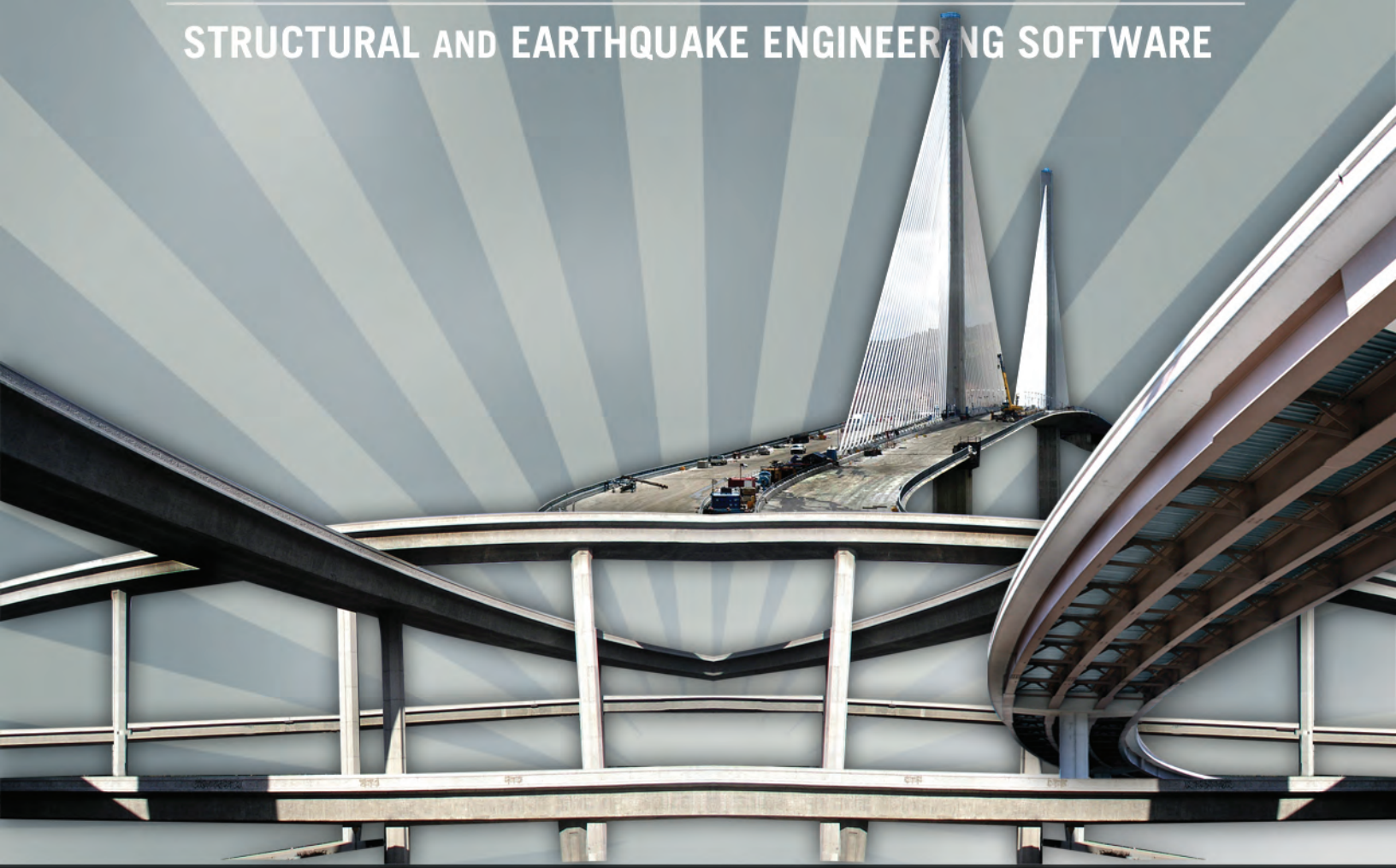# COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

CSiBRiDGE®

INTEGRATED 3-D BRIDGE DESIGN SOFTWARE

VERSION 15

Concrete Frame Design Manual

CSA A23.3-04

CSi

Concrete Frame
Design Manual

CSA A23.3-04

For CSiBridge™

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Computers and Structures, Inc.
1995 University Avenue
Berkeley, California 94704 USA

Phone: (510) 649-2200
FAX: (510) 649-2299
e-mail: info@csiberkeley.com (for general questions)
e-mail: support@csiberkeley.com (for technical support questions)
web: www.csiberkeley.com

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Chapter 1
Introduction

The design of concrete frames is seamlessly integrated within the program. Initiation of the design process, along with control of various design parameters, is accomplished using the **Design menu**.

It should be noted that two design processes are available in CSiBridge: superstructure design (on the Design/Rating tab) and design of the individual elements comprising the structure (the **Advanced > Frame Design** commands). This manual addresses the second design process.

Automated design at the object level is available for any one of a number of user-selected design codes, as long as the structures have first been modeled and analyzed by the program. Model and analysis data, such as material properties and member forces, are recovered directly from the model database, and no additional user input is required if the design defaults are acceptable.

The design is based on a set of user-specified loading combinations. However, the program provides default load combinations for each design code supported in the program. If the default load combinations are acceptable, no definition of additional load combinations is required.

In the design of columns, the program calculates the required longitudinal and shear reinforcement. However, the user may specify the longitudinal steel, in

which case a column capacity ratio is reported. The column capacity ratio gives an indication of the stress condition with respect to the capacity of the column.

The biaxial column capacity check is based on the generation of consistent three-dimensional interaction surfaces. It does not use any empirical formulations that extrapolate uniaxial interaction curves to approximate biaxial action.

Interaction surfaces are generated for user-specified column reinforcing configurations. The column configurations may be rectangular, square or circular, with similar reinforcing patterns. The calculation of moment magnification factors, unsupported lengths and strength reduction factors is automated in the algorithm.

Every beam member is designed for flexure and shear at output stations along the beam span.

All beam-column joints are investigated for existing shear conditions.

For Ductile and Moderately Ductile moment resisting frames, the shear design of the columns, beams and joints is based on the probable moment capacities of the members. Also, the program will produce ratios of the beam moment capacities with respect to the column moment capacities, to investigate weak beam/strong column aspects, including the effects of axial force.

Output data can be presented graphically on the model, in tables for both input and output data, or on the calculation sheet prepared for each member. For each presentation method, the output is in a format that allows the engineer to quickly study the stress conditions that exist in the structure and, in the event the member reinforcing is not adequate, aids the engineer in taking appropriate remedial measures, including altering the design member without rerunning the entire analysis.

1.1   Organization

This manual is designed to help you quickly become productive with the concrete frame design options of CSA A23.3-04. Chapter 2 provides detailed descriptions of the Deign Prerequisites used for CSA A23.3-04. Chapter 3 provides detailed descriptions of the code-specific process used for CSA

A23.3-04. Chapter 4 documents the design output produced by program. The appendices provide details on certain topics referenced in this manual.

1.2   Recommended Reading/Practice

It is strongly recommended that you read this manual and review any applicable "Watch & Learn" Series™ tutorials, which are found on our web site, http://www.csiberkeley.com, before attempting to design a concrete frame. Additional information can be found in the on-line Help facility available from within the program's main menu.

Chapter 2
Design Prerequisites

This chapter provides an overview of the basic assumptions, design preconditions, and some of the design parameters that affect the design of concrete frames.

In writing this manual it has been assumed that the user has an engineering background in the general area of structural reinforced concrete design and familiarity with CSA A23.3-04 codes.

2.1   Design Load Combinations

The design load combinations are used for determining the various combinations of the load cases for which the structure needs to be designed/checked. The load combination factors to be used vary with the selected design code. The load combination factors are applied to the forces and moments obtained from the associated load cases and are then summed to obtain the factored design forces and moments for the load combination.

For multi-valued load combinations involving response spectrum, time history, moving loads and multi-valued combinations (of type enveloping, square-root of the sum of the squares or absolute) where any correspondence between interacting quantities is lost, the program automatically produces multiple sub combinations using maxima/minima permutations of interacting quantities.

Separate combinations with negative factors for response spectrum cases are not required because the program automatically takes the minima to be the negative of the maxima for response spectrum cases and the above described permutations generate the required sub combinations.

When a design combination involves only a single multi-valued case of time history or moving load, further options are available. The program has an option to request that time history combinations produce sub combinations for each time step of the time history. Also an option is available to request that moving load combinations produce sub combinations using maxima and minima of each design quantity but with corresponding values of interacting quantities.

For normal loading conditions involving static dead load, live load, wind load, and earthquake load, or dynamic response spectrum earthquake load, the program has built-in default loading combinations for each design code. These are based on the code recommendations and are documented for each code in the corresponding manuals.

For other loading conditions involving moving load, time history, pattern live loads, separate consideration of roof live load, snow load, and so on, the user must define design loading combinations either in lieu of or in addition to the default design loading combinations.

The default load combinations assume all load cases declared as dead load to be additive. Similarly, all cases declared as live load are assumed additive. However, each load case declared as wind or earthquake, or response spectrum cases, is assumed to be non additive with each other and produces multiple lateral load combinations. Also wind and static earthquake cases produce separate loading combinations with the sense (positive or negative) reversed. If these conditions are not correct, the user must provide the appropriate design combinations.

The default load combinations are included in design if the user requests them to be included or if no other user-defined combination is available for concrete design. If any default combination is included in design, all default combinations will automatically be updated by the program any time the design code is changed or if static or response spectrum load cases are modified.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.

The user is cautioned that if moving load or time history results are not requested to be recovered in the analysis for some or all of the frame members, the effects of those loads will be assumed to be zero in any combination that includes them.

2.2 Design and Check Stations

For each load combination, each element is designed or checked at a number of locations along the length of the element. The locations are based on equally spaced segments along the clear length of the element. The number of segments in an element is requested by the user before the analysis is made. The user can refine the design along the length of an element by requesting more segments.

When using the CSA A23.3-04 design code, requirements for joint design at the beam to column connections are evaluated at the top most station of each column. The program also performs a joint shear analysis at the same station to determine if special considerations are required in any of the joint panel zones. The ratio of the beam flexural capacities with respect to the column flexural capacities considering axial force effect associated with the weak-beam/strong-column aspect of any beam/column intersection are reported.

2.3 Identifying Beams and Columns

In the program, all beams and columns are represented as frame elements. But design of beams and columns requires separate treatment. Identification for a concrete element is accomplished by specifying the frame section assigned to the element to be of type beam or column. If any brace element exists in the frame, the brace element also would be identified as a beam or a column element, depending on the section assigned to the brace element.

2.4   Design of Beams

In the design of concrete beams, in general, the program calculates and reports the required areas of steel for flexure and shear based on the beam moments, shears, load combination factors, and other criteria, which are described in detail in the code-specific chapters. The reinforcement requirements are calculated at a user-defined number of stations along the beam span.

All the beams are only designed for major direction flexure and shear. Effects due to any axial forces, torsion and minor direction bending that may exist in the beams must be investigated independently by the user.

In designing the flexural reinforcement for the major moment at a particular section of a particular beam, the steps involve the determination of the maximum factored moments and the determination of the reinforcing steel. The beam section is designed for the maximum positive and maximum negative factored moment envelopes obtained from all of the load combinations. Negative beam moments produce top steel. In such cases, the beam is always designed as a Rectangular section. Positive beam moments produce bottom steel. In such cases, the beam may be designed as a Rectangular beam or a T beam. For the design of flexural reinforcement, the beam is first designed as a singly reinforced beam. If the beam section is not adequate, the required compression reinforcement is calculated.

In designing the shear reinforcement for a particular beam for a particular set of loading combinations at a particular station due to the beam major shear, the steps involve the determination of the factored shear force, the determination of the shear force that can be resisted by concrete, and the determination of the reinforcement steel required to carry the balance.

Special considerations for seismic design are incorporated into the program for CSA A23.3-04.

2.5   Design of Columns

In the design of the columns, the program calculates the required longitudinal steel, or if the longitudinal steel is specified, the column stress condition is reported in terms of a column capacity ratio, which is a factor that gives an indication of the stress condition of the column with respect to the capacity of the

column. The design procedure for the reinforced concrete columns of the structure involves the following steps:

- Generate axial force-biaxial moment interaction surfaces for all of the different concrete section types of the model.

- Check the capacity of each column for the factored axial force and bending moments obtained from each loading combination at each end of the column. This step is also used to calculate the required reinforcement (if none was specified) that will produce a capacity ratio of 1.0.

The generation of the interaction surface is based on the assumed strain and stress distributions and some other simplifying assumptions. These stress and strain distributions and the assumptions are documented in Chapter 3.

The shear reinforcement design procedure for columns is very similar to that for beams, except that the effect of the axial force on the concrete shear capacity must be considered.

For certain special seismic cases, the design of columns for shear is based on the capacity shear. The capacity shear force in a particular direction is calculated from the moment capacities of the column associated with the factored axial force acting on the column. For each load combination, the factored axial load is calculated, using the load cases and the corresponding load combination factors. Then, the moment capacity of the column in a particular direction under the influence of the axial force is calculated, using the uniaxial interaction diagram in the corresponding direction, as documented in Chapter 3.

2.6 Design of Joints

To ensure that the beam-column joint of special moment resisting frames possesses adequate shear strength, the program performs a rational analysis of the beam-column panel zone to determine the shear forces that are generated in the joint. The program then checks this against design shear strength.

Only joints having a column below the joint are designed. The material properties of the joint are assumed to be the same as those of the column below the joint. The joint analysis is done in the major and the minor directions of the column. The joint design procedure involves the following steps:

- ▪ Determine the panel zone design shear force,

- ▪ Determine the effective area of the joint, and

- ▪ Check panel zone shear stress.

The joint design details are documented in Chapter 3.

2.7 P-Delta Effects

The program design process requires that the analysis results include P-delta effects. The P-delta effects are considered differently for "braced" or "non-sway" and "unbraced" or "sway" components of moments in columns or frames. For the braced moments in columns, the effect of P-delta is limited to "individual member stability." For unbraced components, "lateral drift effects" should be considered in addition to individual member stability effect. The program assumes that "braced" or "nonsway" moments are contributed from the "dead" or "live" loads. Whereas, "unbraced" or "sway" moments are contributed from all other types of loads.

For the individual member stability effects, the moments are magnified with moment magnification factors, as documented in Chapter 3 of this manual.

For lateral drift effects, the program assumes that the P-delta analysis is performed and that the amplification is already included in the results. The moments and forces obtained from P-delta analysis are further amplified for individual column stability effect if required by the governing code, as in the CSA A23.3-04 codes.

Users should be aware that the default analysis option in the program is turned OFF for P-delta effect. The user can turn the P-delta analysis ON and set the maximum number of iterations for the analysis. The default number of iteration for P-delta analysis is 1. Further details on P-delta analysis are provided in Appendix A of this design manual.

2.8 Element Unsupported Lengths

To account for column slenderness effects, the column unsupported lengths are required. The two unsupported lengths are l_{33} and l_{22}. These are the lengths be-

tween support points of the element in the corresponding directions. The length l_{33} corresponds to instability about the 3-3 axis (major axis), and l_{22} corresponds to instability about the 2-2 axis (minor axis).

Normally, the unsupported element length is equal to the length of the element, i.e., the distance between END-I and END-J of the element. The program, however, allows users to assign several elements to be treated as a single member for design. This can be done differently for major and minor bending as documented in Appendix B of this design manual.

The user has options to specify the unsupported lengths of the elements on an element-by-element basis.

2.9   Choice of Input Units

English as well as SI and MKS metric units can be used for input. But the codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. For example, the CSA A23.3-04 code is published in Millimeter-Newton-Second units. By default, all equations and descriptions presented in the chapter "Design for CSA A23.3-04" correspond to Millimeter-Newton-Second units. However, any system of units can be used to define and design the structure in the program.

It should be noted that two design processes are available in CSiBridge: superstructure design (on the Design/Rating tab) and design of the individual elements comprising the structure (the **Advanced > Frame Design** commands). This manual addresses the second design process.

Chapter 3
Design Process

This chapter provides a detailed description of the code-specific algorithms the program uses to design concrete frames when the Canadian code CSAA23.3-04 (CSA 2004) is selected. For simplicity, all equations and descriptions presented in this chapter correspond to Newton-Millimeter-Second units unless otherwise noted.

