Major and minor direction moment coefficients
E	=	Modulus of elasticity, ksi
F_a	=	Allowable compressive axial stress, ksi
F_y	=	Yield stress of material, ksi
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
M_{mx}	=	$\left(1.07 - \frac{(l_y/r_y)\sqrt{F_y}}{3160}\right)M_{px} \leq M_{px}$
M_{px}, M_{py}	=	Major and minor plastic moments, kip-in
M_x, M_y	=	Factored major and minor moments in member, kip-in
P	=	Factored axial force in member, kips
P_{ex}	=	$\pi^2 AE \Big/ \left(K_x\frac{l_x}{r_x}\right)^2$
P_{ey}	=	$\pi^2 AE \Big/ \left(K_y\frac{l_y}{r_y}\right)^2$
P_{cr}	=	Compressive axial strength, kips
P_y	=	Axial force at yield, AF_y, kips
V_{px}, V_{py}	=	Major and minor shear strengths, kips
V_x, V_y	=	Factored major and minor shear forces, kips
Z_x, Z_y	=	Major and minor direction plastic moduli, in^3
b	=	$b_f - 2t_w$ for welded BOX sections, in

Capacity Check Parameters
Figure B-5

	=	b_f - $3t_f$ for rolled BOX (TS) section, in
b_f	=	Flange width, in
d	=	Depth of web, 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_f	=	Flange thickness, in
t_w	=	Thickness of web, in

Capacity Check Parameters
Figure B-5 (continued)

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 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 certain limits. These limits are shown in Figure B-6. 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.

Description of Section	**Ratio checked**	**LIMITS FOR PLASTIC MEMBER**
I-SHAPES flanges	bf/2tf	8.5 for $Fy = 36$ 8.0 $= 42$ 7.4 $= 45$ 7.0 $= 50$ 6.6 $= 55$ 6.3 $= 60$ 6.0 $= 65$
webs	d/tw	$\frac{412}{\sqrt{Fy}}\left(1-1.4\frac{P}{Py}\right)$ for $\frac{P}{Py} \leq 0.27$ $257/\sqrt{Fy}$ for $\frac{P}{Py} > 0.27$
unbraced length	ly/ry	$\frac{1375}{Fy}+25$ for $1.0 > \frac{M}{Mpx} > -0.5$ $\frac{1375}{Fy}$ for $-0.5 \geq \frac{M}{Mpx} > -1.0$
BOX flanges webs unbraced length	b/tf d/tw ly/ry	$190/\sqrt{Fy}$ 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

Plastic Member Criteria
Figure B-6

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 $\frac{kl}{r} \le C_c$,

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

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

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

otherwise, if $Kl/r > C_c$

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

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 stress 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 stress ratio is given by the larger of CR1, CR2 and CR3, where

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

(AISC Eqn. N4-2)

$$CR2 = \frac{P}{P_y} + \frac{M_x}{1.18\,M_{px}} + \frac{M_y}{1.18\,M_{py}}$$

(AISC Eqn. N4-3)

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

Where C_{mx} and C_{my} are co-efficients 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

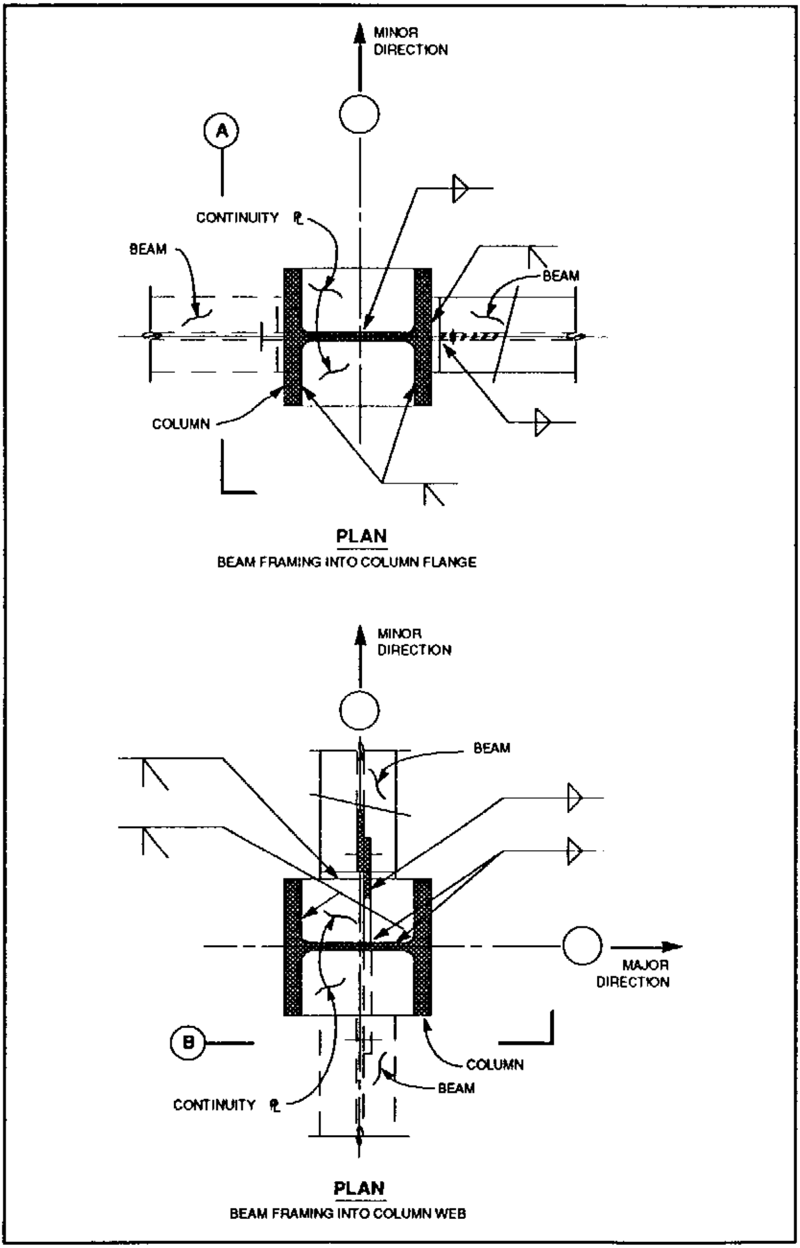
$$TR = \frac{P}{P_y} + \frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \qquad \text{(TENSION)}$$

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

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



Plan Showing Continuity Plates
Figure B-8

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

b) The steel beam frames in a direction parallel to the column minor axis, 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.

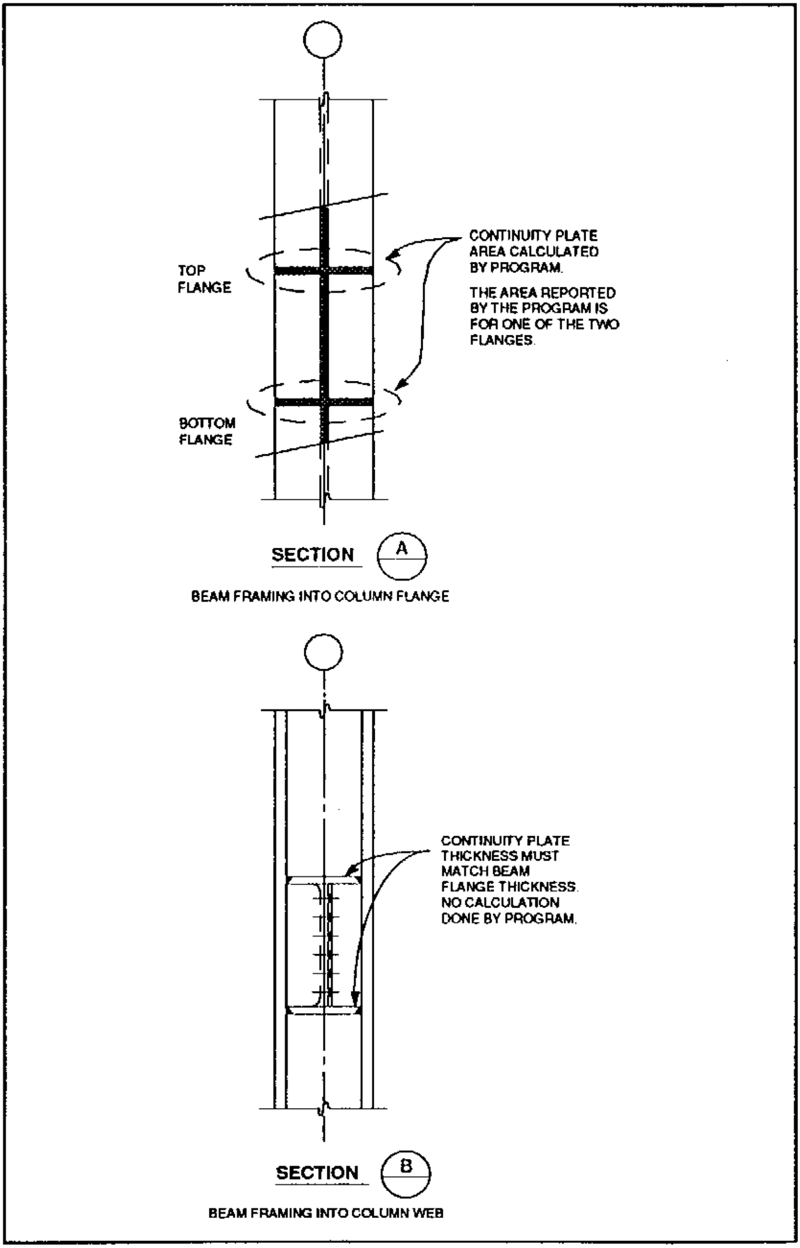
To achieve a beam/column moment connection, continuity plates such as shown in Figures B-8 and B-9 are usually placed on the column, in line with the top and bottom flanges of the beam, to transfer the compression and tension flange forces of the beam into the column.

