

SAP2000®

**Integrated
Finite Element Analysis
and
Design of Structures**

CONCRETE DESIGN MANUAL



Computers and Structures, Inc.
Berkeley, California, USA

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THIS PROGRAM IS A VERY PRACTICAL TOOL FOR THE DESIGN OF REINFORCED CONCRETE STRUCTURES. HOWEVER, THE USER MUST THOROUGHLY READ THE MANUAL AND CLEARLY RECOGNIZE THE ASPECTS OF REINFORCED CONCRETE DESIGN THAT THE PROGRAM ALGORITHMS DO NOT ADDRESS.

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

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Chapter I

Introduction

Overview

SAP2000 features powerful and completely integrated modules for design of both steel and reinforced concrete structures (CSI 1998a, 1998b, 1997). The program provides the user with options to create, modify, analyze and design structural models, all from within the same user interface.

The program provides an interactive environment in which the user can study the stress conditions, make appropriate changes, such as member size revisions, and update the design without re-analyzing the structure. A single mouse click on an element brings up detailed design information. Members can be grouped together for design purposes. The output in both graphical and tabulated formats can be readily displayed and printed.

The program is structured to support a wide variety of design codes for the automated design and check of concrete frame members. The program currently supports the following design codes: U.S. (ACI 1999, AASHTO 1997), Canadian (CSA 1994), British (BSI 1989), European (CEN 1992), and New Zealand (NZS 3101-95).

The design is based upon a set of user-specified loading combinations. However, the program provides a set of default load combinations for each design code sup-

ported in SAP2000. If the default load combinations are acceptable, no definition of additional load combinations are required.

In the design of the 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.

Every beam member is designed for flexure and shear at a user defined number of stations along the beam span.

The presentation of the output is clear and concise. The information is in a form that allows the engineer to take appropriate remedial measures in the event of member overstress. Backup design information produced by the program is also provided for convenient verification of the results.

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

Organization

This manual is organized in the following way:

Chapter II outlines various aspects of the concrete design procedures of the SAP2000 program. This chapter describes the common terminology of concrete design as implemented in SAP2000.

Each of six subsequent chapters gives a detailed description of a specific code of practice as interpreted by and implemented in SAP2000. Each chapter describes the design loading combination, column and beam design procedures, and other special consideration required by the code.

Chapter III gives a detailed description of the ACI code (ACI 1999) as implemented in SAP2000.

Chapter IV gives a detailed description of the AASHTO LRFD concrete code (AASHTO 1997) as implemented in SAP2000.

Chapter V gives a detailed description of the Canadian code (CSA 1994) as implemented in SAP2000.

Chapter VI gives a detailed description of the British code (BSI 1989) as implemented in SAP2000.

Chapter VII gives a detailed description of the Eurocode 2 (CEN 1992) as implemented in SAP2000.

Chapter VIII gives a detailed description of the New Zealand code (NZS 1997) as implemented in SAP2000.

Chapter IX outlines various aspects of the tabular and graphical output from SAP2000 related to concrete design.

Recommended Reading

It is recommended that the user read Chapter II “Design Algorithms” and one of six subsequent chapters corresponding to the code of interest to the user. Finally the user should read “Design Output” in Chapter IX for understanding and interpreting SAP2000 output related to concrete design.

A concrete design tutorial is presented in the chapter “Concrete Design Tutorial” in the *SAP2000 Quick Tutorial* manual. It is recommended that first time users follow through the steps of this tutorial before reading this manual.

Chapter II

Design Algorithms

This chapter outlines various aspects of the concrete design and design-check procedures that are used by the SAP2000 program. The concrete design and check may be performed in SAP2000 according to one of the following design codes:

- The 1995 American Concrete Institute Building Code Requirements for Structural Concrete, **ACI 318-99** (ACI 1999).
- The 1997 American Association of State Highway and Transportation Officials AASHTO LRFD Bridge Design Specifications, **AASHTO LRFD 1997** (AASHTO 1997).
- The 1994 Canadian Standards Association Design of Concrete Structures for Buildings, **CSA-A23.3-94** (CSA 1994).
- The 1989 British Standards Institution Structural Use of Concrete, **BS 8110-85 R1989** (BSI 1989).
- The 1992 European Committee for Standardization, Design of Concrete Structures, **EUROCODE 2** (CEN 1992).
- The 1995 Standards New Zealand Concrete Structures Standard, **NZS 3101-95** (NZS 1995).

Details of the algorithms associated with each of these codes as implemented in SAP2000 are described in the subsequent chapters. However, this chapter provides a background which is common to all the design codes.

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 at least one of the above mentioned design codes.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code. For example, all references to the AASHTO code are preceded by the word “AASHTO”. Similarly,

- References to the ACI 318-99 code has the prefix of “ACI”
- References to the Canadian code carry the prefix of “CSA”
- References to the British code carry the prefix of “BS”
- References to the Eurocode 2 carry the prefix of “EC2”
- References to the New Zealand code carry the prefix of “NZS”

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, and/or dynamic response spectrum earthquake load the program has built-in default loading combinations for each design code. These are based on the code recommendations and are documented for each code in the corresponding chapters.

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

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

The default load combinations are included in design if the user requests them to be included or if no other user defined combination is available for concrete design. If any default combination is included in design, then all default combinations will automatically be updated by the program any time the 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 the frame members, then the effects of these loads will be assumed to be zero in any combination that includes them.

Design and Check Stations

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

Identifying Beams and Columns

Since SAP2000 is a general purpose analysis and design 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 done by specifying the frame section assigned to the element to be of type beam or column.

Design of Beams

In the design of concrete beams, in general, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon 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, minor direction bending, and torsion 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 M_u^+ and maximum negative M_u^- 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- 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, then 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 in SAP2000 for ACI, Canadian, and New Zealand codes.

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. A typical interaction surface is shown in Figure II-1.