The program provides options to design or check Conventional, Moderately Ductile (moderate seismic risk areas), and Ductile (high seismic risk areas) moment resisting frames as required for seismic design. The details of the design criteria used for the different framing systems are described in this chapter.

3.1 Notation

The various notations used in this chapter are described herein:

A_{cv} Area of concrete used to determine shear stress, sq-mm

A_{core} Area of concrete core, sq-mm

A_g Gross area of concrete, sq-mm

A_s Area of tension reinforcement, sq-mm

A'_s	Area of compression reinforcement, sq-mm
$A_{s(required)}$	Area of steel required for tension reinforcement, sq-mm
A_{st}	Total area of column longitudinal reinforcement, sq-mm
A_v	Area of shear reinforcement, sq-mm
a	Depth of compression block, mm
a_b	Depth of compression block at balanced condition, mm
b	Width of member, mm
b_f	Effective width of flange (T beam section), mm
b_w	Width of web (T beam section), mm
C_m	Coefficient, dependent upon column curvature, used to calculate moment magnification factor
c	Depth to neutral axis, mm
c_b	Depth to neutral axis at balanced conditions, mm
d	Distance from compression face to tension reinforcement, mm
d'	Concrete cover to center of reinforcing, mm
d_s	Thickness of slab (T beam section), mm
E_c	Modulus of elasticity of concrete, MPa
E_s	Modulus of elasticity of reinforcement, assumed as 200,000 MPa
f'_c	Specified compressive strength of concrete, MPa
f_y	Specified yield strength of flexural reinforcement, MPa
f_{yh}	Specified yield strength of shear reinforcement, MPa
h	Dimension of beam or column, mm
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4
I_{st}	Moment of inertia of reinforcement about centroidal axis of member cross-section, mm^4
k	Effective length factor

L	Clear unsupported length, mm
M_1	Smaller factored end moment in a column, N-mm
M_2	Larger factored end moment in a column, N-mm
M_c	Factored moment to be used in design, N-mm
M_{ns}	Non-sway component of factored end moment, N-mm
M_s	Sway component of factored end moment, N-mm
M_f	Factored moment at section, N-mm
M_{fx}	Factored moment at section about X-axis, N-mm
M_{fy}	Factored moment at section about Y-axis, N-mm
N_f	Factored axial load at section (tension positive), N
P_b	Axial load capacity at balanced strain conditions, N
P_c	Critical buckling strength of column, N
$P_{r,max}$	Maximum axial load strength allowed, N
P_o	Axial load capacity at zero eccentricity, N
P_f	Factored axial load at section (compression positive), N
V_c	Shear resisted by concrete, N
V_{D+L}	Shear force from span loading, N
V_p	Shear force computed from probable moment capacity, N
V_f	Factored shear force at a section, N
V_s	Shear force at a section resisted by steel, N
α	Reinforcing steel overstrength factor
β	Factor accounting for shear resistance of cracked concrete
θ	Angle of inclination of diagonal compressive stresses with the longitudinal axis of beam or column
β_1	Factor for obtaining depth of compression block in concrete
α_1	Factor for obtaining average compressive stress in concrete block

β_d	Absolute value of the ratio of the maximum factored axial dead load moment to the maximum factored total load moment
δ_b	Moment magnification factor for non-sway moments
δ_s	Moment magnification factor for sway moments
ε_c	Strain in concrete
ε_s	Strain in reinforcing steel
φ_c	Strength reduction factor for concrete
φ_s	Strength reduction factor for steel
φ_m	Strength reduction factor for member
λ	Shear strength factor

3.2 Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure is to be checked. The program creates a number of default design load combinations for a concrete frame design. Users can add their own design load combinations as well as modify or delete the program default design load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. The scale factors are applied to the forces and moments from the load cases to form the factored design forces and moments for each design load combination. There is one exception to the preceding. For spectral analysis modal combinations, any correspondence between the signs of the moments and axial loads is lost. The program uses eight design load combinations for each such loading combination specified, reversing the sign of axial loads and moments in major and minor directions.

As an example, if a structure is subjected to dead load, DL, and live load, LL, only, the CSA A23.3-04 design check may need only one design load combination, namely, 1.25 DL +1.5 LL. However, if the structure is subjected to wind, earthquake or other loads, numerous additional design load combinations may be required.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

The design load combinations are the various combinations of the prescribed load cases for which the structure is to be checked. For this code, if a structure is subjected to dead load (DL), live load (LL), wind (WL), and earthquake (EL) loads, and considering that wind and earthquake forces are reversible, the following load combinations should be considered (CSA 4.1.3.2):

1.4 DL	(CSA 8.3.2, Table C.1, Case 1)
1.25 DL + 1.50 LL	(CSA 8.3.2, Table C.1, Case 2)
1.25 DL 1.40 WL	(CSA 8.3.2, Table C.1, Case 4)
0.90 DL 1.40 WL	(CSA 8.3.2, Table C.1, Case 4)
1.25 DL + 0.5 LL 1.40 WL	(CSA 8.3.2, Table C.1, Case 4)
1.00 DL 1.00 EL	(CSA 8.3.2, Table C.1, Case 5)
1.00 DL + 0.50 LL 1.00 EL	(CSA 8.3.2, Table C.1, Case 5)

These are also the default design load combinations in the program whenever the CSA A23.3-04 code is used. In generating the preceding default loading combinations, the importance factor is taken as 1. The user should use other appropriate design load combinations if roof live load is separately treated, or if other types of loads are present. PLL is the live load multiplied by the Pattern Live Load Factor. The Pattern Live Load Factor can be specified in the Preferences.

When using the CSA A23.3-04 code, the program design assumes that a P-delta analysis has been performed.

3.3 Limits on Material Strength

The upper and lower limits of f_c' should be 80 MPa and 20 MPa respectively, for all framing types (CSA 8.6.1.1, 21.2.6.1).

$$f_c' \leq 80 \, \text{MPa} \hspace{3cm} \text{(CSA 8.6.1.1, 21.2.6.1)}$$

$$f_c' \geq 20\,\text{MPa} \hspace{4cm} \text{(CSA 8.6.1.1)}$$

The upper limit of f_y should be 500 MPa for all frames (CSA 8.5.1, 21.2.7.1)

When the compression strength of concrete used in design is beyond the given limits or when the yield strength of steel used in design exceeds the given limits, the code does not cover such cases. The code allows use of f_c' and f_y beyond the given limits, provided special care is taken regarding the detailing and ductility (CSA 8.6.1.2, 21.2.7.1).

The program does not enforce any of these limits for column P-M-M interaction check or design and flexure design of beam. The specified strengths are used for design. The user is responsible for using the proper strength values while defining the materials. For shear design, special limits are applicable (CSA 11.3.4, 11.3.6.4, 21.2.6.3) and enforced in the program as described herein.

The code allows the use of reinforcement with f_y less than 400 MPa in members resisting earthquake induced forces without any restriction. The program also allows the use of f_y greater than 400 MPa. However, if f_y is between 400 MPa and 500 MPa, the code requires that increased strain be taken into account because higher-yield-strength steel, in general, reduces ductility (CSA 21.2.7.1). The program also considers the increased strain through strain-compatibility relations during P-M-M interaction capacity calculations.

3.4 Strength Resistance Factors

The strength resistance factor, φ, is material dependent and is defined as

$$\varphi_c = 0.65 \text{ for concrete and} \hspace{3cm} \text{(CSA 8.4.2)}$$

$$\varphi_s = 0.85 \text{ for steel.} \hspace{3.5cm} \text{(CSA 8.4.3a)}$$

In some special cases, a member resistance factor, φ_m, is used as an additional reduction factor in addition to φ_c and φ_s (CSA 8.4.1). In connection with buckling resistance, φ_m is taken as 0.75 (CSA 10.15.3.1). The values of φ_c and φ_s given herein are the default values. These values can be modified in the Pref-

erences. For structural concrete manufactured in pre-qualified manufacturing plants, φ_c can be taken as 0.7 (CSA 8.4.2, 16.1.3).

3.5   Column Design

The user may define the geometry of the reinforcing bar configuration of each concrete column section. If the area of reinforcing is provided by the user, the program checks the column capacity. However, if the area of reinforcing is not provided by the user, the program calculates the amount of reinforcing required for the column. The design procedure for the reinforced concrete columns of the structure involves the following steps:

▪ Generate axial force/biaxial moment interaction surfaces for all of the different concrete section types of the model. A typical biaxial interaction surface is shown in Figure 3-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement — 1 to 8 percent for Conventional, Moderately Ductile resisting frames (CSA 10.9.1 and CSA 10.9.2) and 1 to 6 percent for Ductile moment resisting frames (CSA 21.4.3.1).

▪ Calculate the capacity ratio or the required reinforcing area for the factored axial force and biaxial (or uniaxial) bending moments obtained from each loading combination at each station of the column. The target capacity ratio is taken as one when calculating the required reinforcing area.

▪ Design the column shear reinforcement.

The following three subsections describe in detail the algorithms associated with this process.

3.5.1   Generation of Biaxial Interaction Surfaces

The column capacity interaction volume is numerically described by a series of discrete points that are generated on the three-dimensional interaction failure surface. In addition to axial compression and biaxial bending, the formulation allows for axial tension and biaxial bending considerations. A typical interaction surface is shown in Figure 3-1.
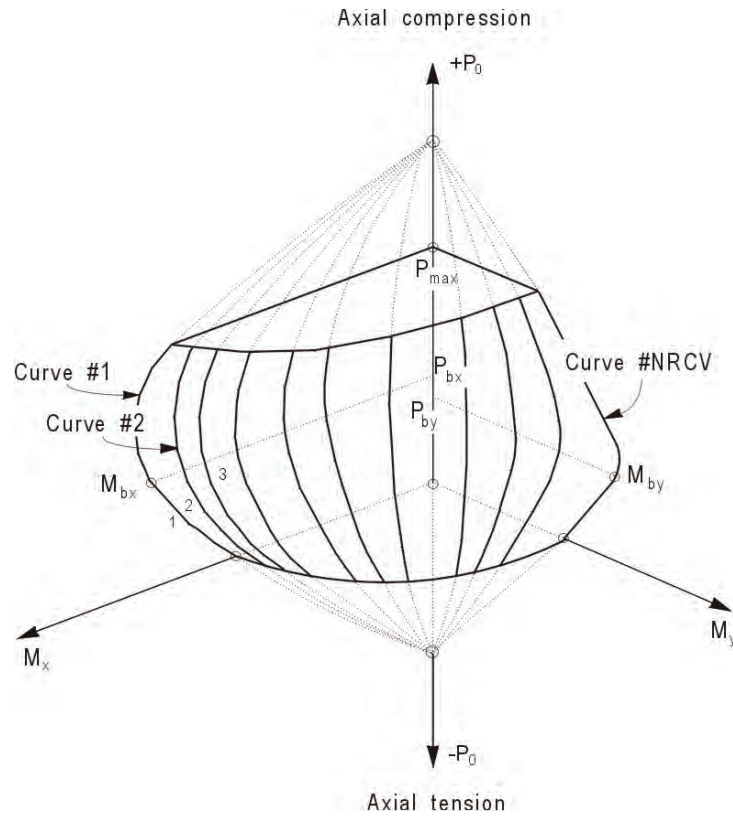
Figure 3-1 A typical column interaction surface

The coordinates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column. See Figure 3-2. The linear strain diagram limits the maximum concrete strain, ε_c, at the extremity of the section to 0.0035 (CSA 10.1.3).

The formulation is based consistently on the general principles of ultimate strength design (CSA 10.1), and allows for any doubly symmetric rectangular, square, or circular column section.
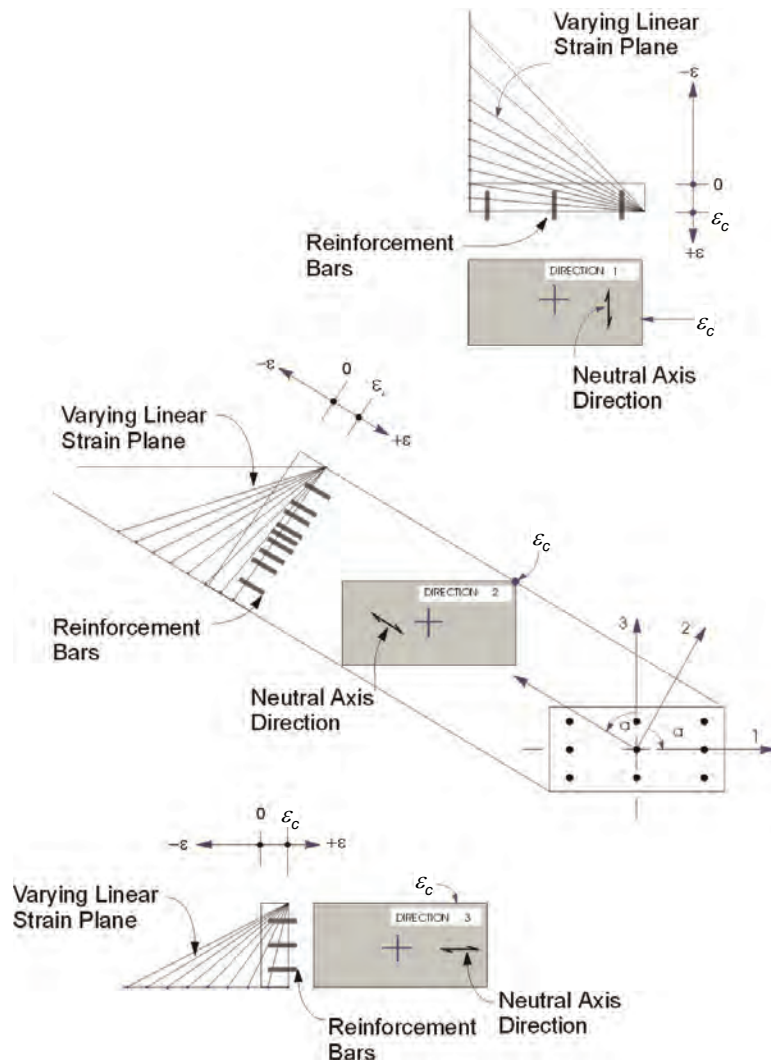
Figure 3-2 Idealized strain distribution for generation of interaction surface

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\varepsilon_s E_s$, and is limited to the yield stress of the steel, f_y (CSA 8.5.3.2). The area associated with each reinforcing bar is assumed to be placed at the actual location of the center of the bar, and the algorithm does not assume any further simplifications in the manner in which the area of steel is distributed over the cross-section of the column (such as an equivalent steel tube or cylinder), as shown in Figure 3-3.

(i) Concrete Section   *(ii) Strain Diagram*   *(iii) Stress Diagram*
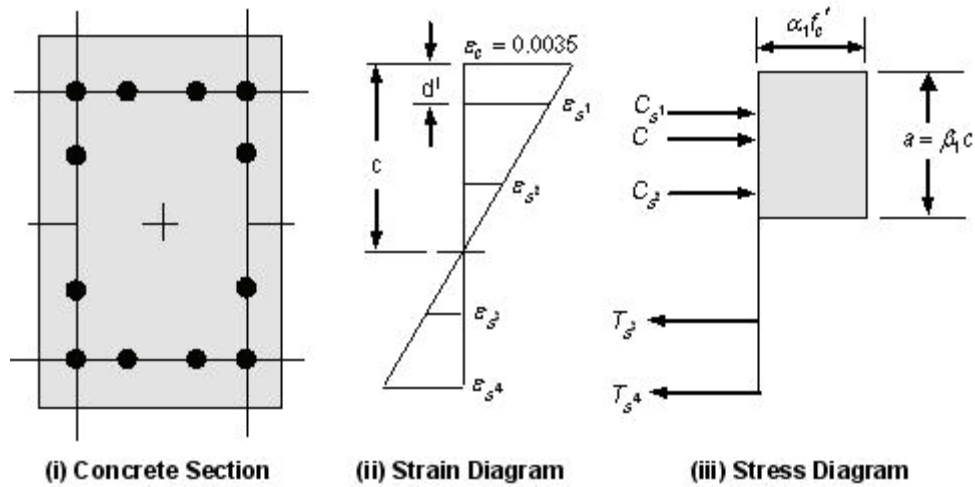
Figure 3-3 Idealization of stress and strain distribution in a column section

The concrete compression stress block is assumed to be rectangular, with a stress value of $\alpha_1 f_c'$ (CSA 10.1.7). See Figure 3-3. The depth of the stress block is $\beta_1 c$, where

$$\alpha_1 = 0.85 - 0.0015 f_c', \qquad \alpha_1 \geq 0.67 \qquad \text{(CSA 10.1.7)}$$

$$\beta_1 = 0.97 - 0.0025 f_c', \qquad \beta_1 \geq 0.67. \qquad \text{(CSA 10.1.7)}$$

The interaction algorithm provides a correction to account for the concrete area that is displaced by the reinforcement in the compression zone.