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 . Also braced frames are not investigated for continuity plate requirements.

The program evaluates the continuity plate requirements for each of the beams that frame into the column flange (i.e. parallel to the column major direction) and reports the maximum continuity plate area that is needed.



Section Showing Continuity Plates
Figure B-9

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

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

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:

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

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

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

Where $P_{bf} = F_{yb} A_{bf}$.

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:

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

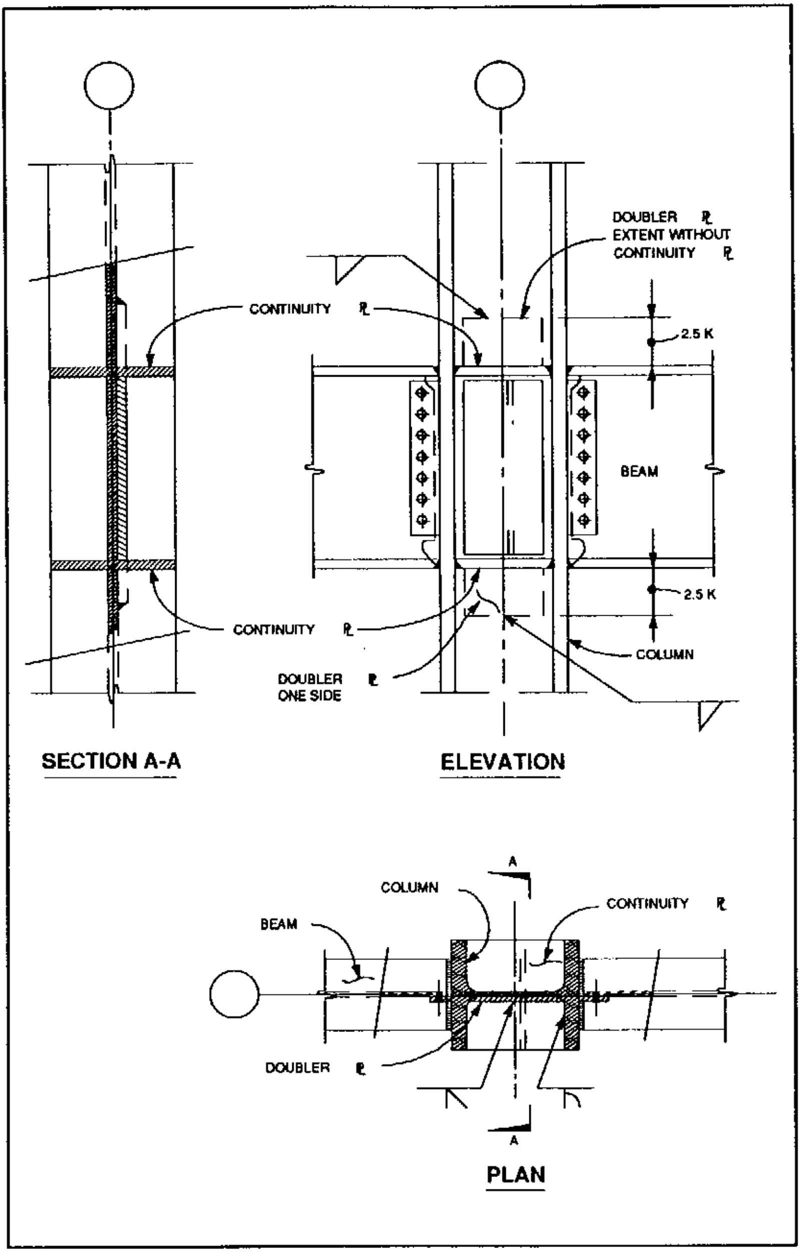
Therefore, the continuity plate area provided by the program is either zero or the greater of A_{cp} and A_{cp}^{min}.

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
b_{fb}	=	Beam flange width
t_{cp}	=	Continuity plate thickness
b_{cp}	=	Continuity plate width

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.



Doubler Plates
Figure B-10

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. See Figure B-10. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear stresses are seldom critical, and this condition is therefore not investigated by the program.

Shear stresses in the panel zone, due to major direction bending in the column, may require additional plates to be welded onto the column web, depending upon the loading and the geometry of the steel beams that frame into the column, either along the column major direction, or at an angle so that the beams have components along the column major direction. The program investigates such situations and reports the thicknesses of any required doubler plates. Only columns with I shapes are investigated for doubler plate requirements. Also braced frames are not investigated for doubler plate requirements.

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 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_y	=	Column yield stress
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
nb	=	Number of beams connecting to column
d_n	=	Depth of n-th beam connecting to column
q_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	=	$0.55F_y$

4. Evaluation of Beam Joint 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.

Appendix C.

CAPACITY CHECK AND DESIGN ALGORITHMS

(AISC-LRFD 1986)

For the capacity check and design algorithms based on the American Institute of Steel Construction's Load and Resistance Factor Design Specifications (AISC-LRFD86) [4] this appendix replaces in its entirety Chapter III which is based on the AISC-ASD89. This appendix describes in detail the various aspects of the capacity check procedures that are used by the program STEELER.

Equivalent chapters for the UBC91 [2], for the AISC Plastic 89 [3] and for the CISC89 [5] specifications are presented in other appendices.

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 is assumed.

References to pertinent sections and equations of the AISC-LRFD86 are indicated with the "AISC" prefix.

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 postprocessing file brings across forces and moments associated with the eight independent load conditions (I, II, III, A, B...) 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.

If a structure is subjected to dead load (DL) and live load (LL) only, the stress check may need the following two loading combinations (AISC A4.1):

1. 1.4 DL
2. 1.2 DL + 1.6 LL

However, if in addition to the dead load 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 ten load combinations may have to be defined :

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-LRFD 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.

It is possible in the postprocessing 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 capacity ratios.

When using the AISC-LRFD86 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 [8]. 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.

All tension and compression effects are consistently evaluated with these combinations.

The capacity ratios from the controlling load combinations are reported.