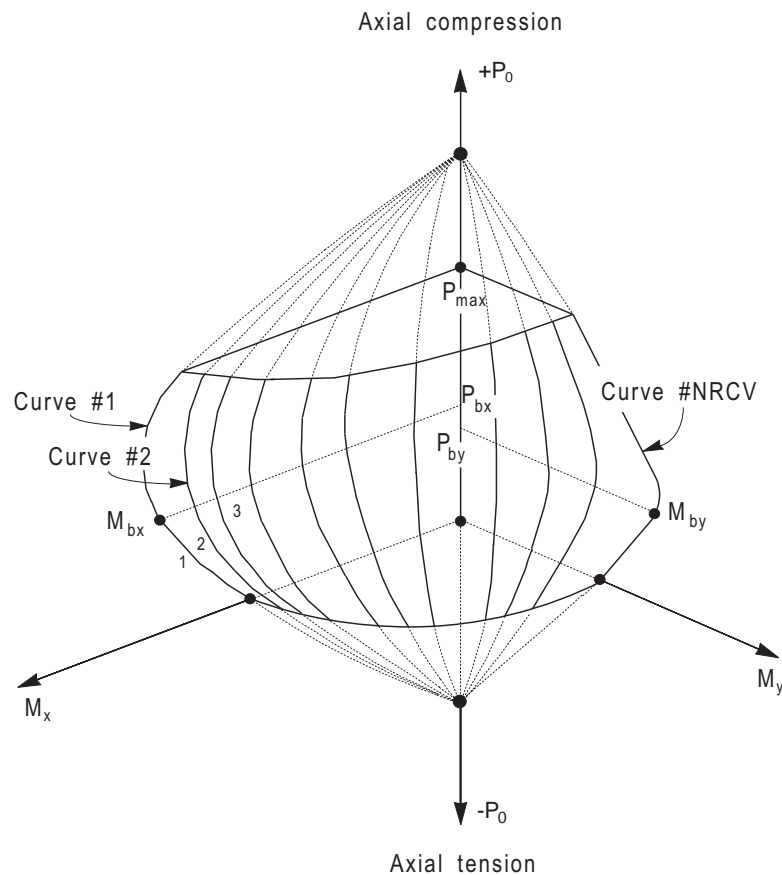


Figure II-1
A Typical Column Interaction Surface

- 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.
- Design the column shear reinforcement.

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 vary from code to code. A typical assumed strain distribution is described in Figure II-2.

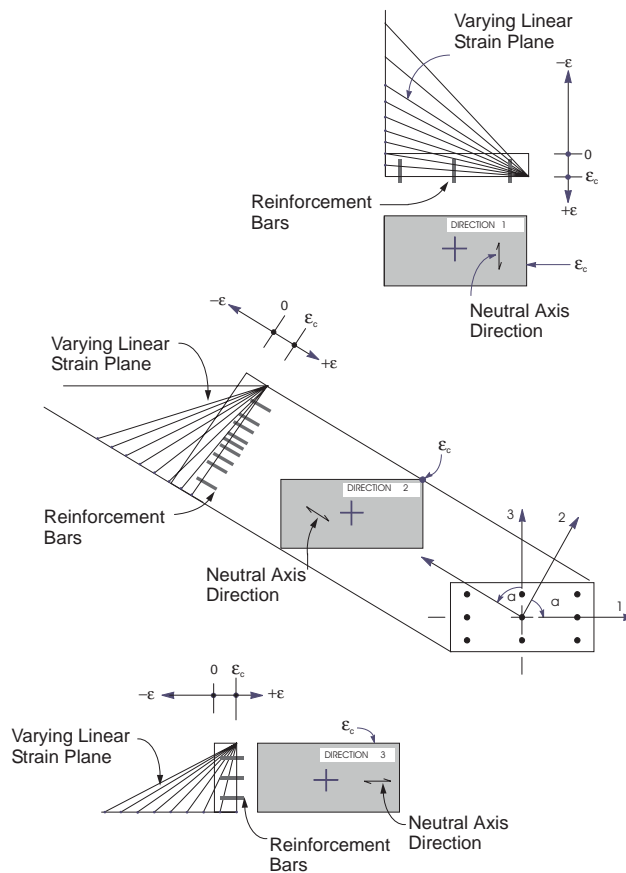


Figure II-2
Idealized Strain Distribution for Generation of Interaction Surfaces

Here maximum compression strain is limited to ϵ_c . For most of the design codes, this assumed distribution remains valid. However, the value of ϵ_c varies from code to code. For example, $\epsilon_c = 0.003$ for ACI, AASHTO and New Zealand codes, and $\epsilon_c = 0.0035$ for Canadian, British and European codes. The details of the generation of interaction surfaces differ from code to code. These are described in the chapters specific to the code.

The capacity check is based on whether the design load points lie inside the interaction volume in a force space, as shown in Figure II-3. If the point lies inside the volume, the column capacity is adequate, and vice versa.

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 needs to be considered.