The effects of the strength reduction factors are included in the generation of the interaction surfaces. The maximum compressive axial load is limited to $P_{r,\max}$, where the maximum factored axial load resistance is given by

$$P_{r,\max} = 0.80 \left[\varphi_c \alpha_1 f_c' \left(A_g - A_{st} \right) + \varphi_s f_y A_{st} \right] \text{ (tied column)} \qquad \text{(CSA 10.10.4)}$$

$$P_{r,\max} = 0.80 \left[\varphi_c \alpha_1 f_c' \left(A_g - A_{st} \right) + \varphi_s f_y A_{st} \right] \text{ (spiral column).} \qquad \text{(CSA 10.10.4)}$$

3.5.2   Calculate Column Capacity Ratio

The column capacity is checked for each loading combination at each check station of each column. In checking a particular column for a particular loading combination at a particular location, the program uses the following steps:

- Determine the factored moments and forces from the load cases and the specified load combination factors to give P_f, M_{fx}, and M_{fy}.

- Determine the moment magnification factors for the column moments.

- Apply the moment magnification factors to the factored loads obtained in the first step. Determine whether the point, defined by the resulting axial load and biaxial moment set, lies within the interaction volume.

The following three sections describe in detail the algorithms associated with this process.

3.5.2.1   Determine Factored Moments and Forces

The factored loads for a particular load combination are obtained by applying the corresponding load factors to all the load conditions, giving P_f, M_{fx}, and M_{fy}. The computed moments are further amplified by using "Moment Magnification Factors" to allow for "Lateral Drift Effect" and "Member Stability Effect."

3.5.2.2   Determine Moment Magnification Factors

The moment magnification factors are applied in two stages. First the moments are separated into their "sway" and "non-sway" components. The sway components are amplified for lateral drift effect (CSA 10.16). Although according to the code, this amplification is significant for "unbraced" frames only and may be avoided for "braced" frames, the program treats all frames uniformly to amplify non-sway components of moments. These amplified moments are further amplified for individual member stability effect. According to the code, the individual member stability effect is only significant if:

$$\frac{kL}{r} > \frac{25 - \dfrac{10M_1}{M_2}}{\sqrt{P_f \Big/ f_c' A_g}}, \qquad \text{for non-sway frames, and} \qquad \text{(CSA 10.15.2)}$$

$$\frac{kL}{r} > \frac{35}{\sqrt{P_f \Big/ f_c' A_g}}, \qquad \text{for sway frames.} \qquad \text{(CSA 10.16.4)}$$

However, the program considers individual member stability effect for all compression column elements.

3.5.2.2.1 Lateral Drift Effect

For all frames, the moment magnification factor for lateral drift effect is applied only to the "sway" moment in the program.

$$M = M_{ns} + \delta_s M_s \qquad\qquad \text{(CSA 10.16.2)}$$

The moment obtained from analysis is separated into two components: the sway (M_s) and the non-sway (M_{ns}) components. The non-sway or braced components, which are identified by "*ns*" subscripts are predominantly caused by gravity load. The sway components are identified by "*s*" subscripts. The sway moments are predominantly caused by lateral loads and are related to the cause of sidesway.

The moment magnification factors in the major and minor directions can, in general, be different. The moment magnification factors for moments causing sidesway, δ_{sx} and δ_{sy}, can be taken as 1.0 if a P-delta analysis is carried out (CSA 10.16.3.1). The program assumes that a P-delta analysis has been performed and, therefore, moment magnification factors for moments causing sidesway are taken as unity. For the P-delta analysis the load should correspond to a load combination of (1.25 DL + 1.40 LL)/ φ_m (CSA 10.16.5), where φ_m is the strength reduction factor for stability and is equal to 0.75 (CSA 10.15.3). See also White and Hajjar (1991).

The user is reminded of the special analysis requirements, especially those related to the value of *EI* used in analysis (CSA 10.14.1). The program analysis uses the *EI* of the gross cross-sectional area in conjunction with a multiplication factor. The multiplication factors are defined on a section-by-section basis, so that the effect of axial force and cracking can be modeled in a first order analysis.

If the program assumptions are not satisfactory for a particular member, the user can explicitly specify values of δ_{sx} and δ_{sy}.

3.5.2.2.2 Member Stability Effects

All compression members are designed using the factored axial load, P_f, from the analysis and a magnified factored moment, M_c. The magnified moment is computed as,

$$M_c = \delta_b M_2 \geq M_2, \tag{CSA 10.15.3.1}$$

where M_2 is the column maximum end moment obtained from elastic analysis after considering minimum eccentricity and lateral drift effect, and M_c is the maximum moment occurring at the end or at an interior point within the span of the column. The moment magnification factor, δ_b, for moments not causing sidesway associated with the major or minor direction of the column is given by

$$\delta_b = \frac{C_m}{1 - \dfrac{P_f}{\varphi_m P_c}} \geq 1.0, \text{ where} \tag{CSA 10.15.3.1}$$

$$\varphi_m = 0.75, \tag{CSA 10.15.3.1}$$

$$P_c = \frac{\pi^2 EI}{(kL)^2}, \tag{CSA 10.15.3.1}$$

k is conservatively taken as 1, however the user can overwrite the value,

EI is associated with a particular column direction given by

$$EI = \frac{0.4 E_c I_g}{1 + \beta_d}, \qquad \text{and} \qquad \text{(CSA 10.15.3.1)}$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4, \qquad \text{(CSA 10.15.3.2)}$$

M_a and M_b are the moments at the ends of the column, and M_b is numerically larger than M_a. M_a/M_b is positive for single curvature bending and negative for double curvature bending. The preceding expression of C_m is valid if there is no transverse load applied between the supports. If transverse load is present on the span, or the length is overwritten, or for any other case, $C_m = 1$ (CSA 10.15.3.3). C_m can be overwritten by the user on an element- by-element basis.

The pre-magnified factored moments are increased, if required, to obtain minimum eccentricities such that M/P_f is at least $(15 + 0.03h)$ mm about each axis separately, where h is the dimension of the column in mm units in the corresponding direction (CSA 10.15.3.1).

$$M \geq P_f (15 + 0.03h) \qquad \text{(CSA 10.15.3.1)}$$

The moment magnification factor, δ_b, must be a positive number and greater than one. Therefore P_f must be less than $\varphi_m P_c$. If P_f is found to be greater than or equal to $\varphi_m P_c$, a failure condition is declared. δ_b is taken as 1 for tension members.

The preceding calculations use the unsupported length of the column. The two unsupported lengths are l_{22} and l_{33} corresponding to instability in the minor and major directions of the element, respectively. These are the lengths between the support points of the element in the corresponding directions.

If the program assumptions are not satisfactory for a particular member, the user can explicitly specify values of δ_s and δ_b.

3.5.2.3   Determine Capacity Ratio

As a measure of the stress condition of the column, a capacity ratio is calculated. The capacity ratio is basically a factor that gives an indication of the stress condition of the column with respect to the capacity of the column.

Before entering the interaction diagram to check the column capacity, the moment magnification factors are applied to the factored loads to obtain P_f, M_{fx}, and M_{fy}. The point (P_f, M_{fx}, M_{fy}) is then placed in the interaction space, shown as point L in Figure 3-4. If the point lies within the interaction volume, the column capacity is adequate; however, if the point lies outside the interaction volume, the column is overstressed.
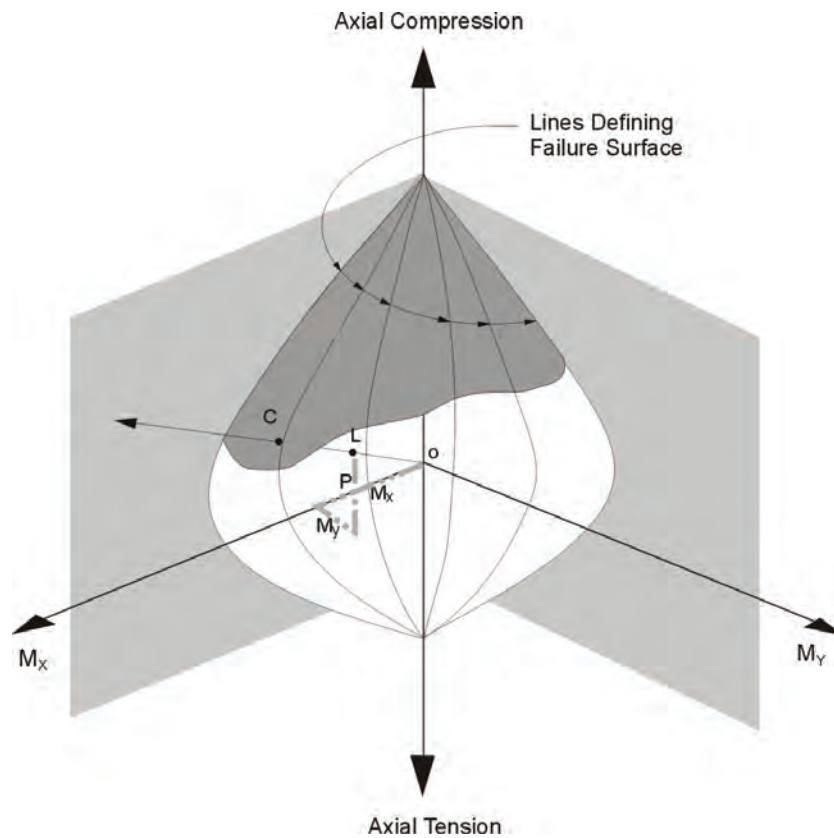
Figure 3-4 Geometric representation of column capacity ratio

This capacity ratio is achieved by plotting the point L and determining the location of point C. The point C is defined as the point where the line OL (if extended outwards) will intersect the failure surface. This point is determined by three-dimensional linear interpolation between the points that define the failure surface. See Figure 3-4. The capacity ratio, CR, is given by the ratio $\frac{OL}{OC}$.

- If OL = OC (or CR = 1) the point lies on the interaction surface and the column is stressed to capacity.

- If OL < OC (or CR < 1) the point lies within the interaction volume and the column capacity is adequate.

- If OL > OC (or CR > 1) the point lies outside the interaction volume and the column is overstressed.

The maximum of all the values of CR calculated from each load combination is reported for each check station of the column, along with the controlling P_f, M_{fx}, and M_{fy} set and associated load combination number.

If the reinforcing area is not defined, the program computes the reinforcement that will give an interaction ratio of unity.

3.5.3   Required Reinforcing Area

If the reinforcing area is not defined, the program computes the reinforcement that will give a column capacity ratio equal to the Utilization Factor Limit, which is set to 0.95 by default.

3.5.4   Design Column Shear Reinforcement

The shear reinforcement is designed for each loading combination in the major and minor directions of the column. In designing the shear reinforcing for a particular column for a particular loading combination due to shear forces in a particular direction, the program uses the following steps:

- Determine the factored forces acting on the section, M_f, P_f, and V_f. Note that M_f and P_f are needed for the calculation of v_c.

- Determine the shear stress, v_c, that can be resisted by concrete alone.

- Calculate the reinforcement steel required to carry the balance.

For Ductile and Moderately Ductile moment resisting concrete frames, the shear design of the columns is also based on the probable and nominal moment capacities, respectively, of the members in addition to the factored moments (CSA 21.4.5.1(b), 21.7.2.3(b)). Effects of the axial forces on the column moment capacities are included in the formulation.

The following three sections describe in detail the algorithms associated with this process.

3.5.4.1   Determine Section Forces

In the design of the column shear reinforcement of a Conventional moment resisting concrete frame, the forces for a particular load combination, namely, the column axial force, P_f, the column moment, M_f, and the column shear force, V_f, in a particular direction are obtained by factoring the program load cases with the corresponding load combination factors.

In the shear design of Ductile moment resisting frames (seismic design), the following are checked in addition to the requirement for Conventional moment resisting frames. In the design of Ductile moment resisting concrete frames, the design shear force, V_f, in a particular direction is also calculated from the probable moment capacities of the column associated with the factored axial force acting on the column (CSA 21.4.5.1(b)).

For each load combination, the factored axial load, P_f, is calculated. Then, the positive and negative moment capacities, M_f^+ and M_f^-, of the column in a particular direction under the influence of the axial force P_f are calculated using the uniaxial interaction diagram in the corresponding direction. The design shear force, V_f, is then given by

$$V_f = V_p + V_{D+L} \qquad\qquad \text{(CSA 21.4.5.1(b))}$$

where, V_p is the shear force obtained by applying the calculated probable ultimate moment capacities at the two ends of the column acting in two opposite directions. Therefore, V_p is the maximum of V_{P1} and V_{P2}, where

$$V_{P1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

M_I^+, M_I^- = Positive and negative moment capacities at end I of the column using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$),

M_J^+, M_J^- = Positive and negative moment capacities at end J of the column using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$), and

L = Clear span of column.

For Ductile moment resisting frames, α is taken as 1.25 (CSA 2.2, 21.4.5.1). V_{D+L} is the contribution of shear force from the in-span distribution of gravity loads. For most of the columns, it is zero.

The V_p is enforced not to be greater than the factored shear force component from specified gravity load increased by $R_d R_o$ (CSA 21.4.5.1).

In the design of Moderately Ductile moment resisting frames (seismic), the shear capacity of the column is also checked for the nominal shear due to the nominal $(\varphi_s = \varphi_c = 1.0)$ moment capacities and the factored gravity load (CSA 21.7.2.3), in addition to the design checks required for Conventional moment resisting frames. The procedure for calculating nominal moment capacity is the same as that for computing the probable moment capacity for Ductile moment resisting frames, except that α is taken equal to 1 (CSA 2.2, 21.7.2.3a) rather than 1.25. The design shear strength is enforced not to be greater than the fac-

tored shear force with the seismic load increased by $R_d R_o$ (CSA 21.7.2.3b). See also Table 3-1 for details.

3.5.4.2 Determine Concrete Shear Capacity

Given the design force set M_f, N_f, and V_f, the shear capacity provided by the concrete alone, V_c, is calculated as follows:

$$V_c = \varphi_c \lambda \beta \sqrt{f_c'} b_w d,$$
(CSA 11.3.4)

φ_c is the resistance factor for concrete. By default, it is taken as 0.65 (CSA8.4.2). For concrete produced in a pre-qualified manufacturing plants, its value can be taken as 0.70 (CSA 16.1.3). This value can be overwritten in the Preferences.

λ is the strength reduction factor to account for low density concrete (CSA 2.2). For normal density concrete, its value is 1 (CSA 8.6.5), which is the program default value. For concrete using lower density aggregate, the user can change the value of λ in the material properties. The recommended values for λ is as follows (CSA 8.6.5).

$$\lambda = \begin{cases} 1.00, & \text{for normal density concrete,} \\ 0.85, & \text{for semi-low-density concrete} \\ & \text{in which all of the fine aggregate is natural sand,} \\ 0.75, & \text{for semi-low-density concrete} \\ & \text{in which none of the fine aggregate is natural sand.} \end{cases}$$

β is the factor for accounting for the shear resistance of cracked concrete (CSA 2.2). Its value is normally between 0.1 and 0.4. It is determine according to section 11.3.6 of the Code, which is described in the sections that follow.

b_w is the width of the cross-section resisting the shear perpendicular to the shear force direction. For columns with rectangular cross-sections, b_w is taken as the width of the section perpendicular to the shear direction. For

columns with circular cross-sections, b_w is taken as the average width appropriate for shear direction.

d_v is the effective shear depth. It is taken as the greater of 0.9d or 0.72h,

$$d_v = \max\{0.9d, 0.72h\} \qquad \text{(CSA 2.2)}$$

where d is the distance from the extreme compression fiber to the centroid of tension reinforcement, and h is the overall depth of the cross-section in the direction of shear force.