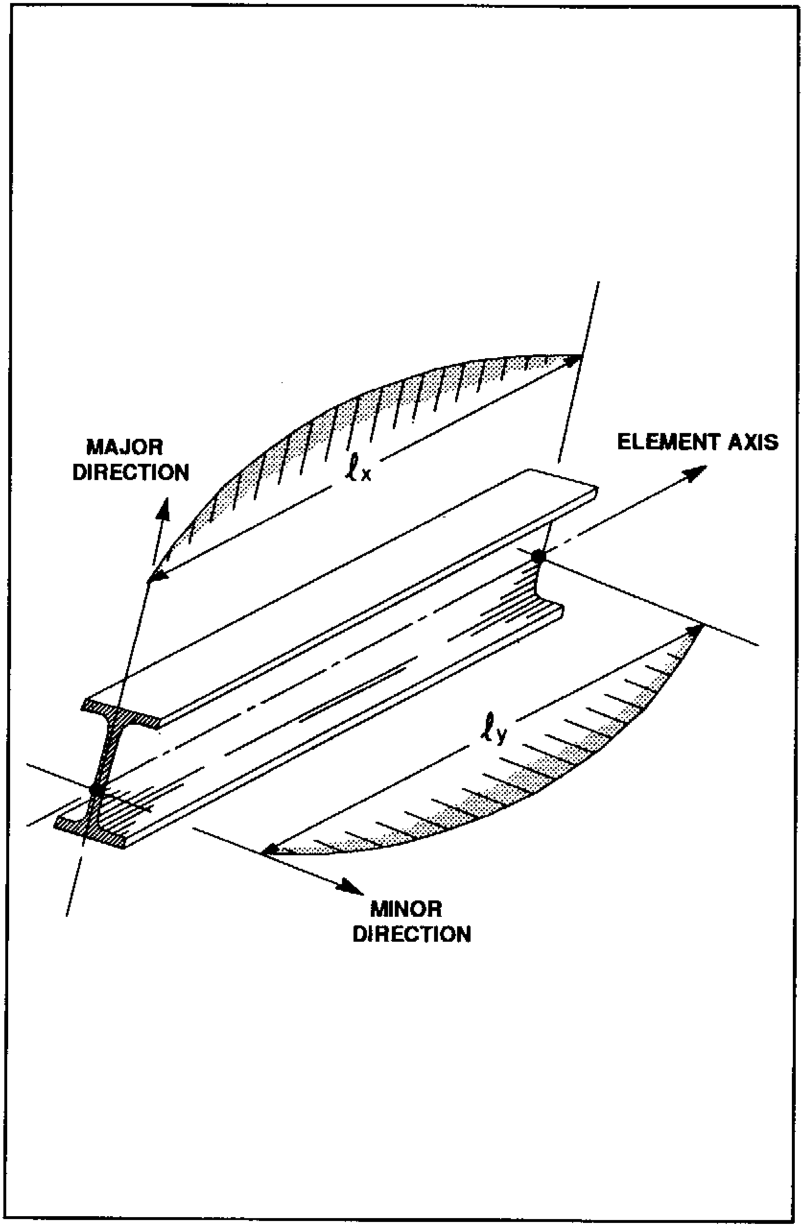
B. THE UNSUPPORTED LENGTHS OF THE ELEMENTS

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

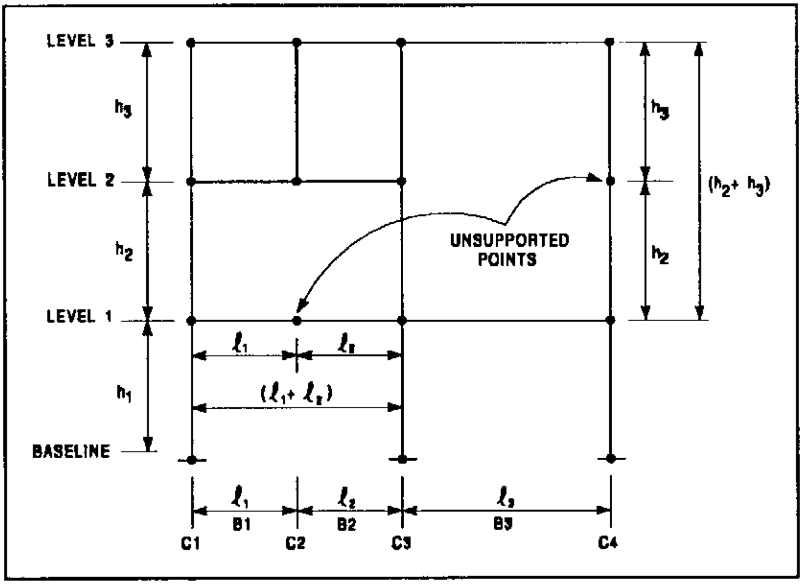
The two unsupported lengths are l_x and l_y, corresponding to instability in the major and minor directions of the element, respectively. See Figure C-1. These are the lengths between the support points of the element in the corresponding directions.

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.



Element Unsupported Length
Figure C-1

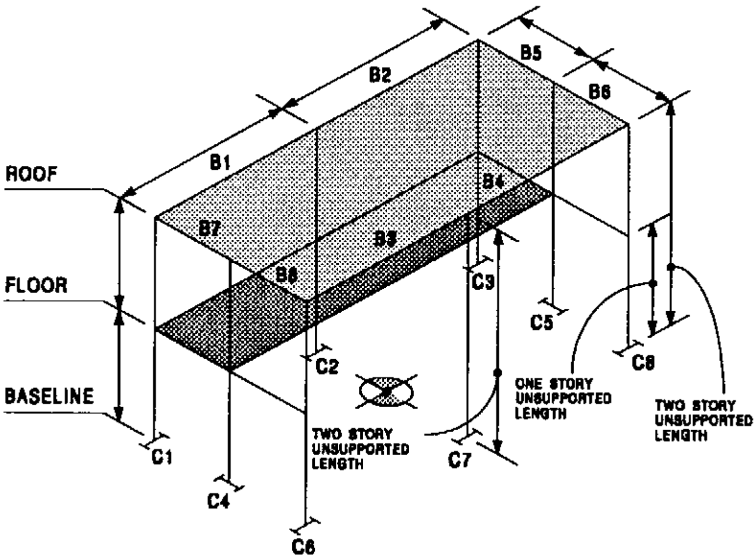


Tracing Unsupported Element Lengths
Figure C-2

As shown in Figure C-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 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.



Conditions Affecting Unsupported Lengths
Figure C-3

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 C-3, column line C8 laterally spans two stories in the column major direction and only spans 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 C-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. THE 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:

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

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.

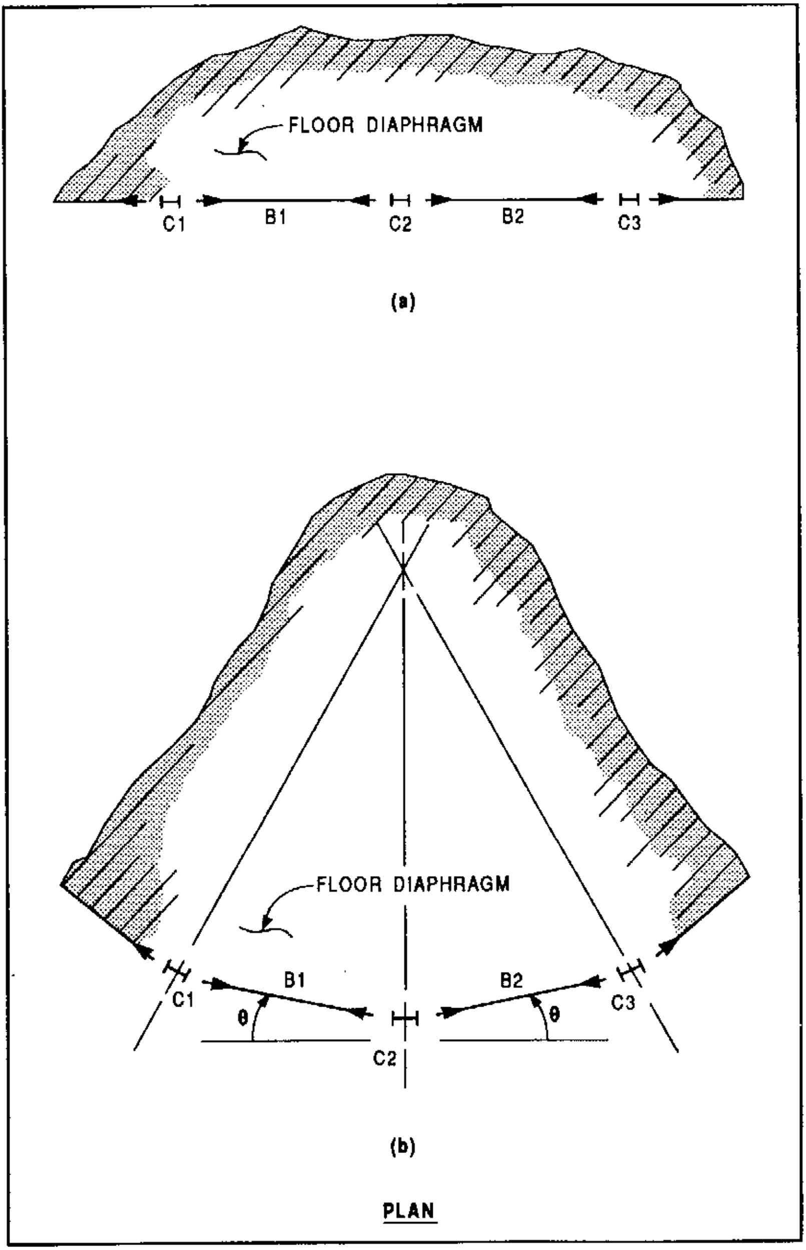
Cantilever beams and beams and columns having pin ends at the joint under consideration are excluded in the calculation of the stiffness EI/L summations.