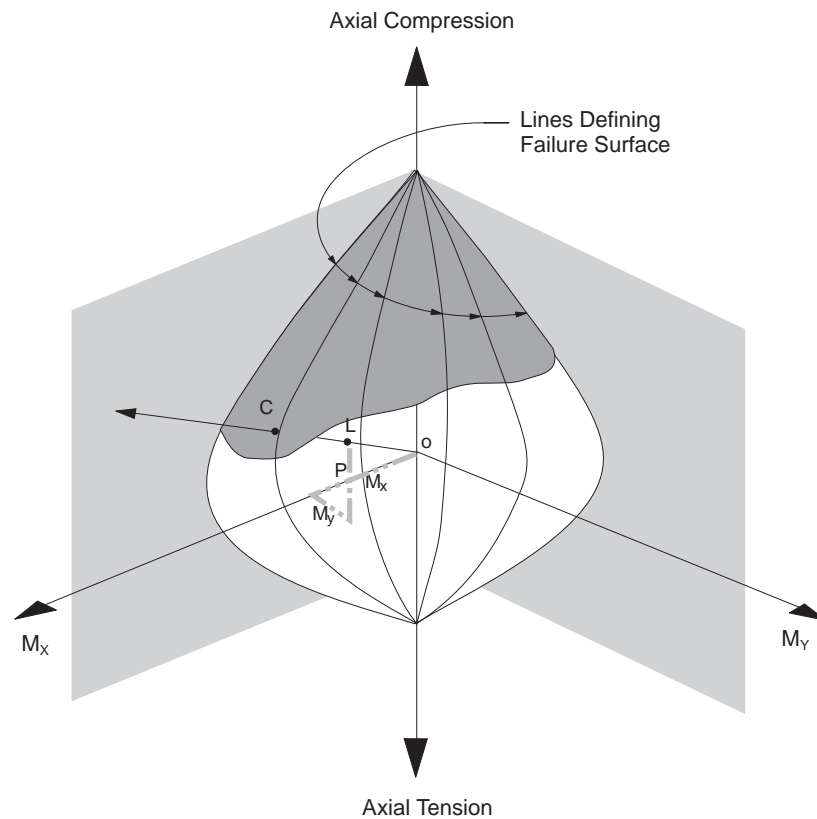


Figure II-3
Geometric Representation of Column Capacity Ratio

P- Δ Effects

The SAP2000 design algorithms require that the analysis results include the P- Δ effects. The P- Δ effects are considered differently for “braced” or “nonsway” and “unbraced” or “sway” components of moments in columns or frames. For the braced moments in columns, the effect of P- Δ is limited to “individual member stability”. For unbraced components, “lateral drift effects” should be considered in addition to individual member stability effect. In SAP2000, it is assumed that “braced” or “nonsway” moments are contributed from the “dead” or “live” loads. Whereas, “unbraced” or “sway” moments are contributed from all other types of loads.

For the individual member stability effects, the moments are magnified with moment magnification factors as in the ACI, AASHTO, Canadian, and New Zealand codes or with additional moments as in the British and European codes.

For lateral drift effects, SAP2000 assumes that the P- Δ analysis is performed and that the amplification is already included in the results. The moments and forces obtained from P- Δ analysis are further amplified for individual column stability effect if required by the governing code as in the ACI, Canadian, and New Zealand codes.

The users of SAP2000 should be aware that the default analysis option in SAP2000 is turned OFF for P- Δ effect. The user can turn the P- Δ analysis ON and set the maximum number of iterations for the analysis. The default number of iteration for P- Δ analysis is 1. For further reference, the user is referred to *SAP2000 Analysis Reference* manual.

The user is also cautioned that SAP2000 currently considers P- Δ effects due to axial loads in frame members only. Forces in other types of elements do not contribute to this effect. If significant forces are present in other type of elements, for example, huge axial loads in shear walls which are modeled as shell elements, then the additional forces computed for P- Δ will not be accurate.

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

Normally, the unsupported element length is equal to the length of the element, i.e., the distance between END-I and END-J of the element. See Figure II-4. The program, however, allows users to assign several elements to be treated as a single member for design. This can be done differently for major and minor bending. Therefore, extraneous joints, as shown in Figure II-5, that affect the unsupported length of an element are automatically taken into consideration.

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

Therefore, the unsupported length of a column may actually be evaluated as being greater than the corresponding element length. If the beam frames into only one direction of the column, the beam is assumed to give lateral support only in that direction.

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

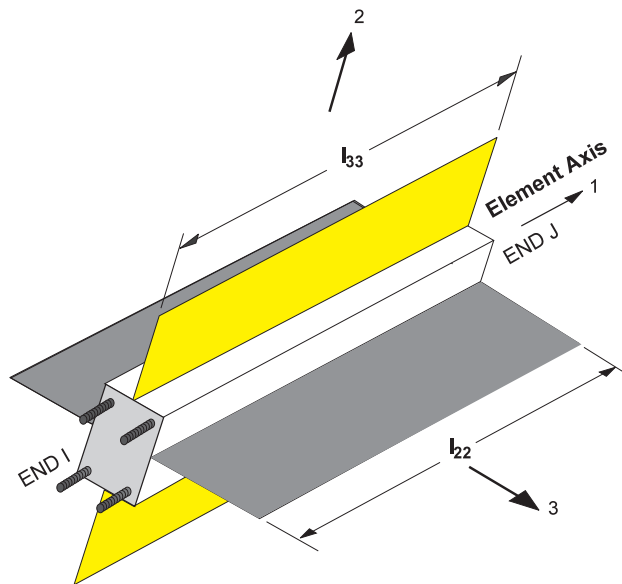


Figure II-4
Axes of Bending and Unsupported Length

Special Considerations for Seismic Loads

The ACI code imposes a special ductility requirement for frames in seismic regions by specifying frames either as Ordinary, Intermediate, or Special moment resisting frames. The Special moment resisting frame can provide the required ductility and energy dissipation in the nonlinear range of cyclic deformation. The AASHTO code requires that the concrete frame must be in either Zone 1, Zone 2, Zone 3, or Zone 4, where Zone 4 is designated as the zone of severe earthquake. The Canadian code requires that the concrete frame must be designed as either an Ordinary, Nominal, or Ductile moment resisting frame. The New Zealand code also requires that the concrete frame must be designed as either an Ordinary, Elastically responding, frames with Limited ductility, or Ductile moment resisting frame.

Unlike the ACI, AASHTO, Canadian, and New Zealand codes, the current implementation of the British code and the Eurocode 2 in SAP2000 does not account for any special requirements for seismic design.

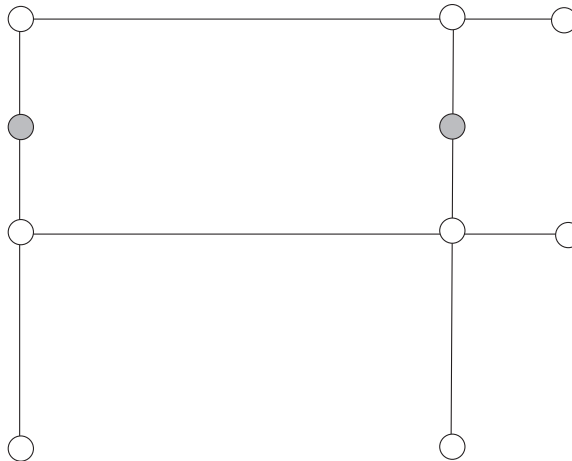


Figure II-5
Unsupported Lengths and Interior Nodes

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 ACI code is published in inch-pound-second units. By default, all equations and descriptions presented in the chapter “Design for ACI 318-99” correspond to inch-pound-second units. However, any system of units can be used to define and design the structure in SAP2000.