The value of the β factor is preferably determine using the Simplified method (CSA 11.3.6.3) if applicable. When the conditions of Simplified method do not apply, the General method (CSA 11.3.6.4) is used. For designing concrete column sections for shear force, the special value of β does not apply (CSA 11.3.6.4)

When the axial force is compressive, the specified yield strength of the longitudinal reinforcing f_y does not exceed 400 MPa, and the specified concrete strength f_c' does not exceed 60 MPa, β is determined in accordance with the Simplified method, as follows (CSA 11.6.3.3):

▪ When the section contains at least the minimum transverse reinforcement, β is taken as 0.18 (CSA 11.6.3.3a).

$$\beta = 0.18 \qquad \text{(CSA 11.6.3.a)}$$

▪ When the section contains no transverse reinforcement, β is determined based on the specified maximum nominal size of coarse aggregate, a_g.

For maximum-sized coarse aggregate not less than 20 mm, β is taken as:

$$\beta = \frac{230}{1000 + d_v} \qquad \text{(CSA 11.3.6.3 b)}$$

where d_v is the effective shear depth expressed in millimeter, as described in preceding sections.

For maximum sized coarse aggregate less than 20 mm, β is taken as:

$$\beta = \frac{230}{1000 + s_{ze}} \qquad \text{(CSA 11.3.6.3 (c))}$$

where, $S_{ze} = \dfrac{35}{15 + a_g} S_z \geq 0.85 S_z.$ \qquad (CSA 11.3.6.3.c)

In the preceding expression, the crack spacing parameter, S_{ze}, shall be taken as the minimum of d_v and the maximum distance between layers of distributed longitudinal reinforcement. However, S_{ze} is conservatively taken as equal to d_v.

In summary, for simplified cases, β can be expressed as follows:

$$\beta = \begin{cases} 0.18, & \text{if minimum transverse reinforcement is provided,} \\[2mm] \dfrac{230}{1000 + d_v}, & \text{if no transverse reinforcement provided, and } a_g \geq 20\text{mm,} \\[2mm] \dfrac{230}{1000 + S_{ze}}, & \text{if no transverse reinforcement provided, and } a_g < 20\text{mm.} \end{cases}$$

When the axial force is tensile, or the specified yield strength of the longitudinal reinforcing $f_y > 400$ MPa, or the specified concrete strength $f_c' > 60$ MPa, β is determined in accordance with the General method, as follows (CSA 11.3.6.1, 11.3.6.4):

$$\beta = \frac{0.40}{\left(1 + 1500\varepsilon_x\right)} \bullet \frac{1300}{\left(1000 + S_{ze}\right)} \qquad \text{(CSA 11.3.6.4)}$$

In the preceding expression, the equivalent crack spacing parameter, S_{ze}, is taken as equal to 300 mm if minimum transverse reinforcement is provided (CSA 11.3.6.4). Otherwise it is determines as stated in simplified method.

$$S_{ze} = \begin{cases} 300, & \text{if minimum transverse} \\ & \text{reinforcement is provided,} \\ \dfrac{35}{15+a_g}S_z \ge 0.85S_z, & \text{otherwise.} \end{cases}$$

The longitudinal strain, ε_x, at mid-depth of the cross-section is computed from the following equation:

$$\varepsilon_x = \frac{M_f/d_v + V_f + 0.5N_f}{s(E_s A_s)} \qquad \text{(CSA 11.3.6.4)}$$

In evaluating the ε_x, the following conditions apply:

- V_f and M_f are taken as positive quantities   (CSA 11.3.6.4a)

- A_s is taken as the total area of longitudinal reinforcement in the column section. For the column section check option, the program uses the sum of user-defined reinforcement in the section. For the column section design option, the longitudinal reinforcement area is taken as the envelope of reinforcement required for all design load combinations. Actual provided reinforcement might be slightly higher than this quantity. The reinforcement should be developed to achieve full strength (CSA 11.3.6.3 b).

- If the value of ε_x calculated from the preceding equation is negative, it is recalculated as follows:

$$\varepsilon_x = \frac{M_f/d_v + V_f + 0.5N_f}{2(E_s A_s + E_c A_{ct})} \ge -0.0002 \qquad \text{(CSA 11.3.6.4 d)}$$

For sections closer than d_v from the face of the support, ε_x is calculated based on $M_f, V_f,$ and N_f at a section at a distance d_v from the face of the support (CSA 11.3.6.4 d).

If the axial tension is large enough to crack the flexural compression face of the section, the value of ε_x is increased by a factor of 2 (CSA 11.3.6.4 e). The program uses a linear elastic stress distribution to check this condition.

An upper limit on ε_x is imposed as follows:

$$e_x \leq 0.003 \qquad\qquad \text{(CSA 11.3.6.4 f)}$$

ε_x is positive for tensile action.

N_f is positive for tensile action.

For Ductile moment resisting frames, an upper limit on β of 0.10 is enforced (CSA 21.4.5.2) as follows:

$$\beta \leq 0.10 \qquad\qquad \text{(CSA 21.4.5.2)}$$

In both the Simplified method and the General method, the shear strength of the section due to concrete, v_c, depends on whether the minimum transverse reinforcement is provided. To check this condition, the program performs the design in two passes. In the first pass, it is assumes that no transverse shear reinforcement is needed. When the program determines that shear reinforcement is need, the program performs the second pass with the assumption that at least minimum shear reinforcement is provided.

3.5.4.3 Determine Required Shear Reinforcement

The minimum possible factored shear resistance, $V_{r,max}$ is computed as follows:

$$V_{r,max} = 0.25\varphi_c f_c' b_w d. \qquad\qquad \text{(CSA 11.3.3)}$$

The shear reinforcement per unit spacing, A_v/s, is computed as follows:

If $V_f \leq V_c$,

$$\frac{A_v}{s} = 0 \qquad\qquad \text{(CSA 11.3.5.1)}$$

else if $V_c < V_f \leq V_{r,max}$

$$\frac{A_v}{s} = \frac{(V_f - V_c)\tan\theta}{\varphi_s f_{yh} d_v}$$ (CSA 11.3.3, 11.3.5.1)

else if $V_f > V_{r,max}$

a failure condition is declared. (CSA 11.3.3)

- A minimum area of shear reinforcement is provided in the following regions (CSA 11.2.8.1):

  (a) in regions of flexural members where the factored shear force V_f exceeds V_c,

  (b) in regions of beams and columns with an overall depth greater than 750 mm, and

  (c) in regions of beams and columns where the factored torsion T_f exceeds $0.25 T_{cr}$.

- Where the minimum shear reinforcement is required by section CSA 11.2.8.1, or by calculations, the minimum area of shear reinforcement per unit spacing is taken as:

  $$\frac{A_v}{s} \geq \frac{\sqrt{f_c'}}{f_y} b_w$$ (CSA 11.2.8.2)

- In the preceding, the term θ is used. Here θ is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the member. The θ value is normally between 22 to 44 degrees. It is determined according to Section 11.3.6 of the Code.

  Similar to the β factor, which was described previously, the value of θ is preferably determined using the Simplified method (CSA 11.3.6.3), whenever applicable. The program uses the General method when the conditions for the Simplified methods are not satisfied (CSA 11.3.6.4). For designing

concrete column sections for shear forces, the special value of θ does not apply (CSA 11.3.6.2).

If the axial force is compressive, the specified yield strength of the longitudinal reinforcing f_y does not exceed 400 MPa, and the specified concrete strength f_c' does not exceed 60 MPa, θ is taken to be 35 degrees (CSA 11.3.6.3).

$$\theta = 35^{\circ} \text{ for } P_f \leq 0 \text{ or } f_y \leq 400 \text{MPa or } f_c' \leq 60 \text{MPa} \qquad \text{(CSA11.3.6.4)}$$

If the axial force is tensile, the specified yield strength of the longitudinal reinforcing $f_y > 400$ MPa, and the specified concrete strength $f_c' > 60$ MPa, θ is determined using the General method as follows (CSA 11.3.6.4).

$$\theta = 29 + 7000\varepsilon_x$$

$$\text{for } P_f < 0 \text{ or } f_y > 400 \text{ MPa or } f_c' > 60 \text{ MPa} \qquad \text{(CSA11.3.6.4)}$$

where ε_x is the longitudinal strain at the mid-depth of the cross-section for the factored load. The calculation procedure has been described in the preceding sections.

The maximum of all of the calculated A_v/s values obtained from each load combination is reported for the major and minor directions of the column, along with the controlling shear force and associated load combination number.

For all columns and at any station, the minimum area of transverse circular hoop reinforcement is imposed as follows:

$$\frac{A_v}{s} \geq 0.45 \left[\frac{A_g}{A_{core}} - 1 \right] \frac{f_c'}{f_{yh}} \frac{h_{core}}{4} \qquad \text{(CSA 10.9.4)}$$

In potential plastic hinge locations of Ductile moment resisting frames as described later, the minimum area of circular hoops and transverse stirrups is imposed as follows:

$$\frac{A_v}{s} \geq \frac{P_f}{P_o} \frac{f_c'}{f_{yh}} \frac{h_{\text{core}}}{4} \qquad \text{(Hoops)} \qquad \text{(CSA 21.4.4.2)}$$

$$\frac{A_v}{s} \geq 0.45 \left[\frac{A_g}{A_{\text{core}}} - 1 \right] \frac{f_c'}{f_{yh}} \frac{h_{\text{core}}}{4} \qquad \text{(Hoops)} \qquad \text{(CSA 21.4.4.2, 10.9.4)}$$

$$\frac{A_v}{s} \geq 0.09 \frac{f_c'}{f_{yh}} h_{\text{core}} \qquad \text{(Stirrups)} \qquad \text{(CSA 21.4.4.2)}$$

$$\frac{A_v}{s} \geq 0.2 K_n K_p \frac{A_g}{A_{\text{core}}} h_{\text{core}} \qquad \text{(Stirrups)} \qquad \text{(CSA 21.4.4.2)}$$

where $K_n = \dfrac{n_1}{(n_1 - 2)}$ and $K_p = \dfrac{P_f}{P_o}$

Transverse reinforcement shall be provided over a length, l_o, from the face of each joint and on both sides of any section where flexural yielding can occur as a result of inelastic displacement of the frame. The length, l_o, is determined as follows:

(a) where $P_f \leq 0.5 \phi_c f_c' A_g$, l_o shall not be less than 1.5 times the largest member cross-section dimension or 1/6 of the clear span of the member; and

(b) where $P_f > 0.5 \phi_c f_c' A_g$, l_o shall not be less than 2.0 times the largest member cross-section dimension or 1/6 of the clear span of the member (CSA 21.4.4.5).

The column shear reinforcement requirements reported by the program are based on the preceding considerations. Any other minimum stirrup requirements to satisfy spacing or volumetric requirements must be investigated independently of the program by the user.

3.6   Beam Design

In the design of concrete beams, the program calculates and reports the required areas of steel for flexure and shear based on the beam moments, shears, load combination factors and other criteria described in the subsections that follow. The reinforcement requirements are calculated at a user-defined number of check stations along the beam span.

All of the beams are designed for major direction flexure and shear only. Effects due to any axial forces, minor direction bending, and torsion that may exist in the beams must be investigated independently by the user.

The beam design procedure involves the following steps:

▪ Design beam flexural reinforcement

▪ Design beam shear reinforcement

3.6.1   Design Beam Flexural Reinforcement

The beam top and bottom flexural steel is designed at a user-defined number of design stations along the beam span. In designing the flexural-reinforcement for the major moment of a particular beam for a particular section, the program uses the following steps:

▪ Determine the maximum factored moments

▪ Determine the reinforcing steel

3.6.1.1   Determine Factored Moments

In the design of flexural reinforcement of Ductile (CSA 21.5), Moderately Ductile (CSA 21.7), or Conventional (CSA 21.8) moment resisting concrete frame beams, the factored moments for each load combination at a particular beam station are obtained by factoring the corresponding moments for different load cases with the corresponding load factors. The beam section is then designed for the maximum positive M_f^+ and maximum negative M_f^- factored moments obtained from all of the load combinations.

Positive beam moments produce bottom steel. In such cases, the beam may be designed as a Rectangular or T beam. Negative beam moments produce top steel. In such cases, the beam is always designed as a Rectangular section.

3.6.1.2 Determine Required Flexural Reinforcement

In the flexural reinforcement design process, the program calculates both the tension and compression reinforcement. Compression reinforcement is added when the applied design moment exceeds the maximum moment capacity of a singly reinforced section. The user has the option of avoiding the compression reinforcement by increasing the effective depth, the width, or the grade of concrete.

The design procedure is based on the simplified rectangular stress block, as shown in Figure 3-5 (CSA 10.1.7).
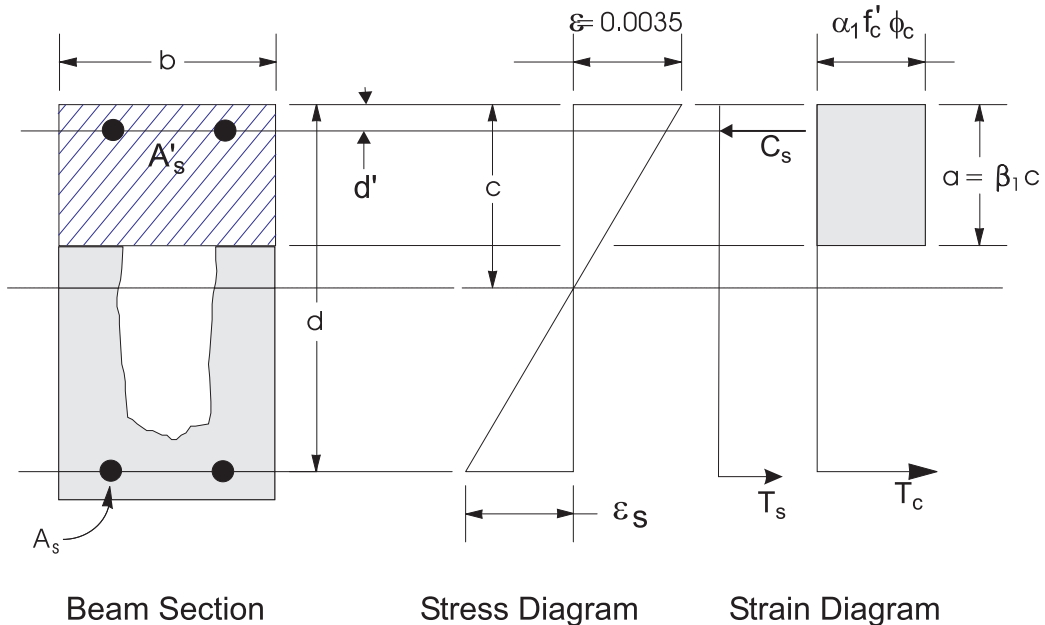

Figure 3-5 Rectangular beam design

Furthermore, it is assumed that the compression carried by concrete is less than that which can be carried at the balanced condition (CSA 10.1.4). When the applied moment exceeds the moment capacity at the balanced condition, the area of compression reinforcement is calculated on the assumption that the ad-

ditional moment will be carried by compression and additional tension rein-forcement.

The design procedure used by the program for both rectangular and flanged sections (L and T beams) is summarized in the sections that follow. It is assumed that the design ultimate axial force in a beam is negligible; hence, all of the beams are designed for major direction flexure and shear only.

3.6.1.2.1 Design for Flexure of a Rectangular Beam

In designing for a factored negative or positive moment, M_f, (i.e., designing top or bottom steel) the depth of the compression block is given by a, as shown in Figure 3-5, where,

$$a = d - \sqrt{d^2 - \frac{2|M_f|}{\alpha_1 f_c' \varphi_c b}}, \qquad\qquad \text{(CSA 10.1)}$$

where the value of φ_c is 0.65 (CSA 8.4.2) in the preceding and following equations. Also , α_1, β_1, and c_b are calculated as follows:

$$\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67, \qquad\qquad \text{(CSA 10.1.7)}$$

$$\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67, \quad \text{and} \qquad\qquad \text{(CSA 10.1.7)}$$

$$c_b = \frac{700}{700 + f_y} d. \qquad\qquad \text{(CSA 10.5.2)}$$

The balanced depth of the compression block is given by

$$a_b = \beta_1 c_b. \qquad\qquad \text{(CSA 10.1.4, 10.1.7)}$$

- If $a \leq a_b$ (CSA 10.5.2), the area of tensile steel reinforcement is then given by

$$A_s = \frac{M_f}{\varphi_s f_y \left(d - \dfrac{a}{2} \right)}.$$

This steel is to be placed at the bottom if M_f is positive, or at the top if M_f is negative.