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 in both directions.

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



K-Factor Ambiguity
Figure C-4

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.

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

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

For example, in Figure C-4(a) 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 Figure C-4(b) for column line C2. The small components of the beam stiffnesses 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.

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 option if the user wants to use inelastic K-factors as opposed to the elastic K-factors calculated by the program.

D. THE ELEMENT STATIONS AND FORCE COMPONENTS

The column, beam and brace elements are capacity 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 diaphragm at the level of the beam.

E. THE CAPACITY CHECK AND DESIGN PROCEDURE

This section defines the capactiy check and design procedure as it applies to the column, beam and brace elements and beam to column joints.

$\mathbf{A}$	=	Cross-sectional area, in^2
$\mathbf{A_g}$	=	Gross cross-sectional area, in^2
$\mathbf{A_{vx}}$, $\mathbf{A_{vy}}$	=	Major and minor shear areas, in^2
$\mathbf{A_w}$	=	Shear area, equal $\mathbf{dt_w}$ per web, in^2
$\mathbf{B_1}$	=	Moment magnification factor for moments not causing sidesway
$\mathbf{B_2}$	=	Moment magnification factor for moments causing sidesway
$\mathbf{C_b}$	=	Bending Coefficient
$\mathbf{C_m}$	=	Moment Coefficient
$\mathbf{C_w}$	=	Warping constant, in^6
$\mathbf{D}$	=	Outside diameter of pipes, in
$\mathbf{E}$	=	Modulus of elasticity, ksi
$\mathbf{F_{cr}}$	=	Critical compressive stress, ksi
$\mathbf{F_r}$	=	Compressive residual stress in flange, ksi assumed 10 ksi for rolled sections (section properties recovered from database) and 16.5 ksi for welded sections (section properties generated by program)
$\mathbf{F_y}$	=	Yield stress of material, ksi
$\mathbf{G}$	=	Shear modulus, ksi
$\mathbf{I_y}$	=	Minor moment of inertia, in^4
$\mathbf{J}$	=	Torsional constant for the section, in^4
$\mathbf{K}$	=	Effective length factor
$\mathbf{K_x}$, $\mathbf{K_y}$	=	Effective length K-factors in the major and minor directions
$\mathbf{L_b}$	=	Laterally unbraced length of member, in
$\mathbf{L_p}$	=	Limiting laterally unbraced length for full plastic capacity, in

Capacity Check Parameters
Figure C-5

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 the 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
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 BOX sections, in
	=	$b_f - 3t_f$ 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 section

Capacity Check Parameters
Figure C-5 (continued)

		and **d** - $2t_f$ for welded sections
k	=	Web plate buckling coefficient, assumed equal to 5 (no stiffeners)
k	=	Distance from outer face of flange to web toe of fillet , in
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

Capacity Check Parameters
Figure C-5 (continued)

The procedure is according to the AISC-LRFD86 [4]. The various parameters referenced in this section are tabulated in Figure C-5.

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 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.

The magnified moment in a particular direction is given by:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(AISC Eqn. H1-2)}$$

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, Dyn-1, Dyn-2 and Dyn-3)

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

$$B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1.0 \qquad \text{(AISC Eqn.H1-3)}$$

Description of Section	**Ratio checked** (λ)	**COMPACT** (λ_p)	**NON-COMPACT** (λ_r)
I-SHAPES flanges (rolled) flanges (welded) webs	 $bf/2tf$ $bf/2tf$ h_c/tw	 $65/\sqrt{Fy}$ $65/\sqrt{Fy}$ for $P_u/\phi_b Py \leq 0.125$ $\frac{640}{\sqrt{Fy}}\left(1 - \frac{2.75Pu}{\phi b Py}\right)$ for $P_u/\phi_b Py > 0.125$ $\frac{191}{\sqrt{Fy}}\left(2.33 - \frac{Pu}{\phi b Py}\right) \geq \frac{253}{\sqrt{Fy}}$	 $141/\sqrt{Fy - Fr}$ $106/\sqrt{Fy - Fr}$ $970/\sqrt{Fy}$
BOX flanges webs	 b/tf h_c/tw	 $190/\sqrt{Fy}$ as for I-shapes	 $238/\sqrt{Fy - Fr}$ as for I-shapes
CHANNELS flanges webs	 bf/tf h_c/tw	 as for I-shapes as for I-shapes	 as for I-shapes as for I-shapes
T-SHAPES flanges stem	 $bf/2tf$ d/tw	 Not applicable Not applicable	 $95/\sqrt{Fy}$ $127/\sqrt{Fy}$
ANGLES or DOUBLE ANGLES	b/t	Not applicable	$76/\sqrt{Fy}$
PIPES	D/t	$2070/Fy$	$3300/Fy$
RECT or CIRCLE	—	Assumed compact	—
USER	—	—	Assumed non-compact

Compact Section Criteria
Figure C-6

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 [8]. **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 C-6 and reports the classifications for the individual members. If the section dimensions satisfy the limits shown in Figure C-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 capacity check is terminated.

c. Calculation of Nominal Strengths

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

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

where

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

for $\lambda_c \leq 1.5$

and

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

for $\lambda_c > 1.5$

and

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

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.

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

The nominal axial **tensile** strength value P_n is assumed to be A_gF_y (AISC Eqn. 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).

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,

$$M_{nx} = C_b \left[M_{px} - (M_{px} - M_{rx})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_{px}$$

(AISC Eqn. F1-3)

when $L_b \leq L_r$

and

$$M_{nx} = M_{crx} \leq C_b M_{rx} \leq M_{px} \qquad \text{(AISC Eqn. F1-12)}$$

when $L_b > L_r$.

Where

M_{nx}	=	Nominal major bending strength
M_{px}	=	Major plastic moment, $Z_x F_y$
M_{rx}	=	Major limiting buckling moment, $(F_y - F_r)S_x$ (AISC Eqn. F1-7) for I-shapes, channels and boxes and $F_y S_x$ (AISC Eqn. F1-11) for rectangular bars
M_{crx}	=	Critical elastic moment, $\frac{C_b \pi}{L_b}\sqrt{EI_yGJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w}$ (AISC Eqn. F1-13) for I-shapes and channels and $\frac{57000 C_b \sqrt{JA}}{L_b / r_y}$ (AISC Eqn. F1-14) for boxes and rectangular bars
L_b	=	Laterally unbraced length, l_y
L_p	=	Limiting laterally unbraced length for full plastic capacity, $\frac{300 r_y}{\sqrt{F_y}}$ (AISC Eqn.F1-4) for I-shapes and channels and $\frac{3750 r_y}{M_{px}}\sqrt{JA}$ (AISC Eqn. F1-5) for boxes and rectangular bars
L_r	=	Limiting laterally unbraced length

for inelastic lateral-torsional buckling

$$\frac{r_y X_1}{(F_y - F_r)}\sqrt{1 + \sqrt{1 + X_2(F_y - F_r)^2}}$$

(AISC Eqn. F1-6)

for I-shapes and channels

and $\dfrac{57000 r_y \sqrt{JA}}{M_{rx}}$ (AISC Eqn. F1-10)

for boxes and rectangular bars

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}$$ •(AISC Eqn.F1-8)

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

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

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 cantilevers, for beams spanning multiple bays and 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.

For compact pipes and circular bars bent about any axis,

$$M_{nx} = M_{ny} = M_p = ZF_y \qquad \text{(AISC F1.7)}$$

For all compact sections bent about their minor axis,

$$M_{ny} = M_{py} = Z_y F_y \qquad \text{(AISC F1.7)}$$

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,

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

for major direction bending, and

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

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
(see Figure C-6)

λ_p = Largest value of λ for which $M_n = M_p$
(see Figure C-6)

λ_r = Largest value of λ for which buckling
is inelastic (see Figure C-6)

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,

$$M_{nx} = \frac{C_b \pi \sqrt{E I_y G J}}{L_b}[B + \sqrt{1 + B^2}] \leq F_y S_x$$

(AISC Eqn. F1-15)

where

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

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,

$$M_{ny} = F_y S y$$

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

$$M_{nx} = F_y S x \quad \text{and} \quad M_{ny} = F_y S y$$

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

For $\frac{h}{t_w} \leq 187\sqrt{k/F_y}$

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

for $187\sqrt{k/F_y} < \frac{h}{t_w} \leq 234\sqrt{k/F_y}$

$$V_{nx} = 0.6F_yA_w\frac{187\sqrt{k/F_y}}{h/t_w} \qquad \text{(AISC Eqn. F2-2)}$$

and for $\frac{h}{t_w} > 234\sqrt{k/F_y}$

$$V_{nx} = A_w\frac{26400k}{(h/t_w)^2} \qquad \text{(AISC Eqn. F2-3)}$$

where k is assumed to be 5 (no stiffeners).