Chapter III

Design for ACI 318-99

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the ACI 318-99 Design Code (ACI 1999). Various notations used in this chapter are listed in Table III-1.

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

SAP2000 provides options to design or check Ordinary, Intermediate (moderate seismic risk areas), and Special (high seismic risk areas) moment resisting frames as required for seismic design provisions. The details of the design criteria used for the different framing systems are described in the following sections.

English as well as SI and MKS metric units can be used for input. But the code is based on Inch-Pound-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Inch-Pound-Second** units unless otherwise noted.

Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs to be checked. For the ACI 318-99 code, if a

A_{cv}	Area of concrete used to determine shear stress, sq-in
A_g	Gross area of concrete, sq-in
A_s	Area of tension reinforcement, sq-in
A'_s	Area of compression reinforcement, sq-in
$A_{s(required)}$	Area of steel required for tension reinforcement, sq-in
A_{st}	Total area of column longitudinal reinforcement, sq-in
A_v	Area of shear reinforcement, sq-in
a	Depth of compression block, in
a_b	Depth of compression block at balanced condition, in
b	Width of member, in
b_f	Effective width of flange (T-Beam section), in
b_w	Width of web (T-Beam section), in
C_m	Coefficient, dependent upon column curvature, used to calculate moment magnification factor
c	Depth to neutral axis, in
c_b	Depth to neutral axis at balanced conditions, in
d	Distance from compression face to tension reinforcement, in
d'	Concrete cover to center of reinforcing, in
d_s	Thickness of slab (T-Beam section), in
E_c	Modulus of elasticity of concrete, psi
E_s	Modulus of elasticity of reinforcement, assumed as 29,000,000 psi
f'_c	Specified compressive strength of concrete, psi
f_y	Specified yield strength of flexural reinforcement, psi
f_{ys}	Specified yield strength of shear reinforcement, psi
h	Dimension of column, in
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in ⁴
I_{se}	Moment of inertia of reinforcement about centroidal axis of member cross section, in ⁴
k	Effective length factor
L	Clear unsupported length, in

Table III-1

List of Symbols Used in the ACI code

M_1	Smaller factored end moment in a column, lb-in
M_2	Larger factored end moment in a column, lb-in
M_c	Factored moment to be used in design, lb-in
M_{ns}	Nonsway component of factored end moment, lb-in
M_s	Sway component of factored end moment, lb-in
M_u	Factored moment at section, lb-in
M_{ux}	Factored moment at section about X-axis, lb-in
M_{uy}	Factored moment at section about Y-axis, lb-in
P_b	Axial load capacity at balanced strain conditions, lb
P_c	Critical buckling strength of column, lb
P_{max}	Maximum axial load strength allowed, lb
P_0	Axial load capacity at zero eccentricity, lb
P_u	Factored axial load at section, lb
r	Radius of gyration of column section, in
V_c	Shear resisted by concrete, lb
V_E	Shear force caused by earthquake loads, lb
V_{D+L}	Shear force from span loading, lb
V_u	Factored shear force at a section, lb
V_p	Shear force computed from probable moment capacity, lb
α	Reinforcing steel overstrength factor
β_1	Factor for obtaining depth of compression block in concrete
β_d	Absolute value of ratio of maximum factored axial dead load to maximum factored axial total load
δ_s	Moment magnification factor for sway moments
δ_{ns}	Moment magnification factor for nonsway moments
ϵ_c	Strain in concrete
ϵ_s	Strain in reinforcing steel
ϕ	Strength reduction factor

Table III-1*List of Symbols Used in the ACI code (continued)*

structure is subjected to dead load (DL) and live load (LL) only, the stress check may need only one load combination, namely 1.4 DL + 1.7 LL (ACI 9.2.1). However, in addition to the dead and live loads, if the structure is subjected to wind (WL) and earthquake (EL) loads, and considering that wind and earthquake forces are reversible, then the following load combinations have to be considered (ACI 9.2).

$$1.4 \text{ DL}$$
$$1.4 \text{ DL} + 1.7 \text{ LL} \quad (\text{ACI 9.2.1})$$

$$0.9 \text{ DL} \pm 1.3 \text{ WL}$$
$$0.75 (1.4 \text{ DL} + 1.7 \text{ LL} \pm 1.7 \text{ WL}) \quad (\text{ACI 9.2.2})$$

$$0.9 \text{ DL} \pm 1.3 * 1.1 \text{ EL}$$
$$0.75 (1.4 \text{ DL} + 1.7 \text{ LL} \pm 1.7 * 1.1 \text{ EL}) \quad (\text{ACI 9.2.3})$$

These are also the default design load combinations in SAP2000 whenever the ACI 318-99 code is used.

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

Strength Reduction Factors

The strength reduction factors, ϕ , are applied on the nominal strength to obtain the design strength provided by a member. The ϕ factors for flexure, axial force, shear, and torsion are as follows:

$$\phi = 0.90 \text{ for flexure,} \quad (\text{ACI 9.3.2.1})$$

$$\phi = 0.90 \text{ for axial tension,} \quad (\text{ACI 9.3.2.2})$$

$$\phi = 0.90 \text{ for axial tension and flexure,} \quad (\text{ACI 9.3.2.2})$$

$$\phi = 0.75 \text{ for axial compression, and axial compression} \\ \text{and flexure (spirally reinforced column),} \quad (\text{ACI 9.3.2.2})$$

$$\phi = 0.70 \text{ for axial compression, and axial compression} \\ \text{and flexure (tied column), and} \quad (\text{ACI 9.3.2.2})$$

$$\phi = 0.85 \text{ for shear and torsion.} \quad (\text{ACI 9.3.2.3})$$

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 II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement — 1 to 8 percent for Ordinary and Intermediate moment resisting frames (ACI 10.9.1) and 1 to 6 percent for Special moment resisting frames (ACI 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 the above-mentioned steps.