- If $a > a_b$ (CSA 10.5.2), compression reinforcement is required and is calculated as follows:

The factored compressive force developed in the concrete alone is given by

$$C = \varphi_c \alpha_{a_c} f_c' b a_b, \text{ and} \qquad \text{(CSA 10.1.7)}$$

the factored moment resisted by the concrete and bottom steel is

$$M_{fc} = C \left(d - \frac{a_b}{2} \right).$$

The moment resisted by compression steel and tensile steel is

$$M_{fs} = M_f - M_{fc}.$$

So the required compression steel is given by

$$A_s' = \frac{M_{fs}}{\left(\varphi_s f_s' - \varphi_c \alpha_1 f_c' \right) (d - d')}, \text{ where}$$

$$f_s' = 0.0035 E_s \left[\frac{c - d'}{c} \right] \le f_y. \qquad \text{(CSA 10.1.2 , 10.1.3)}$$

The required tensile steel for balancing the compression in concrete is

$$A_{s1} = \frac{M_{fc}}{f_y \left(d - \frac{a_b}{2} \right) \varphi_s}, \text{ and}$$

the tensile steel for balancing the compression in steel is

$$A_{s2} = \frac{M_{fs}}{f_y (d - d') \varphi_s}.$$

Therefore, the total tensile reinforcement $A_s = A_{s1} + A_{s2}$, and total compression reinforcement is $A_s{}'$. A_s is to be placed at the bottom, and $A_s{}'$ is to be placed at the top if M_f is positive, and vice versa.

3.6.1.2.2 Design for Flexure of a T Beam

Flanged Beam Under Negative Moment

In designing for a factored negative moment, M_f, (i.e., designing top steel), the contribution of the flange to the strength of the beam is ignored. The design procedure is therefore identical to the one used for Rectangular beams, except that in the corresponding equations, b is replaced by b.

Flanged Beam Under Positive Moment

If $M_f > 0$, the depth of the compression block is given by (see Figure 3-6).

$$a = d - \sqrt{d^2 - \frac{2M_f}{\alpha_1 f_c' \varphi_c b_f}}.$$

(CSA 10.1)

where the value of φ_c is 0.65 (CSA 8.4.2) in the preceding and following equations. See Figure 3-6. Also α_1, β_1, and c_b are calculated as follows:

$$\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67,$$

(CSA 10.1.7)

$$\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67, \text{ and}$$

(CSA 10.1.7)

$$c_b = \frac{700}{700 + f_y} d.$$

(CSA 10.5.2)

The depth of the compression block under balanced condition is given by

$$a_b = \beta_1 c_b.$$

(CSA 10.1.4, 10.1.7)

- If $a \leq d_s$, the subsequent calculations for A_s are exactly the same as previously performed for the rectangular section design. However, in this case,

the width of the beam is taken as b_f. Whether compression reinforcement is required depends on if $a > a_b$.

Beam Section   Strain Diagram   Stress Diagram
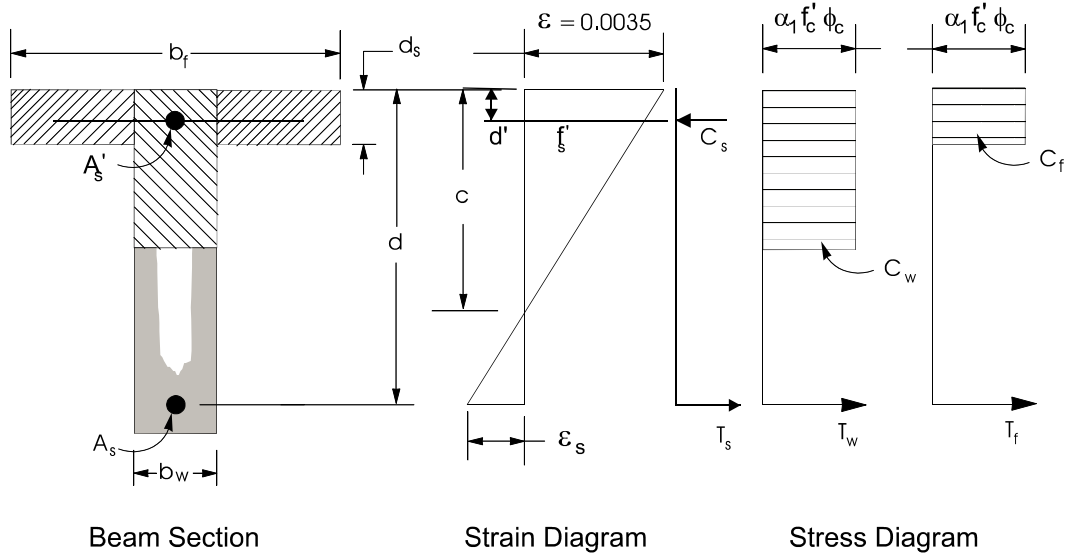
Figure 3-6 T beam design

- If $a > d_s$, calculation for A_s is performed in two parts. The first part is for balancing the compressive force from the flange, C_f, and the second part is for balancing the compressive force from the web, C_w. As shown in,

$$C_f = \alpha_1 f_c' \left(b_f - b_w \right) d_s.$$ (CSA 10.1.7)

Therefore, $A_{s1} = \dfrac{C_f \varphi_c}{f_y \varphi_s}$ and the portion of M_f that is resisted by the flange is

$$M_{ff} = C_f \left(d - \frac{d_s}{2} \right) \varphi_c.$$

Therefore, the balance of the moment, M_f to be carried by the web is given by

$$M_{fw} = M_f - M_{ff}.$$

The web is a Rectangular section of dimensions b_w and d, for which the depth of the compression block is recalculated as

$$a_1 = d - \sqrt{d^2 - \frac{2M_{fw}}{\alpha_1 f_c' \varphi_c b_w}}.$$

(CSA 10.1)

- If $a_1 \leq a_b$ (CSA 10.5.2), the area of tensile steel reinforcement is then given by

$$A_{s2} = \frac{M_{fw}}{\varphi_s f_y \left(d - \dfrac{a_1}{2} \right)}, \text{ and}$$

$$A_s = A_{s1} + A_{s2}.$$

This steel is to be placed at the bottom of the T beam.

- If $a_1 > a_b$ (CSA 10.5.2), compression reinforcement is required and is calculated as follows:

The factored compressive force in the concrete web alone is given by

$$C = \varphi_c \alpha_1 f_c' b a_b, \text{ and}$$

(CSA 10.1.7)

the factored moment resisted by the concrete web and tensile steel is

$$M_{fc} = C \left(d - \frac{a_b}{2} \right).$$

The moment resisted by compression steel and tensile steel is

$$M_{fs} = M_{fw} - M_{fc}.$$

Therefore, the compression steel is computed as

$$A_s' = \frac{M_{fs}}{\left(\varphi_s f_s' - \varphi_c \alpha_1 f_c' \right)(d - d')}, \text{ where}$$

$$f_s' = 0.0035E_s \left[\frac{c - d'}{c} \right] \le f_y. \qquad\qquad \text{(CSA 10.1.2, 10.1.3)}$$

The tensile steel for balancing compression in web concrete is

$$A_{s2} = \frac{M_{fc}}{f_y \left(d - \dfrac{a_b}{2} \right) \varphi_s}, \quad \text{and}$$

the tensile steel for balancing compression in steel is

$$A_{s3} = \frac{M_{fs}}{f_y \left(d - d' \right) \varphi_s}.$$

Total tensile reinforcement $A_s = A_{s1} + A_{s2} + A_{s3}$, and total compression reinforcement is A_s'. A_s is to be placed at the bottom and A_s' is to be placed at the top.

3.6.1.3 Minimum and Maximum Tensile Reinforcement

The minimum flexural tensile steel required for a beam section is given by the minimum of the two limits:

$$A_s \ge \frac{0.2\sqrt{f_c'}}{f_y} b_w h, \quad \text{or} \qquad\qquad \text{(CSA 10.5.1.2)}$$

$$A_s = \frac{4}{3} A_{s(required)}. \qquad\qquad \text{(CSA 10.5.1.3)}$$

In addition, the minimum flexural tensile steel provided in a T section with flange under tension is given by the limit:

$$A_s \ge 0.004 \left(b - b_w \right) d_s. \qquad\qquad \text{(CSA 10.5.3.1)}$$

An upper limit of 0.04 times the gross web area on both the tension reinforcement and the compression reinforcement is arbitrarily imposed as follows:

$$A_s \leq \begin{cases} 0.04bd & \text{Rectangular beam} \\ 0.04b_w d & \text{T beam} \end{cases}$$

$$A_s' \leq \begin{cases} 0.04bd & \text{Rectangular beam} \\ 0.04b_w d & \text{T beam} \end{cases}$$

3.6.1.4 Special Consideration for Seismic Design

For Ductile moment resisting concrete frames (seismic design), the beam design satisfies the following additional conditions (see also Table 3-2 for a comprehensive listing):

▪ The minimum longitudinal reinforcement shall be provided at both the top and the bottom. Any of the top and bottom reinforcement shall not be less than $A_{s(\min)}$.

$$A_{s(\min)} \geq \frac{1.4}{f_y} b_w d \qquad\qquad \text{(CSA 21.3.2.1)}$$

▪ The beam flexural steel is limited to a maximum given by

$$A_s \leq 0.025 b_w d. \qquad\qquad \text{(CSA 21.3.2.1)}$$

▪ At any end (support) of the beam, the beam positive moment capacity (i.e., associated with the bottom steel) would not be less than one-half of the beam negative moment capacity (i.e., associated with the top steel) at that end (CSA 21.3.2.2).

▪ Neither the negative moment capacity nor the positive moment capacity at any of the sections within the beam would be less than one-fourth of the maximum of positive or negative moment capacities of any of the beam end (support) stations (CSA 21.3.2.2).

For Conventional moment resisting concrete frames (seismic design), the beam design would satisfy the following conditions:

▪ At any support of the beam, the beam positive moment capacity would not be less than one-third of the beam negative moment capacity at that end (CSA 21.7.2.1.1).

▪ Neither the negative moment capacity nor the positive moment capacity at

any of the sections within the beam would be less than one-fifth of the maximum of positive or negative moment capacities of any of the beam end (support) stations (CSA 21.7.2.1.1).

Table 3-2: Design Criteria

Type of Check/ Design	Conventional Moment Resisting Frames (Non-Seismic)	Moderately Ductile Moment Resisting Frames (Seismic)	Ductile Moment Resisting Frames (Seismic)
Strength of Materials	$f_c' \leq 80$ MPa $f_c' \leq 500$ MPa $\sqrt{f_c'} \leq 8$ MPa	$f_c' \leq 80$ MPa $f_c' \leq 500$ MPa $\sqrt{f_c'} \leq 8$ MPa	$f_c' \leq 80$ MPa $f_c' \leq 500$ MPa $\sqrt{f_c'} \leq 8$ MPa
Column Check (interaction)	NLD[a] Combinations	NLD[a] Combinations	NLD[a] Combinations
Column Design (Interaction)	NLD[a] Combinations $1\% < \rho < 8\%$	NLD[a] Combinations $1\% < \rho < 8\%$	NLD[a] Combinations $1\% < \rho < 6\%$ $\alpha = 1.0$
Column Shears	NLD[a] Combinations	Modified NLD[a] Combinations or Nominal Capacity Shear (V_p) $\varphi = 1.0$ and $\alpha = 1.0$	NLD[a] Combinations and Probable Capacity Shear (V_p) $\varphi = 1.0$ and $\alpha = 1.25$ Strength based on $\beta \leq 0.1$, $\theta \geq 45°$ at potential plastic hinge region
	Minimum hoop	Minimum hoop reinforcement	Minimum hoop reinforcement Special reinforcement in hinge
Beam Design Flexure	NLD[a] Combinations $\rho \leq 0.04$ $\rho \geq 0.2 \sqrt{f_c'}/f_y$	NLD[a] Combinations $\rho \leq 0.04$ $\rho \geq 0.2 \sqrt{f_c'}/f_y$	NLD[a] Combinations $\rho \leq 0.04$ $\rho \geq 0.2 \sqrt{f_c'}/f_y$
Beam Min. Moment Override Check	No Requirement	$M_{uEND}^+ \geq \dfrac{1}{3} M_{uEND}^-$ $M_{uSPAN}^+ \geq \dfrac{1}{5} max\left\{M_u^+, M_u^-\right\}_{END}$ $M_{uSPAN}^- \geq \dfrac{1}{5} max\left\{M_u^+, M_u^-\right\}_{END}$	$M_{uEND}^+ \geq \dfrac{1}{2} M_{uEND}^-$ $M_{uSPAN}^+ \geq \dfrac{1}{4} max\left\{M_u^+, M_u^-\right\}_{END}$ $M_{uSPAN}^- \geq \dfrac{1}{4} max\left\{M_u^+, M_u^-\right\}_{END}$
Beam Design Shear	NLD[a] Combinations	NLD[a] Combinations or Nominal Capacity Shear (V_p) with $\alpha = 1.0$ and $\varphi = 1.0$ plus V_{D+L}, or modified load combination with earthquake load amplified by $R_d R_o$.	NLD[a] Combinations and Probable Capacity Shear (V_p) with $\alpha = 1.0$ and $\varphi = 1.0$ plus V_{D+L}, or modified load combination with earthquake load amplified by $R_d R_o$; strength based on $\beta = 0$ and $\beta = 45$ degree in potential plastic hinge.

3.6.2 Design Beam Shear Reinforcement

The shear reinforcement is designed for each load combination at a user-defined number of stations along the beam span. In designing the shear reinforcement for a particular beam for a particular loading combination at a particular station due to the beam major shear, the program uses the following steps:

▪ Determine the factored forces acting on the section, M_f and V_f. Note that M_f is needed for the calculation of v_c.

▪ Determine the shear stress, v_c, that can be resisted by concrete alone.

▪ Calculate the reinforcement steel required to carry the balance.

For Ductile and Moderately ductile moment resisting frames, the shear design of the beams is also based on the probable and nominal moment capacities of the members, respectively (CSA 21.3.4.1, 21.7.2.3).

The following three sections describe in detail the algorithms associated with these steps.

3.6.2.1 Determine Shear Force and Moment

In the design of the beam shear reinforcement of an Conventional moment re-sisting concrete frame, the shear forces and moments for a particular load combination at a particular beam section are obtained by factoring the associated shear forces and moments with the corresponding load combination factors.

In the design of Ductile moment resisting concrete frames (seismic design), the shear capacity of the beam is also checked for the probable shear due to the probable moment capacities and the factored gravity load, in addition to the design checks required for Conventional moment resisting frames. The proce-dure for calculating the design shear force in a beam from probable moment capacities is the same as that described for a column in the *Column Design* sec-tion. See also Table 3-2 and Figure 3-7 for more details. The design shear force V_f is then given by

$$V_f = V_p + V_{D+L} \qquad\qquad \text{(CSA 21.3.4.1)}$$

where, V_p is the shear force obtained by applying the calculated probable ultimate moment capacities at the two ends of the beams acting in two opposite directions. Therefore, V_p is the maximum of V_{P1} and V_{P2}, where

$$V_{P1} = \left(M_I^- + M_J^+ \right) / L, \text{ and}$$

$$V_{P2} = \left(M_I^- + M_J^+ \right) / L, \text{ where}$$

M_I^+ = Moment capacity at end I, with bottom steel in tension, using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$),

M_J^+ = Moment capacity at end J, with bottom steel in tension, using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$),

M_I^- = Moment capacity at end I, with top steel in tension, using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$),

M_J^- = Moment capacity at end J, with top steel in tension, using a steel yield stress value of αf_y and no φ factors ($\varphi_s = \varphi_c = 1.0$), and

L = Clear span of beam.

The overstrength factor α is always taken as 1.25 for Ductile moment resisting frames (CSA 2.2, 21.3.4.1). V_{D+L} is the contribution of shear force from the in-span distribution of gravity loads.