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

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

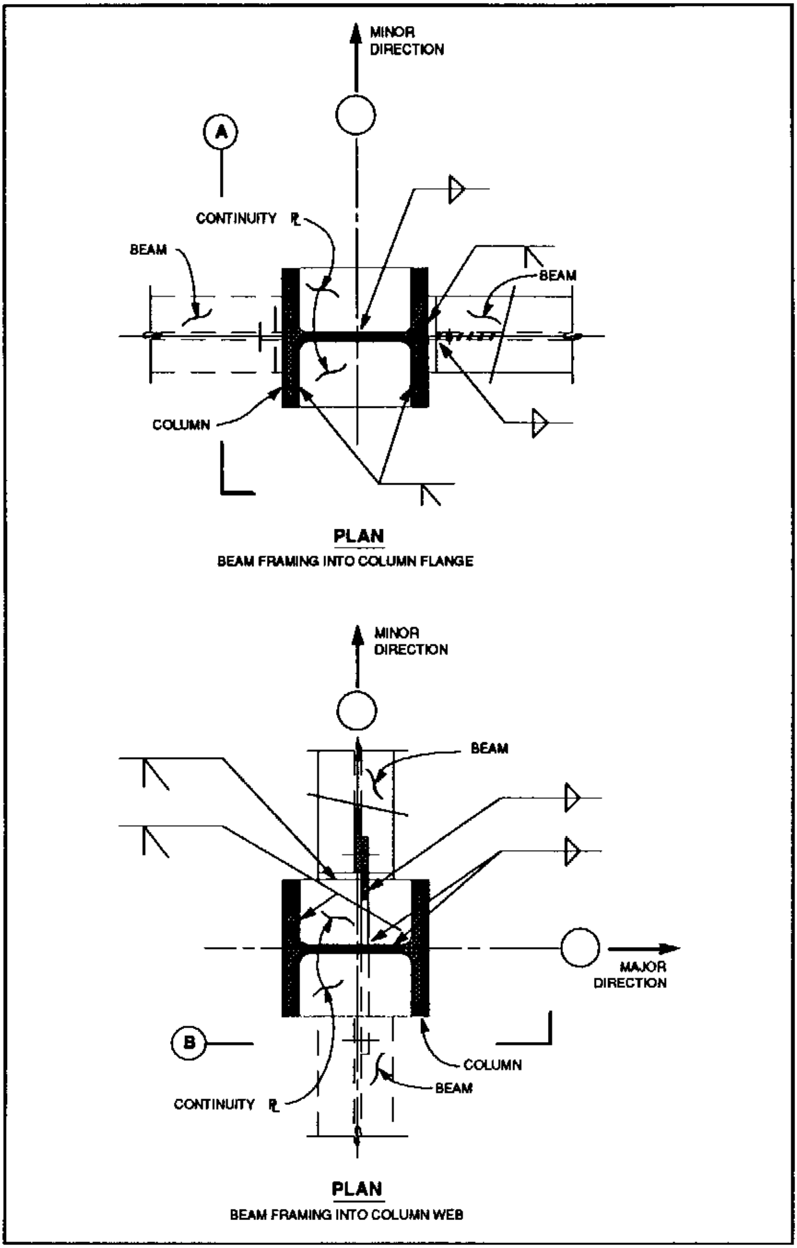
for $\frac{P_u}{\phi P_n} < 0.2$ the capacity ratio is given as

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

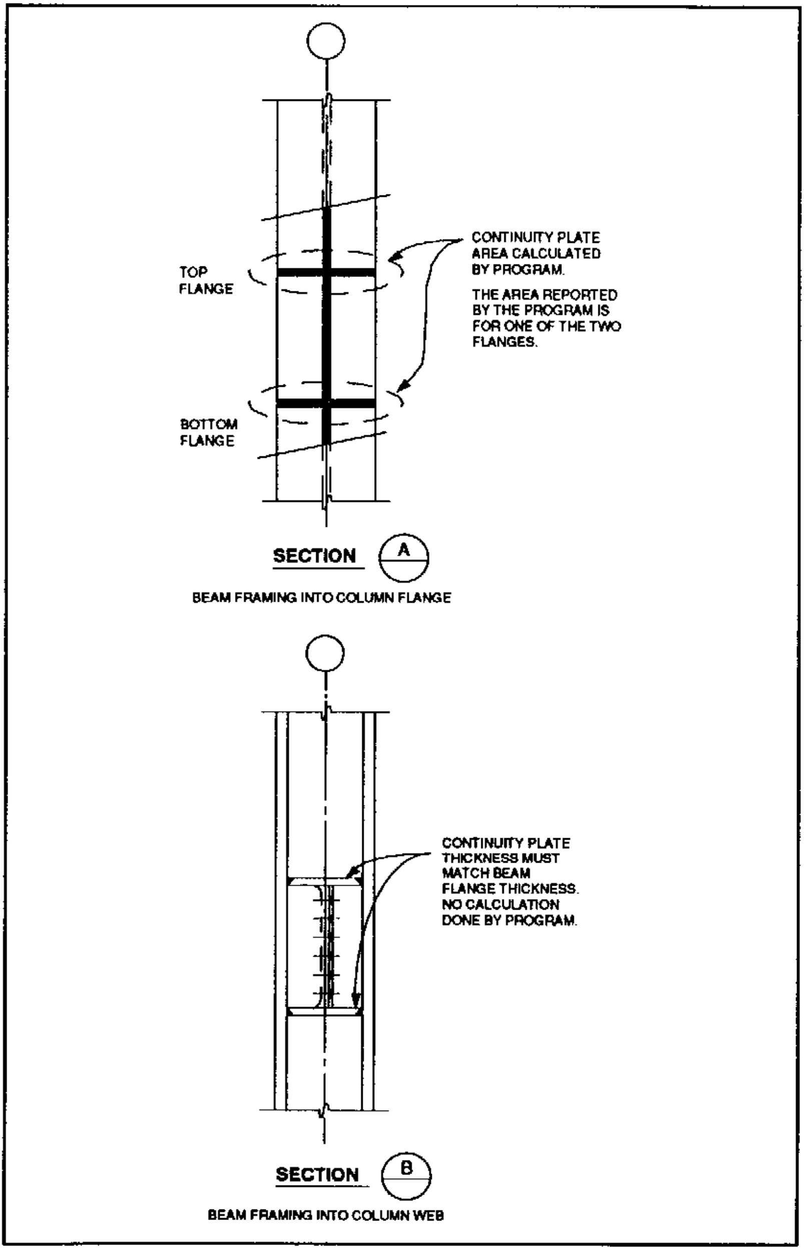
Where P_n is the nominal axial tensile strength and $\phi = \phi_t = 0.9$ if P_u is tensile; and P_n is the nominal axial compressive strength and $\phi = \phi_c = 0.85$ if P_u is compressive. The resistance factor for bending, $\phi_b = 0.9$.

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):



Plan Showing Continuity Plates
Figure C-8



Section Showing Continuity Plates
Figure C-9

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

Where $\phi_v = 0.9$.

2. Design of Continuity Plates

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 axis, i.e. the beam frames into the column flange.

b) The steel beam frames in a direction parallel to the column minor axis, 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 Figures C-8 and C-9 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 direction) for each of the loading combinations specified and reports the maximum continuity plate area that is needed.