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 diagram is shown in Figure II-1.

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 II-2. The linear strain diagram limits the maximum concrete strain, ϵ_c , at the extremity of the section to 0.003 (ACI 10.2.3).

The formulation is based consistently upon the general principles of ultimate strength design (ACI 10.3), and allows for any doubly symmetric rectangular, square, or circular column section.

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\epsilon_s E_s$, and is limited to the yield stress of the steel, f_y (ACI 10.2.4). 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. See Figure III-1.

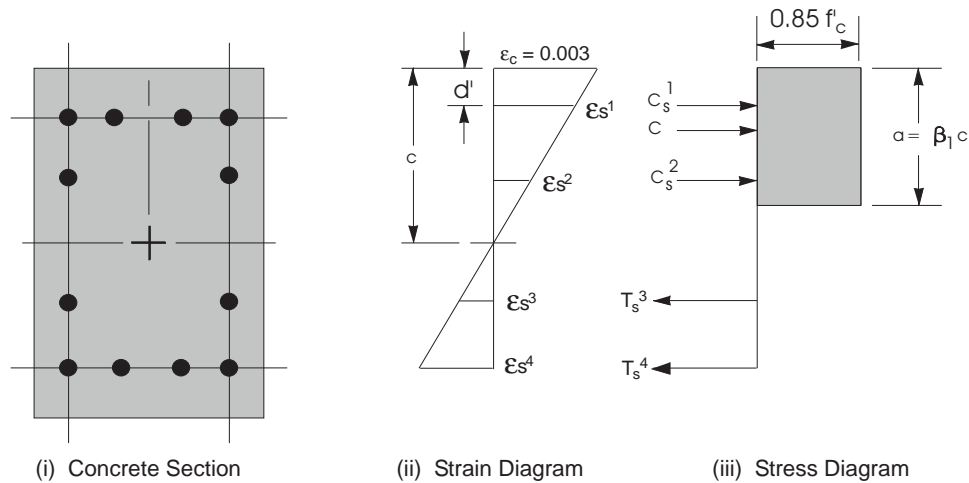


Figure III-1
Idealization of Stress and Strain Distribution in a Column Section

The concrete compression stress block is assumed to be rectangular, with a stress value of $0.85f'_c$ (ACI 10.2.7.1). See Figure III-1. The interaction algorithm provides correction to account for the concrete area that is displaced by the reinforcement in the compression zone.

The effects of the strength reduction factor, ϕ , are included in the generation of the interaction surfaces. The maximum compressive axial load is limited to $\phi P_{n(max)}$, where

$$\phi P_{n(max)} = 0.85 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ spiral column, (ACI 10.3.5.1)}$$

$$\phi P_{n(max)} = 0.80 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ tied column, (ACI 10.3.5.2)}$$

$\phi = 0.70$ for tied columns, and

$\phi = 0.75$ for spirally reinforced columns.

The value of ϕ used in the interaction diagram varies from ϕ_{min} to 0.9 based on the axial load. For low values of axial load, ϕ is increased linearly from ϕ_{min} to 0.9 as the axial load decreases from the smaller of P_b or $0.1f'_cA_g$ to zero, where P_b is the axial force at the balanced condition. In cases involving axial tension, ϕ is always 0.9 (ACI 9.3.2.2).

Check Column Capacity

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 station, the following steps are involved:

- Determine the factored moments and forces from the analysis load cases and the specified load combination factors to give P_u , M_{ux} , and M_{uy} .
- Determine the moment magnification factors for the column moments.
- Apply the moment magnification factors to the factored moments. Determine whether the point, defined by the resulting axial load and biaxial moment set, lies within the interaction volume.

The factored moments and corresponding magnification factors depend on the identification of the individual column as either “sway” or “non-sway”.

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

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 cases, giving P_u , M_{ux} , and M_{uy} . The factored moments are further increased for non-sway columns, if required, to obtain minimum eccentricities of $(0.6 + 0.03 h)$ inches, where h is the dimension of the column in the corresponding direction (ACI 10.12.3.2).

Determine Moment Magnification Factors

The moment magnification factors are calculated separately for sway (overall stability effect), δ_s , and for non-sway (individual column stability effect), δ_{ns} . Also the moment magnification factors in the major and minor directions are in general different.

The program assumes that a P- Δ analysis has been performed in SAP2000 and, therefore, moment magnification factors for moments causing sidesway are taken

as unity (ACI 10.10.2). For the P-Δ analysis the load should correspond to a load combination of 0.75 (1.4 dead load + 1.7 live load)/φ, where φ is the understrength factor for stability which is taken as 0.75 (ACI 10.12.3). See also White and Hajjar (1991).

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

For individual columns or column-members in a floor, the magnified moments about two axes at any station of a column can be obtained as

$$M = M_{ns} + \delta_s M_s . \quad (\text{ACI 10.13.3})$$

The factor δ_s is the moment magnification factor for moments causing side sway. The moment magnification factors for sway moments, δ_s , is taken as 1 because the component moments M_s and M_{ns} are obtained from a “second order elastic (P-Δ) analysis” (ACI R10.13).

The computed moments are further amplified for individual column stability effect (ACI 10.13.5) by the nonsway moment magnification factor, δ_{ns} , as follows:

$$M_c = \delta_{ns} M_2 , \text{ where} \quad (\text{ACI 10.12.3})$$

M_c is the factored moment to be used in design, and

M_2 is the larger factored and amplified end moment.