In the design of Moderately Ductile moment resisting frames (seismic), the shear capacity of the beam is also checked for the nominal shear due to the nominal ($\varphi_s = \varphi_c = 1.0$) moment capacities and the factored gravity load (CSA 21.7.2.3(a), 2.2), in addition to the design checks required for Conventional moment resisting frames. The procedure for calculating nominal moment capacity is the same as that for computing the probable moment capacity for Ductile moment resisting frames, except that α is taken equal to 1 (CSA 21.7.2.3.(a), 2.2) rather than 1.25. The design shear strength is enforced not to

be greater than the factored shear force with the seismic load amplified by a factor of $R_d R_o$ (CSA 21.3.4.1, 21.7.2.3(b)). See also Table 3-2 for details.
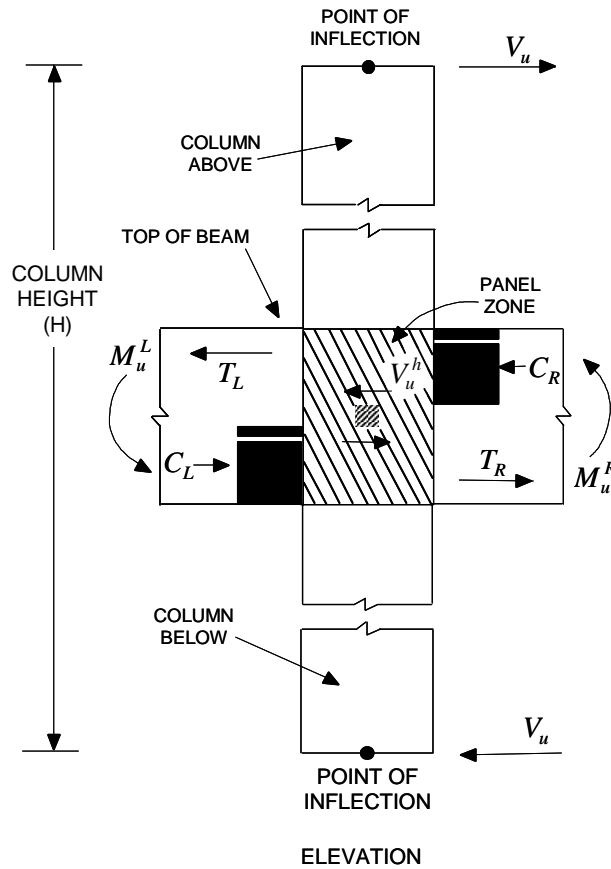

Figure 3-7 Column shear force V_u

3.6.2.2 Determine Concrete Shear Capacity

Given the design force set M_f and V_f, the shear capacity provided by the concrete alone, V_c, is calculated as follows:

$$V_c = \varphi_c \lambda \beta \sqrt{f_c'} \, b_w d, \tag{CSA 11.3.4}$$

φ_c is the resistance factor for concrete. By default it is taken as 0.65 (CSA8.4.2). For concrete produced in a pre-qualified manufacturing plants, its value can be taken as 0.70 (CSA 16.1.3). This value can be overwritten in the Preferences.

λ is the strength reduction factor to account for low density concrete (CSA 2.2). For normal density concrete, its value is 1 (CSA 8.6.5), which is taken by the program as the default value. For concrete using lower density aggregate, the user can change the value of λ in the material property data. The recommended value for λ is as follows (CSA 8.6.5).

$$\lambda = \begin{cases} 1.00, & \text{for normal density concrete,} \\ 0.85, & \text{for semi-low-density concrete} \\ & \text{in which all of the fine aggregate is natural sand,} \\ 0.75, & \text{for semi-low-density concrete} \\ & \text{in which none of the fine aggregate is natural sand.} \end{cases}$$

β is the factor for accounting for the shear resistance of cracked concrete (CSA 2.2). Its value is normally between 0.1 and 0.4. It is determine according to the section 11.3.6 of the Code, which is described in the sections that follow.

b_w is the effective web width. For rectangular beam, it is the width of the beam. For T beam, it is the width of the web of the beam.

d_v is the effective shear depth. It is taken as the greater of 0.9d or 0.72h,

$$d_v = \max\{0.9d, 0.72h\} \qquad\qquad \text{(CSA 2.2)}$$

where d is the distance from the extreme compression fiber to the centroid of tension reinforcement, and h is the overall depth of the cross-section in the direction of shear force.

The value of the β factor is preferably taken as the Special value (CSA 11.3.6.2) or it is determined using the Simplified method (CSA 11.3.6.3), if applicable. When the conditions of the Special value or Simplified method do not apply, the General method is used to determine the β factor (CSA 11.3.6.4).

If the overall beam depth, h, is less than 250 mm or if the beam depth of a T beam before the slab is not greater than one-half of the width of web or 350 mm, β is taken as 0.21 (CSA 11.3.6.2)

When the specified yield strength of the longitudinal reinforcing f_y does not exceed 400 MPa, and the specified concrete strength f_c' does not exceed 60 MPa, β is determined in accordance with the Simplified method, as follows (CSA 11.6.3.3):

- When the section contains at least the minimum transverse reinforcement, β is taken as 0.18 (CSA 11.6.3.3a).

$$\beta = 0.18 \hspace{4cm} \text{(CSA 11.6.3.a)}$$

- When the section contains no transverse reinforcement, β is determined based on the specified maximum nominal size of coarse aggregate, a_g.

For maximum size of coarse aggregate not less than 20 mm, β is taken as:

$$\beta = \frac{230}{1000 + d_v} \hspace{3cm} \text{(CSA 11.3.6.3 b)}$$

where is the effective shear depth expressed in millimeter, which is described in preceding sections.

For maximum size of coarse aggregate less than 20 mm, β is taken as:

$$\beta = \frac{230}{1000 + s_{ze}} \hspace{3cm} \text{(CSA 11.3.6.3 c)}$$

$$\text{where, } S_{ze} = \frac{35}{15 + a_g} S_z \geq 0.85 S_z \hspace{2cm} \text{(CSA 11.3.6.3.c)}$$

In the preceding expression, the crack spacing parameter, S_{ze}, shall be taken as the minimum of d_v and the maximum distance between layers of distributed longitudinal reinforcement. However, S_{ze} is conservatively taken as equal to d_v.

In summary, for Simplified cases, β can be expressed as follows:

$$\beta = \begin{cases} 0.18, & \text{if minimum transverse reinforcement is provided,} \\ \dfrac{230}{1000 + d_v}, & \text{if no transverse reinforcement provided, and } a_g \geq 20\text{mm,} \\ \dfrac{230}{1000 + S_{ze}}, & \text{if no transverse reinforcement provided, and } a_g < 20\text{mm.} \end{cases}$$

- When the specified yield strength of the longitudinal reinforcing $f_y > 400$ MPa, or the specified concrete strength $f_c' > 60$ MPa, β is determined in accordance with the General method as follows (CSA 11.3.6.1, 11.3.6.4):

$$\beta = \frac{0.40}{\left(1 + 1500\varepsilon_x\right)} \bullet \frac{1300}{\left(1000 + S_{ze}\right)} \qquad \text{(CSA 11.3.6.4)}$$

In the preceding expression, the equivalent crack spacing parameter, S_{ze}, is taken as equal to 300 mm if minimum transverse reinforcement is provided (CSA 11.3.6.4). Otherwise it is determine as stated in the Simplified method.

$$S_{ze} = \begin{cases} 300 & \text{if minimum transverse reinforcement is provided,} \\ \dfrac{35}{15 + a_g} S_z \geq 0.85 S_z & \text{otherwise.} \end{cases}$$

The longitudinal strain, ε_x, at mid-depth of the cross-section is computed from the following equation:

$$\varepsilon_x = \frac{M_f / d_v + V_f}{s\left(E_s A_s\right)} \qquad \text{(CSA 11.3.6.4)}$$

In evaluating the ε_x, the following conditions apply:

V_f and M_f are taken as positive quantities   (CSA 11.3.6.4a)

A_s is taken as the total area of longitudinal reinforcement in the beam section. It is taken as the envelope of reinforcement required for all de-

sign load combinations. Actual provided reinforcement might be slightly higher than this quantity. The rebar should be developed to achieve full strength (CSA 11.3.6.3 b).

For sections closer than d_v from the face of the support, ε_x is calculated based on M_f and V_f at a section at a distance d_v from the face of the support (CSA 11.3.6.4 d).

An upper limit on ε_x is imposed as follows:

$$e_x \leq 0.003 \qquad \text{(CSA 11.3.6.4 f)}$$

ε_x is positive for tensile action.

- For Ductile moment resisting frames, in the region of potential plastic hinge, the shear reinforcement is designed assuming β is equal to zero:

$$\beta = 0 \qquad \text{(CSA 21.3.4.2)}$$

In both the Simplified and General methods, the shear strength of the section due to concrete, v_c, depends on whether the minimum transverse reinforcement is provided. To check this condition, the program performs the design in two passes. In the first pass, it is assumes that no transverse shear reinforcement is needed. When the program determines that shear reinforcement is need, the program performs the second pass with the assumption that at least minimum shear reinforcement is provided.

3.6.2.3 Determine Required Shear Reinforcement

The minimum possible factored shear resistance, $V_{r,\max}$ is computed as follows:

$$V_{r,\max} = 0.25\varphi_c f_c' b_w d. \qquad \text{(CSA 11.3.3)}$$

The shear reinforcement per unit spacing, A_v/s, is computed as follows:

If $V_f \leq V_c$

$$\frac{A_v}{s} = 0 \qquad \text{(CSA 11.3.5.1)}$$

else if $V_c < V_f \le V_{r,\max}$

$$\frac{A_v}{s} = \frac{\left(V_f - V_c\right)\tan\theta}{\varphi_s f_{yh} d_v}$$
(CSA 11.3.3, 11.3.5.1)

else if $V_f > V_{r,\max}$

a failure condition is declared. (CSA 11.3.3)

A minimum area of shear reinforcement is provided in the following regions (CSA 11.2.8.1):

(a) in regions of flexural members where the factored shear force V_f exceeds V_c,

(b) in regions of beams and columns with an overall depth greater than 750 mm, and

(c) in regions of beams and columns where the factored torsion T_f exceeds 0.25 .

Where the minimum shear reinforcement is required by section CSA 11.2.8.1, or by calculations, the minimum area of shear reinforcement per unit spacing is taken as T_{cr}.

$$\frac{A_v}{s} \ge 0.06 \frac{\sqrt{f_c'}}{f_y} b_w$$
(CSA 11.2.8.2)

In the preceding, the term θ is used. Here θ is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the member. The θ value is normally between 22 to 44 degree. It is determined according to section 11.3.6 in the Code.

Similar to the β factor, which was described previously, the value of θ is preferably taken as the Special value (CSA 11.3.6.2) or it is determined using the Simplified method (CSA 11.3.6.3), whenever applicable. The program uses the General method when conditions for the Simplified method are not satisfied (CSA 11.3.6.4).

- If the overall beam depth, h, is less than 250 mm or if the depth of the T beam below the slab is not greater than one-half of the width of web or 350 mm, θ is taken as 42 degree (CSA 11.3.6.2).

- If the specified yield strength of the longitudinal reinforcing f_y does not exceed 400 MPa, and the specified concrete strength f_c' does not exceed 60 MPa, θ is taken to be 35 degree (CSA 11.3.6.3).

$$\theta = 35^o \text{ for } P_f \leq 0 \text{ or } f_y \leq 400 \text{ MPa or } 60 \text{ MPa} \qquad (CSA 11.3.6.4)$$

- If the axial force is tensile, the specified yield strength of the longitudinal reinforcing $f_y > 400$ MPa, and the specified concrete strength $f_c' > 60$ MPa, θ is determined using the General method as follows (CSA 11.3.6.4),

$$\theta = 29 + 7000\varepsilon_x \text{ for } P_f < 0 \text{ or } f_y > 400 \text{ MPa or } 60 \text{ MPa} \qquad (CSA 11.3.6.4)$$

where ε_x is the longitudinal strain at the mid-depth of the cross-section for the factored load. The calculation procedure has been described in preceding sections.

The maximum of all of the calculated A_v/s values, obtained from each load combination, is reported for the major directions of the beam along with the controlling shear force and associated load combination number.

The beam shear reinforcement requirements reported by the program are based purely on the considerations herein. Any other minimum stirrup requirements to satisfy spacing and volumetric requirements must be investigated independently of the program by the user.

3.7   Joint Design

To ensure that the beam-column joint of Ductile and Moderately Ductile moment resisting frames possesses adequate shear strength, the program performs a rational analysis of the beam-column panel zone to determine the shear forces that are generated in the joint. The program then checks this against design shear strength.

Only joints having a column below the joint are designed. The material properties of the joint are assumed to be the same as those of the column below the joint.

The joint analysis is completed in the major and the minor directions of the column. The joint design procedure involves the following steps:

- Determine the panel zone design shear force, V_u^h

- Determine the effective area of the joint

- Check panel zone shear stress

The following three sections describe in detail the algorithms associated with these steps.

3.7.1 Determine the Panel Zone Shear Force

For a particular column direction, major or minor, the free body stress condition of a typical beam-column intersection is shown in Figure 3-4.

The force V_u^h is the horizontal panel zone shear force that is to be calculated. The forces that act on the joint are P_f, V_f, M_f^L and M_f^R. The forces P_f and V_f are axial force and shear force, respectively, from the column framing into the top of the joint. The moments M_f^L and M_f^R are obtained from the beams framing into the joint. The joint shear force V_u^h is calculated by resolving the moments into C and T forces. Noting that $T_L = C_L$ and $T_R = C_R$,

$$V_u^h = T_L + T_R - V_u$$

The location of C or T forces is determined by the direction of the moment. The magnitude of C or T forces is conservatively determined using basic principles of ultimate strength theory (CSA 10.1).

The moments and the forces from beams that frame into the joint in a direction that is not parallel to the major or minor direction of the column are resolved along the direction that is being investigated, thereby contributing force components to the analysis.

Also, the program calculates the positive and negative moments, considering the fact that the concrete cover may be different for the direction of moment.

In the design of Ductile moment resisting concrete frames, the evaluation of the design shear force is based on the maximum probable moment capacities (with reinforcing steel overstrength factor, α, and no φ factors) of the beams framing into the joint (CSA 21.5.1.2, 21.7.2.4.1). In the design of Moderately Ductile moment resisting frames, the evaluation of the design shear force is based on the nominal capacities (with no reinforcing steel overstrength factor, i.e., $\alpha = 1$, and no φ factors, i.e., $\varphi_c = \varphi_s = 1$ of the beams framing into the joint (CSA 21.7.2.4.1). The C and T force are based on these moment capacities. The program calculates the column shear force, V_u, from the beam moment capacities as follows:

$$V_{u,beam} = \frac{M_u^L + M_u^R}{H}$$

It should be noted that the points of inflection shown on Figure 3-4 are taken as midway between actual lateral support points for the columns. If there is no column at the top of the joint, the shear force from the top of the column is taken as zero.