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.,

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

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.,

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

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

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

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

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

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

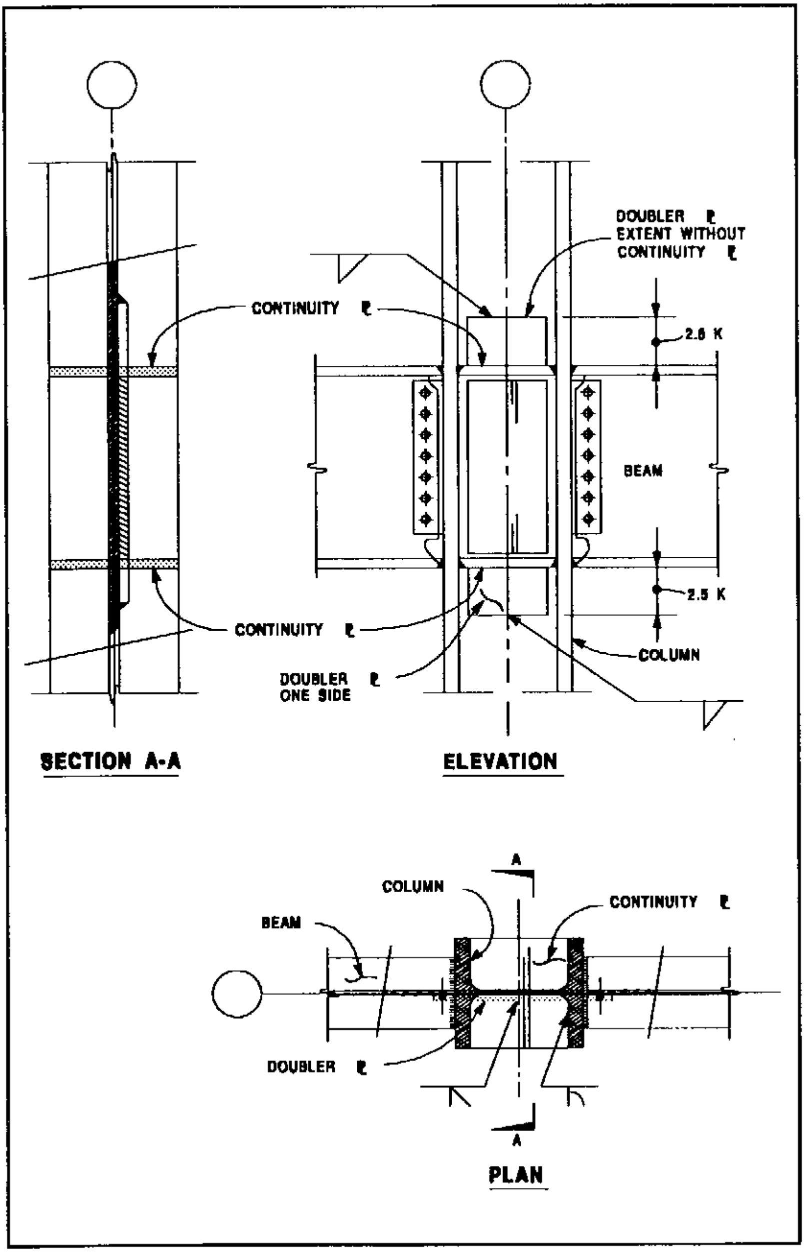
$$A_{cp} = \frac{P_{bf}}{(0.85)(0.9F_{yc})} - 12t_{wc}^2$$

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 F8). 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 Eqns. E2-2 and E2-3). 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 F8) the required minimum radius of gyration of the stiffener cross-section would be $r = 0.02h$ to obtain $\lambda = 0.5$ (AISC Eqn. 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.



Doubler Plates
Figure C-10

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

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 strengths seldom control the design of a beam or column member. However, in a moment resisting frame, the shear strength of 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. See Figure C-10. In minor direction bending, the joint shear is carried by the column flanges, in which case the shear strengths are seldom critical, and this condition is therefore not investigated by the program.

Shear forces 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. The program investigates such situations and reports the thicknesses of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements.

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

$$R_v = 0.7F_y d_c t_r \quad \text{for } P_u \leq 0.75P_n \text{ or if } P_u \text{ is tensile}$$
(AISC Eqn. K1-9)

and

$$R_v = 0.7F_y d_c t_r\,[1.9 - 1.2(P_u/P_n)] \quad \text{for } P_u > 0.75P_n$$
(AISC Eqn. 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_n	=	Column nominal axial compressive strength

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 Joint 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.

Appendix D.

CAPACITY CHECK AND DESIGN ALGORITHMS

(CISC 1989)

For the capacity check and design algorithms based on the Canadian Standards Association's Limit States Design of Steel Structures, CAN/CSA-S16.1-M89 [5] (CISC89), this appendix replaces in its entirety Chapter III which is based on the AISC-ASD89. This appendix describes in detail the various aspects of the capacity check procedures that are used by the program STEELER.

Equivalent chapters for the UBC91 [2], for the AISC Plastic 89 [3] and for the AISC-LRFD86 [4] specifications are presented in other appendices.

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 is assumed.

References to pertinent sections and equations of the CAN/CSA-S16.1-M89 are indicated with the CISC prefix.

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 postprocessing file brings across forces and moments associated with the eight independent load conditions (I, II, III, A, B...) 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.

If a structure is subjected to dead load (DL) and live load (LL) only, the capacity check may need the following two loading combinations (CISC 7.2):

1. 1.25 DL
2. 1.25 DL + 1.5 LL

However, if in addition to the dead load 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 fourteen load combinations may have to be defined :

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.

It is possible in the postprocessing 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 capacity ratios.

For the gravity load case the code requires (CISC 8.6.2) that the translational moments not be less than the moments produced by psuedo-lateral loads, applied at each story, equal to

0.005 times the factored gravity loads acting at each story. If extra load conditions are used for such analysis they should be included in the loading combinations with due consideration of the fact that the psuedo-lateral forces can be positive or negative.

When using the CISC89 code the program asumes 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 [8]. 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.

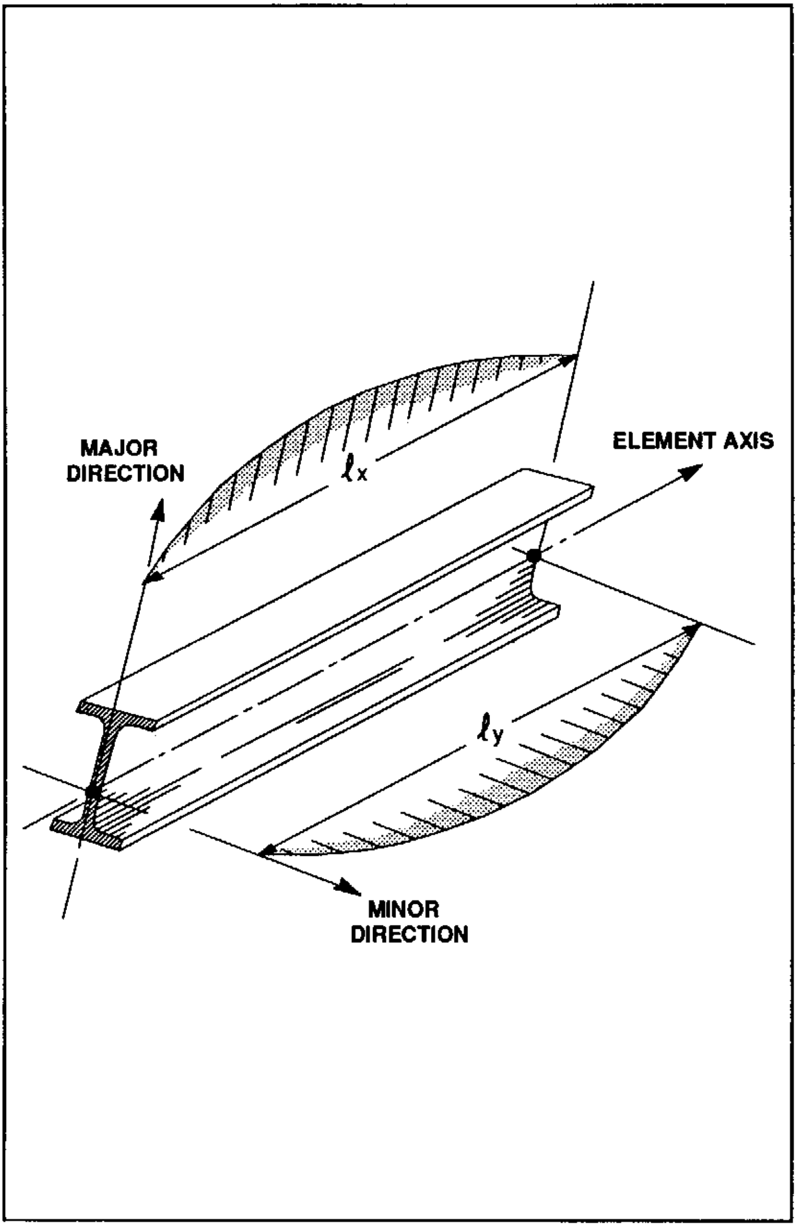
All tension and compression effects are consistently evaluated with these combinations.