The non-sway moment magnification factor, δ_{ns} , associated with the major or minor direction of the column is given by (ACI 10.12.3)

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0, \text{ where}$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2},$$

k is conservatively taken as 1, however SAP2000 allows the user to override this value, and

EI is associated with a particular column direction given by:

$$EI = \frac{0.4 E_c I_g}{1 + \beta_d},$$

$$\beta_d = \frac{\text{maximum factored axial dead load}}{\text{maximum factored axial total load}}, \text{ and}$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4. \quad (\text{ACI 10.12.3.1})$$

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 above 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, $C_m = 1$. C_m can be overwritten by the user on an element by element basis.

The magnification factor, δ_{ns} , must be a positive number and greater than one. Therefore P_u must be less than $0.75P_c$. If P_u is found to be greater than or equal to $0.75P_c$, a failure condition is declared.

The above 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. See Figure II-4. 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_{ns} .

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_u , M_{ux} , and M_{uy} . The point (P_u, M_{ux}, M_{uy}) is then placed in the interaction space shown as point L in Figure II-3. 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.

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 out-

wards) will intersect the failure surface. This point is determined by three-dimensional linear interpolation between the points that define the failure surface. See Figure II-3. 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_u , M_{ux} , and M_{uy} set and associated load combination number.

If the reinforcing area is not defined, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

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 following steps are involved:

- Determine the factored forces acting on the section, P_u and V_u . Note that P_u is needed for the calculation of V_c .
- Determine the shear force, V_c , that can be resisted by concrete alone.
- Calculate the reinforcement steel required to carry the balance.

For Special and Intermediate moment resisting frames (ductile frames), the shear design of the columns is also based upon the probable and nominal moment capacities of the members, respectively, in addition to the factored moments. 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 the above-mentioned steps.

Determine Section Forces

- In the design of the column shear reinforcement of an **Ordinary moment resisting concrete frame**, the forces for a particular load combination, namely, the column axial force, P_u , and the column shear force, V_u , in a particular direction are obtained by factoring the SAP2000 analysis load cases with the corresponding load combination factors.
- In the shear design of **Special moment resisting frames** (seismic design) the following are checked in addition to the requirement for the Ordinary moment resisting frames. In the design of Special moment resisting concrete frames, the design shear force in a column, V_u , 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.

For each load combination, the factored axial load, P_u , is calculated. Then, the positive and negative moment capacities, M_u^+ and M_u^- , of the column in a particular direction under the influence of the axial force P_u is calculated using the uniaxial interaction diagram in the corresponding direction. The design shear force, V_u , is then given by (ACI 21.4.5.1)

$$V_u = V_p + V_{D+L} \quad (\text{ACI 21.4.5.1})$$

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_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

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 ($\phi = 1.0$),

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 ($\phi = 1.0$), and

L = Clear span of column.

For Special moment resisting frames α is taken as 1.25 (ACI R21.4.5.1). 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.

- For **Intermediate moment resisting frames**, the shear capacity of the column is also checked for the design nominal shear based on the nominal moment capacities at the ends and the factored gravity loads, in addition to the check required for Ordinary moment resisting frames. The design shear force is taken to be the minimum of that based on the nominal ($\phi = 1.0$) moment capacity and factored shear force. The procedure for calculating nominal moment capacity is the same as that for computing the probable moment capacity for special moment resisting frames, except that α is taken equal to 1 rather than 1.25 (ACI R21.10). The factored shear forces are based on the specified load factors except the earthquake load factors are doubled (ACI 21.10.3).

Determine Concrete Shear Capacity

Given the design force set P_u and V_u , the shear force carried by the concrete, V_c , is calculated as follows:

- If the column is subjected to axial compression, i.e. P_u is positive,

$$V_c = 2\sqrt{f'_c} \left(1 + \frac{P_u}{2000A_g} \right) A_{cv} , \quad (\text{ACI 11.3.1.2})$$

where,

$$\sqrt{f'_c} \leq 100 \text{ psi, and} \quad (\text{ACI 11.1.2})$$

$$V_c \leq 3.5\sqrt{f'_c} \sqrt{\left(1 + \frac{P_u}{500A_g} \right)} A_{cv} . \quad (\text{ACI 11.3.2.2})$$

The term $\frac{P_u}{A_g}$ must have psi units. A_{cv} is the effective shear area which is shown shaded in Figure III-2.

- If the column is subjected to axial tension, P_u is negative,

$$V_c = 2\sqrt{f'_c} \left(1 + \frac{P_u}{500A_g} \right) A_{cv} \geq 0 \quad (\text{ACI 11.3.2.3})$$

- For **Special moment resisting concrete frame** design, V_c is set to zero if the factored axial compressive force, P_u , including the earthquake effect is small ($P_u < f'_c A_g / 20$) and if the shear force contribution from earthquake, V_E , is more than half of the total factored maximum shear force over the length of the member V_u ($V_E \geq 0.5V_u$) (ACI 21.4.5.2).

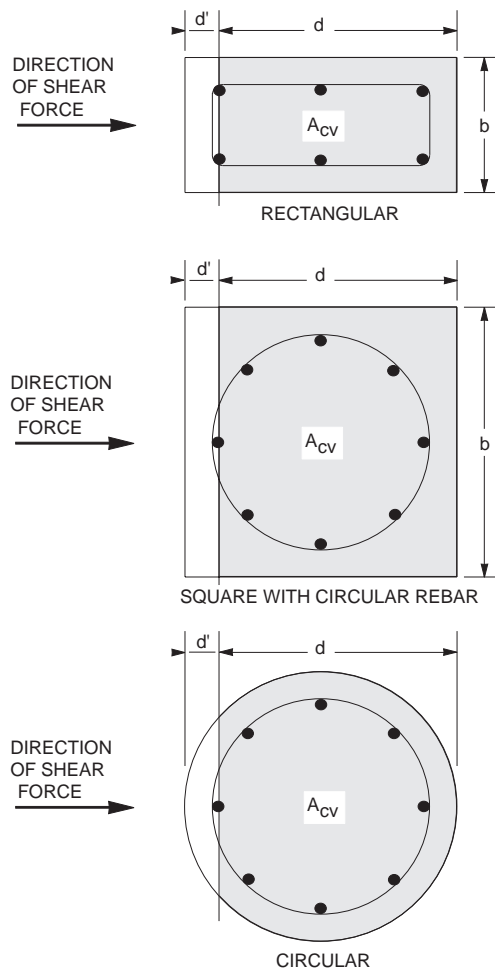


Figure III-2
Shear Stress Area, A_{cv}

Determine Required Shear Reinforcement

Given V_u and V_c , the required shear reinforcement in the form of stirrups or ties within a spacing, s , is given by