The program also checks the capacity shear that can be developed in the column based on its positive and negative moment capacities for the presence of factored axial load, P_f.

$$V_{u,col} = \min\left\{\frac{M_u^{+top} + M_u^{-bot}}{H}, \frac{M_u^{-top} + M_u^{+bot}}{H}\right\}$$

V_u is taken as the minimum of the two values $V_{u,beam}$ and $V_{u,col}$.

$$V_u = \min\left\{V_{u,beam}, V_{u,col}\right\}$$

The effects of load reversals, as illustrated in Case 1 and Case 2 of Figure 3-8 are investigated, and the design is based on the maximum of the joint shears obtained from the two cases.
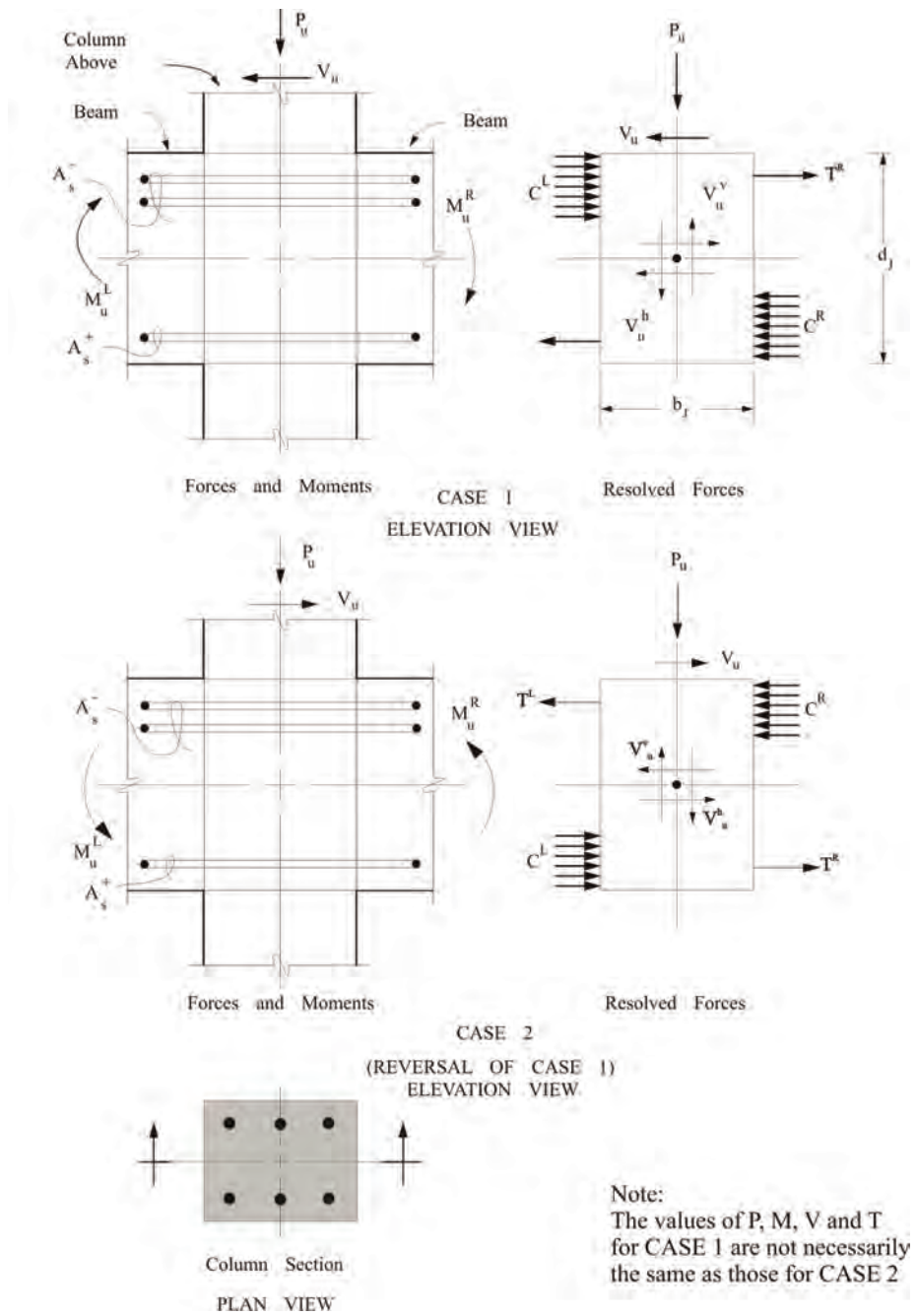
Figure 3-8 Beam-Column joint analysis

3.7.2   Determine the Effective Area of Joint

The joint area that resists the shear forces is assumed always to be rectangular in plan view. The dimensions of the rectangle correspond to the major and minor dimensions of the column below the joint, except if the beam framing into the joint is very narrow. The effective width of the joint area to be used in the calculation is limited to two times the width of the beam. The area of the joint is assumed not to exceed the area of the column below. The joint area for joint shear along the major and minor directions is calculated separately (CSA 2.3).

The effective joint shear area, A_J, is given by

$$A_J = \min\{b_{col}h_{col}, 2b_w h_{col}\} \qquad\qquad \text{(CSA 2.3)}$$

where,

b_{col} is the width of column section perpendicular to direction of shear,

h_{col} is the depth of column section in the direction of shear, and

b_w is the width of the beam or its web.

For circular column, both b_{col} and h_{col} are taken equal to the overall diameter of the column. For a section designer section, b_{col} and h_{col} are based on equivalence.

It should be noted that if the beam frames into the joint eccentrically, the preceding assumptions may not be conservative and the user should investigate the acceptability of the particular joint.

3.7.3   Check Panel Zone Shear Stress

The panel zone shear force V_u^h is compared to the factored shear resistance of the joint V_c (CSA 21.5.4.1, 21.7.2.4.2). V_c is calculated from the following equation:

$$V_c = \begin{cases} 2.2\varphi_c\sqrt{f_c'}A_J, & \text{for joints confined on all four sides,} \\ 1.6\varphi_c\sqrt{f_c'}A_J, & \text{for joints confined on three faces or on two opposite faces,} \\ 1.3\varphi_c\sqrt{f_c'}A_J, & \text{for all other joints.} \end{cases}$$

φ_c is the resistance factor for concrete. By default, it is taken as 0.65 (CSA 8.4.2). For concrete produced in a pre-qualified manufacturing plant, its value can be taken as 0.70 (CSA 16.1.3). Its value can be overwritten in the Preferences.

f_c' is the specified compressive strength of concrete. It is assumed to be the same as that of the column below the joint. The $\sqrt{f_c'}$ should not be greater than 8 MPa,

$$\sqrt{f_c'} \leq 8\text{MPa}$$

A_J is the effective joint shear area appropriate for the direction.

A beam that frames into a face of a column at the joint is considered in the program to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member (CSA 21.5.4.1, 21.7.2.4.2).

3.7.4 Beam-Column Flexural Capacity Ratios

At a particular joint for a particular column direction, major or minor, the program will calculate the ratio of the sum of the beam moment capacities to the sum of the column moment capacities. For Ductile and Moderately Ductile moment resisting frames, the following CSA provision must be satisfied (CSA 21.4.2.2, 21.7.2.2.2).

$$\sum M_c \geq \sum M_b \qquad \text{(CSA 21.4.2.2, 21.7.2.2.2)}$$

For Ductile moment resisting frames $\sum M_c$ and $\sum M_b$ are defined as follows:

- $\sum M_c$ is the sum of the nominal flexural strength of columns, M_{nc}, framing into the joint evaluated at the center of the joint. Individual column flexural strength is calculated for the associated factored axial forces, P_f. The nominal capacity is calculated with no overstrength factor, i.e., $\alpha = 1$, and no φ factors or $\varphi_c = \varphi_s = 1$.

- $\sum M_b$ is the sum of the maximum probable flexural strength of beams, M_{pb}, framing into the joint evaluated at the face of the joint. The probable capacity is calculated with an overstrength factor, i.e., $\alpha = 1.25$, and no φ factors or $\varphi_c = \varphi_s = 1$.

For Moderately Ductile moment resisting frames, $\sum M_c$ and $\sum M_b$ are defined as follows:

- $\sum M_c$ is the sum of the factored flexural strength of columns, φM_{nc}, framing into the joint evaluated at the center of the joint. Individual column flexural strength is calculated for the associated factored axial forces, P_f. The nominal capacity is calculated with no overstrength factor, i.e., $\alpha = 1$, and with appropriate φ_c and φ_s factors.

- $\sum M_b$ is the sum of the nominal flexural strength of beams, M_{pb}, framing into the joint evaluated at the face of the joint. The probable capacity is calculated with an overstrength factor, i.e., $\alpha = 1$, and no φ factors, i.e., $\varphi_c = \varphi_s = 1$.

The moment capacities of beams that frame into the joint in a direction that is not parallel to the major or minor direction of the column are resolved along the direction that is being investigated and the resolved components are added to the summation.

The column capacity summation includes the column above and the column below the joint. For each load combination, the factored axial force, P_f, in each of the columns is calculated from the load combinations. For each load combination, the moment capacity of each column under the influence of the corresponding factored axial load P_f is then determined separately for the major and minor directions of the column, using the uniaxial column interaction

diagram; see Figure 3-8. The moment capacities of the two columns are added to give the capacity summation for the corresponding load combination. The maximum capacity summations obtained from all of the load combinations is used for the beam-column capacity ratio.

The beam-column capacity ratio is determined for a beam-column joint only when the following conditions are met:

- The frame is a Ductile or Moderately Ductile moment resisting frame.

- When a column exists above the beam-column joint, it is a concrete column.

- All of the beams framing into the column are concrete beams.

- The connecting member design results are available.

- The load combination involves seismic load.

The beam-column flexural capacity ratios are reported only for Ductile and Moderately Ductile moment resisting frames involving seismic design load combinations. If this ratio is greater than 1.0, a warning message is printed in the output file.

Chapter 4
Design Output

4.1 Overview

The program creates design output in different formats – graphical display, tabular output, and member specific detailed design information.

The graphical display of design output includes input and output design information. Input design information includes design section labels, *K*-factors, live load reduction factors, and other design parameters. The output design information includes longitudinal reinforcing, shear reinforcing and column capacity ratios. All graphical output can be printed.

The tabular output can be saved in a file or printed. The tabular output includes most of the information that can be displayed. This is generated for added convenience to the designer.

The member specific detailed design information shows the details of the calculation from the designer's point of view. It shows the design forces, design section dimensions, reinforcement, and some intermediate results for all of the load combinations at all of the design sections of a specific frame member. For a column member, it also can show the position of the current state of design forces on the column interaction diagram.

In the following sections, some of the typical graphical display, tabular output, spreadsheet output, and member specific detailed design information are described. The CSA A23.3-04 design code is described in this manual.

4.2 Graphical Display of Design Information

The graphical display of design output includes input and output design information. Input design information includes design section label, K-factors, live load reduction factor, and other design parameters. The output design information includes longitudinal reinforcing, shear reinforcing, column capacity ratio, beam-column capacity ratio, joint shear check, and other design information.

The graphical output can be produced in color or in gray-scaled screen display. The active screen display can be sent directly to the printer.

4.2.1 Input and Output

Input design information for the CSA A23.3-04 code includes the following:

- Design sections

- Design framing type

- Live load reduction factors (RLLF)

- Unbraced length, L-factors, for major and minor direction of bending

- Effective length Factors, K-factors, for major and minor direction of bending

- C_m factors, for major and minor direction of bending

- δ_{ns} factors, for major and minor direction of bending

- δ_s factors, for major and minor direction of bending

The output design information that can be displayed consists of the following:

- Longitudinal reinforcing area

- Longitudinal reinforcing area as percent of concrete gross area

- Shear reinforcing areas per unit spacing

- Column P-M-M interaction ratios

- $\frac{6}{5}$ Beam-column capacity ratios

- Column-beam capacity ratios

- Joint shear capacity ratios

- Torsion reinforcing

- General reinforcing details

Use the **Design menu > Concrete Frame Design > Display Design Info** command in SAP2000/ETABS and the **Advanced > Frame Design > Concrete > Display Design Information** command in CSiBridge to plot input and output values directly on the model in the active window. Clicking this command will access the Display Design Results form. Select the Design Output or Design Input option, and then use the drop-down lists to choose the type of design data to be displayed, such as longitudinal reinforcement, rebar percentages, shear reinforcing and so on. Click the **OK** button on the form to close the form and display the selected data in the active window.

The graphical displays can be viewed in 2D or 3D mode. Use the various toolbar buttons (e.g., **Set Default 3D View**, **Set X-Y View**) to adjust the view, or use the **View menu > Set 2D View or View menu > Set 3D View** commands in SAP2000/ETABS and the **Home > View > Set 2D View** or **Set 3D View** command in CSiBridge to refine the display.

The graphical display in the active window can be printed by clicking the **File menu > Print Graphics** command in SAP2000/ETABS and the **Orb > Print > Print Graphics** command in CSiBridge, the **Print Graphics** button on the toolbar, or the Ctrl+G keyboard shortcut. The display also can be captured as a bit map file (.bmp) using one of the subcommands on the **File menu > Capture Picture** command in SAP2000/ETABS and the **Orb > Picture** command in CSiBridge, or as a metafile (.emf) using one of the subcommands on the **File menu > Capture Enhanced Metafile** command in SAP2000/ETABS and the **Orb > Picture > Metafile** command in CSiBridge. The captured picture file can then be used in popular graphics programs, including Paint and Power-

Point. Alternatively, the standard Windows screen capture command (click the **Print Screen** button on the keyboard) can be used to create a screen capture of the entire window, or use the **Alt+Print Screen** command to capture only the "top layer," such as a form displayed from within the program.

By default, graphics are displayed and printed in color, assuming a color printer is available. Use the **Options menu > Colors > Output** command in SAP2000/ETABS and the **Orb > Settings > Colors** command in CSiBridge to change default colors, as necessary, including changing the background color from the default black to white. A white background can be useful when printing design output to save ink/toner. In addition, the **Options menu > Colors > Set Active Theme** command can be used to view or print graphics in grayscale in SAP2000/ETABS.

4.3   Tabular Display of Design Output

The tabular design output can be sent directly to a printer or saved to a file. The printed form of the tabular output is the same as that produced for the file output except that the font size is adjusted for the printed output.

 The tabular design output includes input and output design information that depends on the design code chosen. For the CSA A23.3-04 code, the tabular output includes the following. All tables have formal headings and are self-explanatory, so further description of these tables is not given.

Input design information includes the following:

- Concrete Column Property Data
    – Material label
    – Column dimensions
    – Reinforcement pattern
    – Concrete cover
    – Bar area

- Concrete Beam Property Data
    – Material label
    – Beam dimensions
    – Top and bottom concrete cover

- Top and bottom reinforcement areas

- Concrete Column Property Data
    - Material label
    - Column dimensions
    - Reinforcement pattern
    - Concrete cover
    - Bar area

- Load Combination Multipliers
    - Combination name
    - Load types
    - Load factors

- Concrete Design Element Information
    - Design section ID
    - Factors for major and minor direction of bending
    - Unbraced length ratios for major and minor direction of bending, *L*-factors
    - Live load reduction factors (RLLF)

- Concrete Moment Magnification Factors
    - Section ID
    - Element Type
    - Framing Type
    - δ_{ns} -factors
    - δ_s -factors

The output design information includes the following:

- Column design Information
    - Section ID
    - Station location
    - Total longitudinal reinforcement and the governing load combination
    - Major shear reinforcement and the governing load combination
    - Minor shear reinforcement and the governing load combination

- Beam Design Information
  - Section ID
  - Station location
  - Top longitudinal reinforcement and the governing load combination
  - Bottom reinforcement and the governing load combination
  - Longitudinal torsional reinforcement and the governing load combination
  - Major shear reinforcement and the governing load combination for shear and torsion design

- Concrete Column Joint Information
  - Section ID
  - (6/5) Beam/column capacity ratios for major and minor direction and the governing load combination
  - Joint shear capacity for major and minor direction and the governing load combination

Tabular output can be printed directly to a printer or saved in a file using the **File menu > Print Tables** command in SAP/ETABS and the **Orb > Print > Print Tables** command in CSiBridge. A form will display when this command is used. Depress the F1 key on the keyboard to access the Help topic specific to that form, which will identify the types of output available (e.g., plain text with or without page breaks, rich text format Word document, and so on).

4.4   Member Specific Information

Member specific design information shows the details of the calculation from the designer's point of view. It includes the geometry and material data, other input data, design forces, design section dimensions, reinforcement details, and some of the intermediate results for the selected member. The design detail information can be displayed for a specific load combination and for a specific station of a column or beam member. For columns, member specific design information also can show the position of the current state of design forces using a column interaction diagram.

After an analysis has been performed and the **Design menu > Concrete Frame Design > Start Design/Check** command in SAP2000/ETABS and the

Advanced > Frame Design > Concrete > Start Design/Check of Structure command in CSiBridge has been used, access the detailed design information by right clicking a frame member to display the Concrete Column Design Information form if a column member was right clicked or the Concrete Beam Design Information form if a beam member was right clicked. Table 4-1 identifies the types of data provided by the forms.

The longitudinal and shear reinforcing area are reported in their current units, which are displayed in the drop-down list in the lower right corner of the program window. Typically, the longitudinal reinforcing area is reported in in^2, mm^2, cm^2 and so on. Shear reinforcing areas typically are reported in in^2/in, mm^2/mm, cm^2/cm and so on.