The capacity ratios from the controlling load combinations are reported.

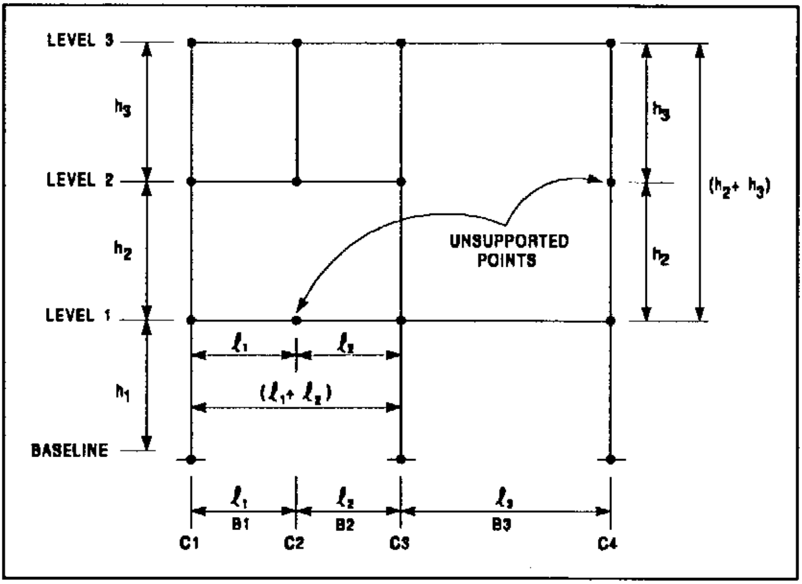
B. THE UNSUPPORTED LENGTHS OF THE ELEMENTS

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

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 D-1. These are the lengths between the support points of the element in the corresponding directions.



Element Unsupported Length
Figure D-1



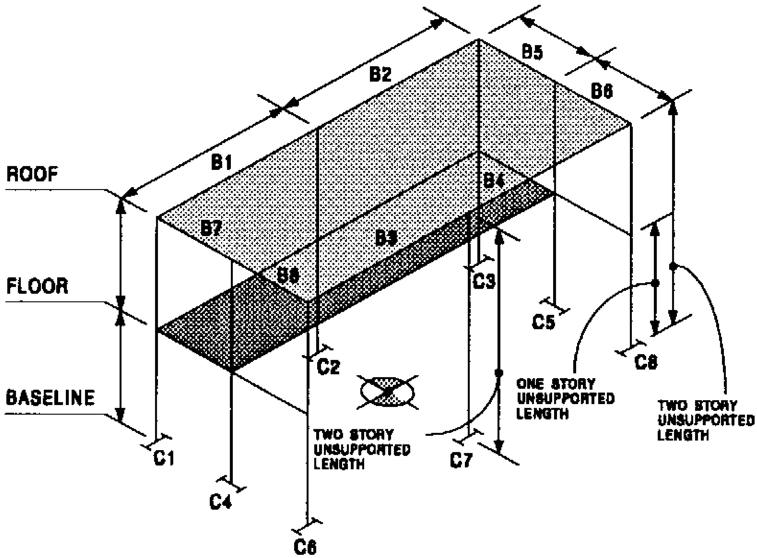
Tracing Unsupported Element Lengths
Figure D-2

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 D-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 lengths, such as



Conditions Affecting Unsupported Lengths
Figure D-3

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 D-3, column line C8 laterally spans two stories in the column major direction and only spans 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 D-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. THE EFFECTIVE LENGTH (K-) FACTORS

For this code the program assumes that a P-Delta analysis has been performed and, therefore, all effective length (K-) factors are conservatively assumed to be 1.0. These values may be overridden by user-specified values as part of the STEELER input data.

D. THE ELEMENT STATIONS AND FORCE COMPONENTS

The column, beam and brace elements are capacity 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 diaphragm at the level of the beam.

E. THE CAPACITY CHECK AND DESIGN PROCEDURE

This section defines the capactiy 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-M98 [5]. The various parameters referenced in this section are tabulated in Figure D-4.

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.

The program also evaluates the need for continuity and doubler plates in the columns at the beam to column joints and reports

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
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

Capacity Check Parameters
Figure D-4

V_{rx}, V_{ry}	=	Factored major and minor shear strengths, N
Z_x, Z_y	=	Major and minor plastic moduli, mm^3
b	=	Nominal dimension of longer leg of angles
	=	$b_f - 2t_w$ for welded BOX sections, mm
	=	$b_f - 3t_f$ for rolled BOX (TS) sections, mm
b_f	=	Flange width, mm
d	=	Overall depth of member, mm
h	=	Clear distance between flanges , taken as $d - 2t_f$, mm
k	=	Web plate buckling coefficient, assumed equal to 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

Capacity Check Parameters
Figure D-4 (continued)

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 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 [8]. 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
Mfg	=	Factored moments not causing translation (assumed due to gravity load conditions I, II, III)

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
Figure D-5

radio button can be selected for the group. As shown above, it is possible for both groups to have a button selected at the same time.

Mft = Factored moments causing sidesway (assumed due to lateral load conditions A, B, Dyn-1, Dyn-2, Dyn-3)

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 D-5 and reports the classifications for the individual members. If the section dimensions satisfy the limits shown in Figure D-5, 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)

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

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

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).

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).

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, $Z_x F_y$

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 & 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,

$$M_{rx} = \phi \frac{\omega 2\pi\sqrt{EI_yGJ}}{L}[B + \sqrt{1 + B^2}] \leq \phi\, F_yS_x$$

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,

$$M_{ry} = \phi\, F_yS_y$$

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

$$M_{rx} = \phi\, F_yS_x \quad \text{and} \quad M_{ry} = \phi\, F_yS_y$$

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):

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

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

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

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 U1 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:

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

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

and

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} - \frac{T_f\, Z_x}{M_{rx}\, A} \text{ for Class 1 \& 2 sections}$$

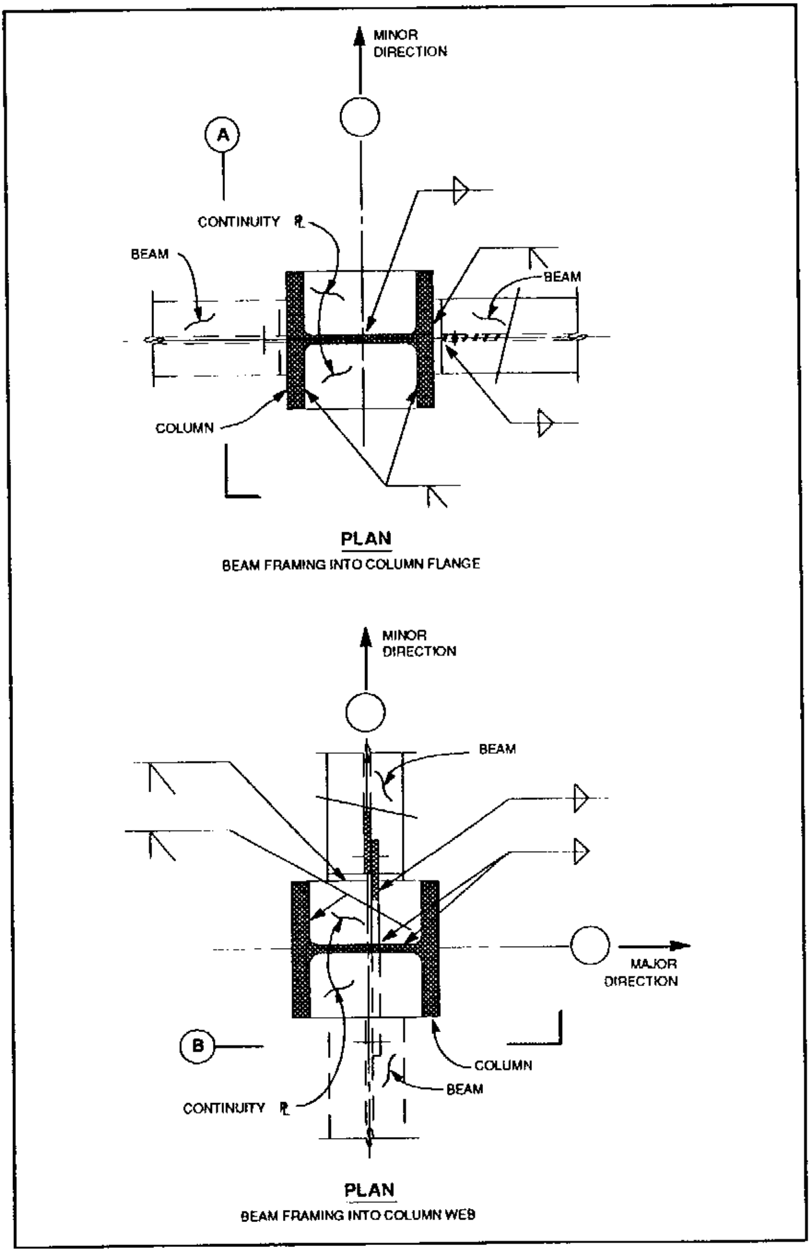
or

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

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:

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



Plan Showing Continuity Plates
Figure D-6

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 axis, i.e. the beam frames into the column flange.