$$A_v = \frac{(V_u/\phi - V_c)s}{f_{ys}d}, \quad (\text{ACI 11.5.6.2})$$

$$(V_u / \phi - V_c) \leq 8\sqrt{f'_c}A_{cv} \quad (\text{ACI 11.5.6.9})$$

Otherwise redimensioning of the concrete section is required. Here ϕ , the strength reduction factor, is 0.85 (ACI 9.3.2.3). The maximum of all the calculated A_v values obtained from each load combination are reported for the major and minor directions of the column along with the controlling shear force and associated load combination label.

The column shear reinforcement requirements reported by the program are based purely upon shear strength consideration. Any minimum stirrup requirements to satisfy spacing considerations or transverse reinforcement volumetric considerations must be investigated independently of the program by the user.

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, shears, load combination factors, and other criteria described below. The reinforcement requirements are calculated at a user defined number of check/design stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

Design Beam Flexural Reinforcement

The beam top and bottom flexural steel is designed at check/design stations along the beam span. In designing the flexural reinforcement for the major moment for a particular beam for a particular section, the following steps are involved:

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement of Special, Intermediate, or Ordinary moment resisting concrete frame beams, the factored moments for each load combination at a particular beam section 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_u^+ and maximum negative M_u^- factored moments 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- or a T-beam.

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 III-3 (ACI 10.2). Furthermore it is assumed that the compression carried by concrete is less than 0.75 times that which can be carried at the balanced condition (ACI 10.3.3). When the applied moment exceeds the moment capacity at this designed balanced condition, the area of compression reinforcement is calculated on the assumption that the additional moment will be carried by compression and additional tension reinforcement.

The design procedure used by SAP2000, for both rectangular and flanged sections (L- and T-beams) is summarized below. It is assumed that the design ultimate axial force does not exceed $0.1f_c' A_g$ (ACI 10.3.3), hence all the beams are designed for major direction flexure and shear only.

Design for Rectangular Beam

In designing for a factored negative or positive moment, M_u , (i.e. designing top or bottom steel) the depth of the compression block is given by a (see Figure III-3), where,

$$a = d - \sqrt{d^2 - \frac{2|M_u|}{0.85f'_c\phi b}}$$

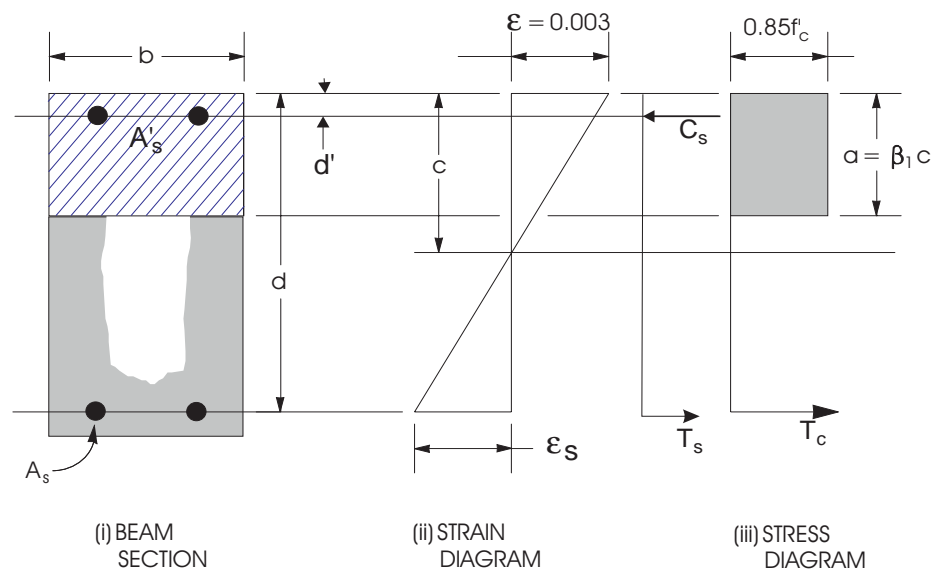


Figure III-3
Design of Rectangular Beam Section

where, the value of ϕ is 0.90 (ACI 9.3.2.1) in the above and the following equations. Also β_1 and c_b are calculated as follows:

$$\beta_1 = 0.85 - 0.05 \left(\frac{f'_c - 4000}{1000} \right), \quad 0.65 \leq \beta_1 \leq 0.85, \quad (\text{ACI } 10.2.7.3)$$

$$c_b = \frac{\epsilon_c E_s}{\epsilon_c E_s + f_y} d = \frac{87000}{87000 + f_y} d. \quad (\text{ACI } 10.2.3, 10.2.4)$$

The maximum allowed depth of the compression block is given by

$$a_{max} = 0.75\beta_1 c_b. \quad (\text{ACI } 10.2.7.1)$$

- If $a \leq a_{max}$, the area of tensile steel reinforcement is then given by

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2} \right)} .$$

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

- If $a > a_{max}$, compression reinforcement is required (ACI 10.3.3) and is calculated as follows:

- The compressive force developed in concrete alone is given by

$$C = 0.85 f'_c b a_{max} , \text{ and} \quad (\text{ACI 10.2.7.1})$$

the moment resisted by concrete compression and tensile steel is

$$M_{uc} = C \left(d - \frac{a_{max}}{2} \right) \phi .$$

- Therefore the moment resisted by compression steel and tensile steel is

$$M_{us} = M_u - M_{uc} .$$

- So the required compression steel is given by

$$A'_s = \frac{M_{us}}{f'_s (d - d') \phi} , \text{ where}$$

$$f'_s = 0.003 E_s \left[\frac{c - d'}{c} \right] . \quad (\text{ACI 10.2.4})$$

- The required tensile steel for balancing the compression in concrete is

$$A_{s1} = \frac{M_{uc}}{f_y \left[d - \frac{a_{max}}{2} \right] \phi} , \text{ and}$$

the tensile steel for balancing the compression in steel is given by

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

- 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 bottom and A'_s is to be placed at top if M_u is positive, and vice versa if M_u is negative.