Table 4-1 Member Specific Data for Columns and Beams

Column	Beam
▪ Load combination ID ▪ Station locations ▪ Longitudinal reinforcement area ▪ Major shear reinforcement areas ▪ Minor shear reinforcement areas	▪ Load combination ID ▪ Station location ▪ Top reinforcement areas ▪ Bottom reinforcement areas ▪ Longitudinal reinforcement for torsion design ▪ Shear reinforcement area for shear ▪ Shear reinforcement area for torsion design
Buttons on the forms can be used to access additional forms that provide the following data	

▪ Overwrites
  – Element section ID
  – Element framing type
  – Code-dependent factors
  – Live load reduction factors
  – Effective length factors, K, for major and minor direction bending
  – C_m factors for major and minor bending
  – δ_s factors for major and minor directions
 	
▪ Summary design data
  – Geometric data and graphical representation
  – Material properties
  – Minimum design moments
  – Moment factors
  – Longitudinal reinforcing areas
  – Design shear forces
  – Shear reinforcing areas
	▪ Overwrites
	  – Element section ID
	  – Element framing type
	  – Code-dependent factors
	  – Live load reduction factors
	  – Effective length factors, K, for major and minor direction bending
	  – C_m factors for major and minor bending
	  – δ_s factors for major and minor directions
	
	▪ Summary design data
	  – Geometric data and graphical representation
	  – Material properties
	  – Design moments and shear forces
	  – Minimum design moments
	  – Top and bottom reinforcing areas
	  – Shear capacities of concrete and steel
	  – Shear reinforcing area
	  – Torsion reinforcing area

Table 4-1 Member Specific Data for Columns and Beams

– Shear capacities of steel and concrete – Torsion reinforcing – Interaction diagram, with the axial force and biaxial moment showing the state of stress in the column ▪ Detailed calculations for flexural details, shear details, joint shear, and beam/ column capacity ratios

The load combination is reported by its name, while station data is reported by its location as measured from the I-end of the column. The number of line items reported is equal to the number of design combinations multiplied by the number of stations. One line item will be highlighted when the form first displays. This line item will have the largest required longitudinal reinforcing, unless any design overstress or error occurs for any of the items. In that case, the last item among the overstressed items or items with errors will be highlighted. In essence, the program highlights the critical design item.

If a column has been selected and the column has been specified to be checked by the program, the form includes the same information as that displayed for a designed column, except that the data for a checked column includes the capacity ratio rather than the total longitudinal reinforcing area. Similar to the design data, the line item with the largest capacity ratio is highlighted when the form first displays, unless an item has an error or overstress, in which case, that item will be highlighted. In essence, the program highlights the critical check item.

The program can be used to check and to design rebar in a column member. When the users specifies that the program is to check the rebar in the column, the program checks the rebar as it is specified. When the user specifies that the program design the rebar configuration, the program starts with the data specified for rebar and then increases or decreases the rebar in proportion to the relative areas of rebar at the different locations of rebar in the column.

4.4.1   Interactive Concrete Frame Design

The interactive concrete frame design and review is a powerful mode that allows the user to review the design results for any concrete frame design, to revise the design assumptions interactively, and to review the revised results immediately.

Before entering the interactive concrete frame design mode, the design results must be available for at least one member. That means the design must have been run for all the members or for only selected members. If the initial design is not been performed yet, run a design by clicking the **Design menu > Concrete Frame Design > Start Design/Check of Structure** command in SAP2000/ETABS and **Advanced > Frame Design > Concrete > Start Design/Check of Structure** command in CSiBridge.

There are three ways to initiate the interactive concrete frame design mode:

- Click the **Design menu > Concrete Frame Design > Start Design/Check of Structures** command in SAP2000/ETABS and **Advanced > Frame Design > Concrete > Start Design/Check of Structure** command in CSiBridge to run a design.

- Click the **Design menu > Concrete Frame Design > Display Design Info** command in SAP2000/ETABS and the **Advanced > Frame Design > Concrete > Display Design Information** command in CSiBridge to access the Display Design Results form and select a type of result.

- Click the **Design menu > Concrete Frame Design > Interactive Concrete Frame Design** command in SAP2000/ETABS and the **Advanced > Frame Design > Concrete > Interactive Concrete Frame Design** command in CSiBridge.

After using any of the three commands, right click on a frame member to enter the interactive Concrete Frame Design Mode and access the Concrete Column Design Information form if a column member was right clicked or the Concrete Beam Design Information form if a beam member was right clicked. These forms have **Overwrites** buttons that accesses the Concrete Frame Design Overwrites form. The form can be used to change the design sections, element type, live load reduction factor for reducible live load, and many other design factors. See Appendix D for a detailed description of the overwrite items. When changes to the design parameters are made using the Overwrites form, the Concrete Beam or Column Design Information forms update immediately to reflect the changes. Then other buttons on the Concrete Beam or Column Design Information forms can be used to display additional forms showing the details of the updated design. See the *Member Specific Information* section of this chapter for more information.

In this way, the overwrites can be changed any number of times to produce a satisfactory design. After an acceptable design has been produced by changing this section or other design parameters, click the **OK** button on the Concrete Beam or Column Design Information forms to permanently change the design sections and other overwrites for that member. However, if the **Cancel** button is used, all changes made to the design parameters using the Concrete Frame Design Overwrites form are temporary and do not affect the design.

4.5   Error Messages and Warnings

In many places of concrete frame design output, error messages and warnings are displayed. The messages are numbered. A complete list of error messages and warnings used in Concrete Frame Design for all the design codes is provided in Appendix E. However, all of the messages are not applicable to CSA A23.3-04 code.

APPENDICES

Appendix A
Second Order P-Delta Effects

Typically, design codes require that second order P-delta effects be considered when designing concrete frames. They are the global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure A-1, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by Δ. The local deformation of the member is shown as δ. The total second order P-delta effects on this frame object are those caused by both Δ and δ.

The program has an option to consider P-delta effects in the analysis. When P-delta effects are considered in the analysis, the program does a good job of capturing the effect due to the Δ deformation shown in Figure A-1, but it does not typically capture the effect of the δ deformation (unless, in the model, the frame object is broken into multiple elements over its length).
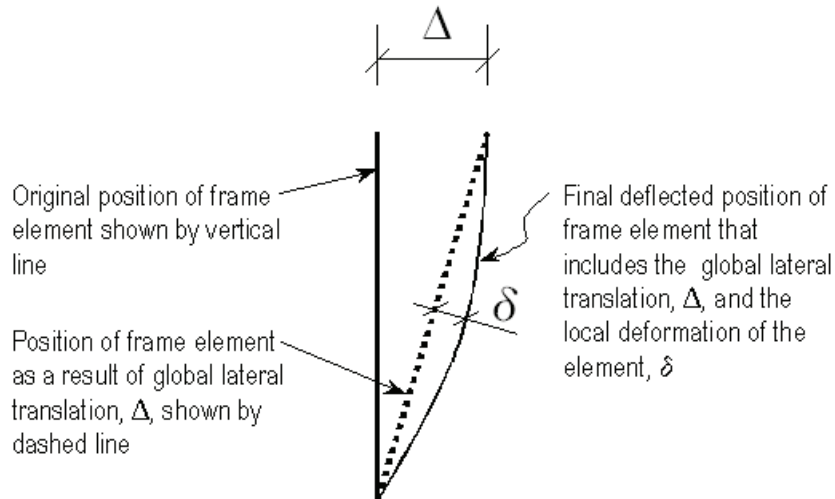
Figure A-1 The Total Second Order P-Delta Effects on a
Frame Element Caused by Both Δ and δ

Consideration of the second order P-delta effects are generally achieved by computing the flexural design capacity using a formula similar to that shown in the following equation.

$$M_{CAP} = aM_{nt} + bM_{lt} \qquad \text{where,}$$

M_{CAP} = Flexural design capacity required

M_{nt} = Required flexural capacity of the member assuming there is no joint translation of the frame (i.e., associated with the δ deformation in Figure A-1)

M_{lt} = Required flexural capacity of the member as a result of lateral translation of the frame only (i.e., associated with the Δ deformation in Figure A-1)

a = Unitless factor multiplying M_{nt}

b = Unitless factor multiplying M_{lt} (assumed equal to 1 by the program; see below)

When the program performs concrete frame design, it assumes that the factor b is equal to 1 and calculates the factor a. That $b = 1$ assumes that P-delta effects have been considered in the analysis, as previously described. Thus, in general, when performing concrete frame design in this program, **consider P-delta effects in the analysis before running the program**.

Appendix B
Member Unsupported Lengths and
Computation of K-Factors

The column unsupported lengths are required to account for column slenderness effects. The program automatically determines the unsupported length ratios, which are specified as a fraction of the frame object length. Those ratios times the frame object length gives the unbraced lengths for the members. Those ratios can also be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

There are two unsupported lengths to consider. They are L_{33} and L_{22}, as shown in Figure B-1. These are the lengths between support points of the member in the corresponding directions. The length L_{33} corresponds to instability about the 3-3 axis (major axis), and L_{22} corresponds to instability about the 2-2 axis (minor axis).
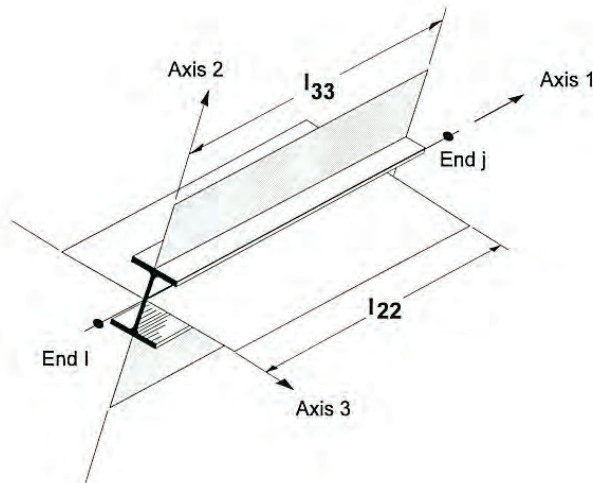
Figure B-1 Axis of bending and unsupported length

In determining the values for L_{22} and L_{33} of the members, the program recognizes various aspects of the structure that have an effect on these lengths, such as member connectivity, diaphragm constraints, and support points. The program automatically locates the member support points and evaluates the corresponding unsupported length.

It is possible for the unsupported length of a frame object to be evaluated by the program as greater than the corresponding member length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In that case, the column is assumed to be supported in one direction only at that story level, and its unsupported length in the other direction will exceed the story height.

Appendix C
Concrete Frame Design Preferences

The concrete frame design preferences are general assignments that are applied to all of the concrete frame members. The design preferences should be reviewed and any changes from the default values made before performing a design. The following table lists the design preferences that are specific to using CSA A23.3-04; the preferences that are generic to all codes are not included in this table.

Table C-1 Preferences

Item	Possible Values	Default Value	Description
Time History Design	Envelopes, Step-by-Step	Envelopes	Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has *more than one* time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.

Item	Possible Values	Default Value	Description
Number Interaction Curves	Multiple of 4 ≥ 4	24	Number of equally spaced interaction curves used to create a full 360 *deg* interaction surface (this item should be a multiple of four). We recommend 24 for this item.
Number	Any odd value ≥ 5	11	Number of points used for defining a single curve in a concrete frame; should be odd
Consider Minimum Eccentricity	No, Yes	Yes	Toggle to consider if minimum eccentricity is considered in design.
Phi (Steel)	> 0	0.85	Strength reduction factor for rebar
Phi (Concrete)	> 0	0.6	The strength reduction factor for concrete
Pattern Live Load Factor	≥ 0	0.75	The scale factor for performing pattern loading for live loads
Utilization Factor Limit	> 0	0.95	Stress ratios that are less than or equal to this value are considered acceptable

Appendix D
Concrete Frame Overwrites

The concrete frame design overwrites are basic assignments that apply only to those elements to which they are assigned. Table D-1 lists concrete frame design overwrites for CSA A23.3-04. Default values are provided for all overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values to ensure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned.

Table D-1 Overwrites

Item	Possible Values	Default Value	Description
Current Design Section	Any defined concrete section	Analysis section	The design section for the selected frame objects. When this overwrite is applied, any previous auto select section assigned to the frame object is removed.
Element Type	Ductile Moderately Ductile, Conventional	From Reference	Frame type per moment frame definition given in CSA 2.2. The program default is Ductile, which the user can overwrites if needed.
Live Load	≤ 0	Calculated	The reduced live load factor. A reduci-

Item	Possible Values	Default Value	Description
Reduction Factor			ble live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying 0 means the value is program determined.
Unbraced Length Ratio (Major)	≥ 0	Calculated	Unbraced length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined.
Unbraced Length Ratio (Minor)	≥ 0	Calculated	Unbraced length factor for buckling about the frame object minor axis. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined. This factor is also used in determining the length for lateral-torsional buckling.
Effective Length Factor (K Major)	> 0	Calculated	See CSA, 10.15.1. Effective length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length.
Effective Length Factor (K Minor)	> 0	Calculated	See CSA, 10.15.1. Effective length factor for buckling about the frame object minor axis. This item is specified as a fraction of the frame object length.
Moment Coefficient (C_m Major)	> 0	Calculated	See CSA 10.15.3.2. Factor relating actual moment diagram to an equivalent uniform moment diagram about the frame object major axis.
Moment Coefficient (C_m Minor)	> 0	Calculated	See CSA 10.15.3.2. Factor relating actual moment diagram to an equivalent uniform moment diagram about the frame object minor axis.
NonSway Moment Factor (D_b major)	> 0	Calculated	See CSA 10.15.3.1. The magnified non-sway moment factor amplified for the effects of member curvature about the frame object major axis.

Item	Possible Values	Default Value	Description
NonSway Moment Factor (D_b minor)	> 0	Calculated	See CSA 10.15.3.1. The magnified non-sway moment factor amplified for the effects of member curvature about the frame object minor axis.
Sway Moment Factor (D_s major)	1	From Reference	See CSA 10.16.3.1. The magnified sway moment factor shall be taken as the column end moments calculated using a second-order analysis about the frame object major axis.
Sway Moment Factor (D_s minor)	1	From Reference	See CSA 10.16.3.1. The magnified sway moment factor shall be taken as the column end moments calculated using a second-order analysis about the frame object minor axis.

Appendix E
Error Messages and Warnings

Table E-1 provides a complete list of Concrete Errors messages and Warnings.

Table E-1 Error Messages

Error Number	Description
1	Beam concrete compression failure
2	Reinforcing required exceeds maximum allowed
3	Shear stress exceeds maximum allowed
4	Column design moments cannot be calculated
5	Column factored axial load exceeds Euler Force
6	Required column concrete area exceeds maximum
7	Flexural capacity could not be calculated for shear design
8	Concrete column supports non-concrete beam/column

Error Number	Description
9	$k*L/r > 115$, $zeta\_2 < 0$, $eta < 1.0$ (GB50010 7.3.10)
10	Column is overstressed for P-M-M
11	Axial compressive capacity for concrete exceeded (TBM 6.4.2)
12	Beam frames into column eccentrically (11.6.3)
13	Torsion exceeds maximum allowed
14	Reinforcing provided is below minimum required
15	Reinforcing provided exceeds maximum allowed
16	Tension reinforcing provided is below minimum required
17	$k*L/r > 30$ (GB 7.3.10)
21	The column is not ductile. Beam/column capacity ratio is not needed.
22	The load is not seismic. Beam/column capacity ratio is not needed.
23	There is no beam on top of column. Beam/column capacity ratio is not needed.
24	At least one beam on top of column is not of concrete. Beam/column capacity ratio is not calculated.
25	The column on top is not of concrete. Beam/column capacity ratio is not calculated.
26	The station is not at the top of the column. Beam/column capacity ratio is not needed.
27	The column is not ductile. Joint shear ratio is not needed.

Error Number	Description
28	The load is not seismic. Joint shear ratio is not needed.
29	There is no beam on top of column. Joint shear ratio is not needed.
30	At least one beam on top of column is not of concrete. Joint shear ratio is not calculated.
31	The column on top is not of concrete. Joint shear ratio is not needed.
32	The station is not at the top of the column. Joint shear ratio is not needed.
33	Beam/column capacity ratio exceeds limit.
34	Joint shear ratio exceeds limit.
35	Capacity ratio exceeds limit.
36	All beam s around the joint have not been designed. Beam/column capacity ratio is not calculated.
37	At least one beam around the joint have failed. Beam/column capacity ratio is not calculated.
38	The column above the joint have not been designed. Beam/column capacity ratio is not calculated.
39	The column above the joint have failed. Beam/column capacity ratio is not calculated.
40	All beams around the joint have not been designed. Joint shear ratio is not calculated.
41	At least one beam around the joint have failed. Joint shear ratio is not calculated.
42	The column above the joint has not been designed. Joint shear

Error Number	Description
	ratio is not calculated.
43	The column above the joint has failed. Joint shear ratio is not calculated.
45	Shear stress due to shear force and torsion together exceeds maximum allowed.

References

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