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

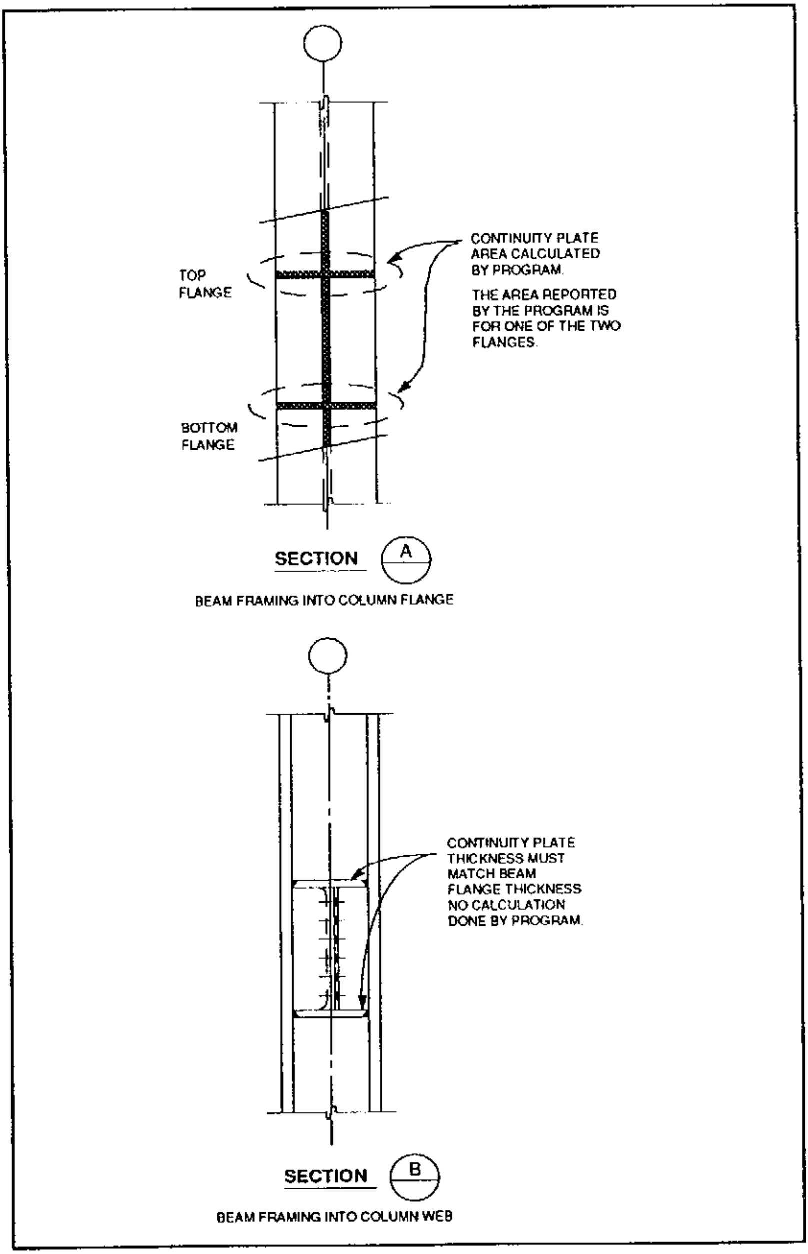
c) The steel beam frames in a direction that is at an angle to both of the principal axes of the column, i.e. the beam frames partially into the column web and partially into the column flange.

To achieve a beam/column moment connection, continuity plates such as shown in Figures D-6 and D-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 direc-



Section Showing Continuity Plates
Figure D-7

tion) for each of the loading combinations specified and reports the maximum continuity plate area that is needed.

The program first evaluates the need for continuity plates. Continuity plates will be required if **any** 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 any 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.

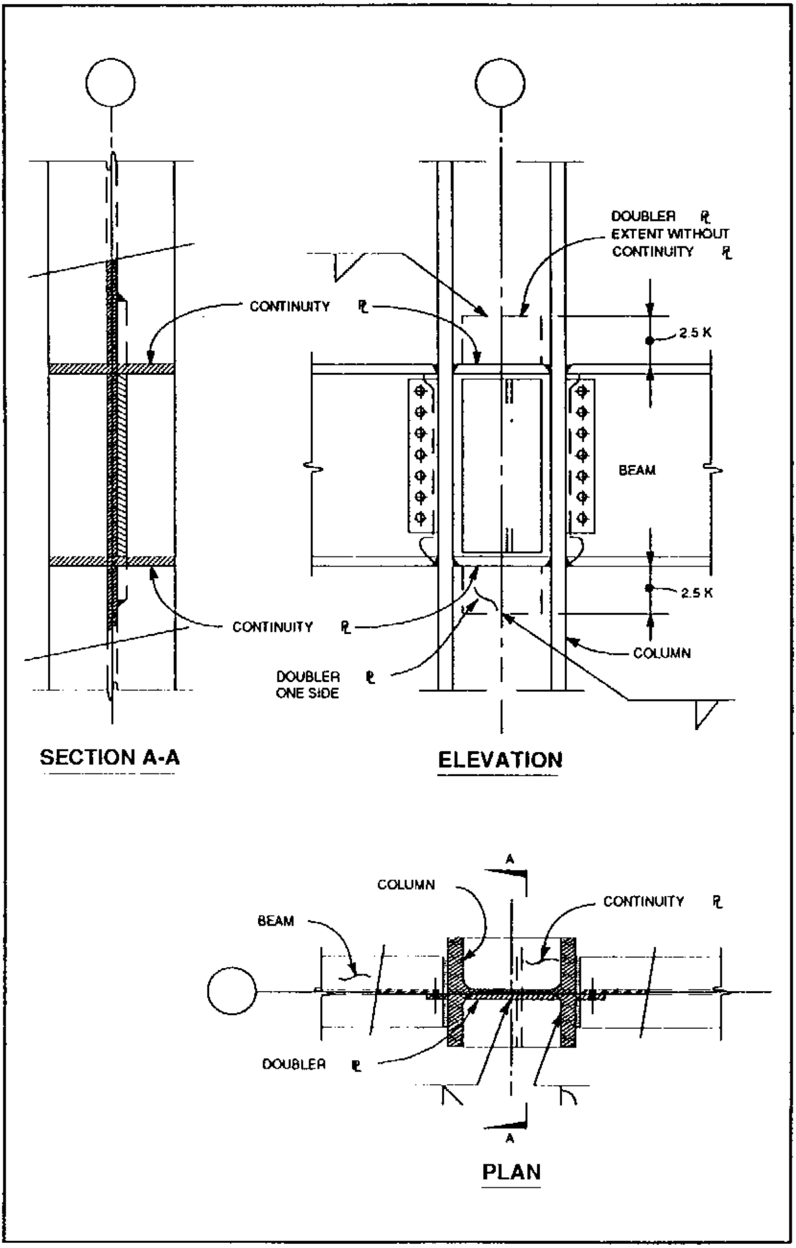
In the equations above,

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 between outer face of the column flange and web toe of its 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

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 strengths seldom control the design of a beam or column member. However, in a moment resisting frame, the shear strength of 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. See Figure D-8. In minor direction bending,



Doubler Plates
Figure D-8

the joint shear is carried by the column flanges, in which case the shear strengths are seldom critical, and this condition is therefore not investigated by the program.

Shear forces 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. The program investigates such situations and reports the thicknesses of any required doubler plates. Only columns with I-shapes are investigated for doubler plate requirements.

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 Joint 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.