Design for T-Beam

In designing for a factored negative moment, M_u , (i.e. designing top steel), the calculation of the steel area is exactly the same as above, i.e., no T-Beam data is to be used. See Figure III-4. If $M_u > 0$, the depth of the compression block is given by

$$a = d - \sqrt{d^2 - \frac{2 M_u}{0.85 f'_c \phi b_f}}$$

The maximum allowed depth of compression block is given by

$$a_{max} = 0.75 \beta_1 c_b \quad \text{(ACI 10.2.7.1)}$$

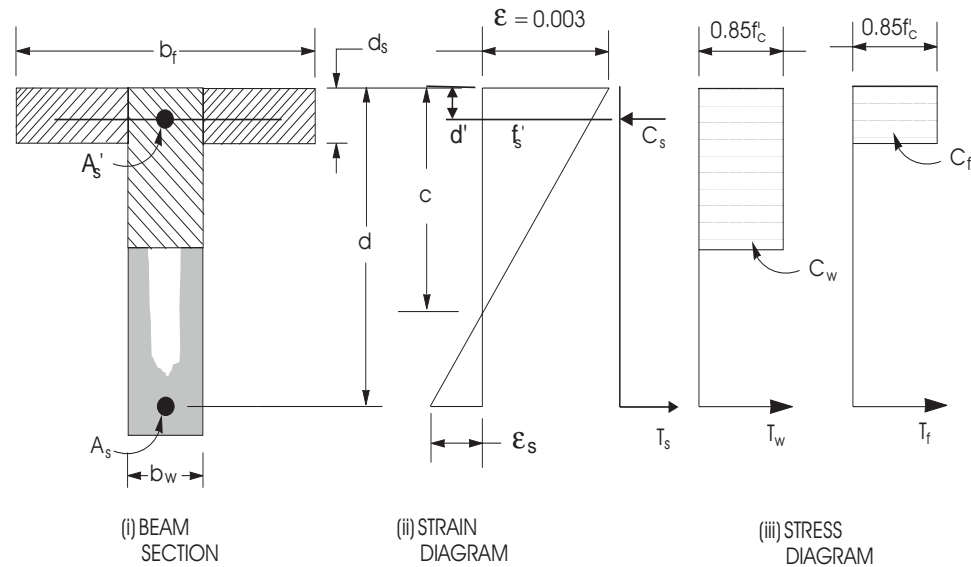


Figure III-4
Design of a T-Beam Section

- If $a \leq d_s$, the subsequent calculations for A_s are exactly the same as previously defined for the rectangular section design. However, in this case the width of the compression flange is taken as the width of the beam for analysis. Whether compression reinforcement is required depends on whether $a > a_{max}$.

- If $a > d_s$, calculation for A_s is done 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 Figure III-4. C_f is given by

$$C_f = 0.85f'_c(b_f - b_w)d_s.$$

Therefore, $A_{s1} = \frac{C_f}{f_y}$ and the portion of M_u that is resisted by the flange is given by

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

Again, the value for ϕ is 0.90. Therefore, the balance of the moment, M_u to be carried by the web is given by

$$M_{uw} = M_u - M_{uf}.$$

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

$$a_1 = d - \sqrt{d^2 - \frac{2M_{uw}}{0.85f'_c\phi b_w}}.$$

- If $a_1 \leq a_{max}$, the area of tensile steel reinforcement is then given by

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

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

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

- If $a_1 > a_{max}$, compression reinforcement is required (ACI 10.3.3) and is calculated as follows:

– The compressive force in web concrete alone is given by

$$C = 0.85f'_c b a_{max}. \quad (\text{ACI 10.2.7.1})$$

- Therefore the moment resisted by concrete web and tensile steel is

$$M_{uc} = C \left(d - \frac{a_{max}}{2} \right) \phi, \text{ and}$$

the moment resisted by compression steel and tensile steel is

$$M_{us} = M_{uw} - M_{uc}.$$

- Therefore, the compression steel is computed as

$$A'_s = \frac{M_{us}}{f'_s (d - d') \phi}, \text{ where}$$

$$f'_s = 0.003 E_s \left[\frac{c - d'}{c} \right]. \quad (\text{ACI 10.2.4})$$

- The tensile steel for balancing compression in web concrete is

$$A_{s2} = \frac{M_{uc}}{f_y \left(d - \frac{a_{max}}{2} \right) \phi}, \text{ and}$$

the tensile steel for balancing compression in steel is

$$A_{s3} = \frac{M_{us}}{f_y (d - d') \phi}.$$

- The 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 bottom and A'_s is to be placed at top.

Minimum Tensile Reinforcement

The minimum flexural tensile steel provided in a rectangular section in an Ordinary moment resisting frame is given by the minimum of the two following limits:

$$A_s \geq \max \left\{ \frac{3\sqrt{f'_c}}{f_y} b_w d \quad \text{and} \quad \frac{200}{f_y} b_w d \right\} \text{ or} \quad (\text{ACI 10.5.1})$$

$$A_s \geq \frac{4}{3} A_{s(\text{required})}. \quad (\text{ACI 10.5.3})$$

Special Consideration for Seismic Design

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

- The minimum longitudinal reinforcement shall be provided at both at the top and bottom. Any of the top and bottom reinforcement shall not be less than $A_{s(min)}$ (ACI 21.3.2.1).

$$A_{s(min)} \geq \max \left\{ \frac{3\sqrt{f'_c}}{f_y} b_w d \quad \text{and} \quad \frac{200}{f_y} b_w d \right\} \quad \text{or} \quad \text{(ACI 10.5.1)}$$

$$A_{s(min)} \geq \frac{4}{3} A_{s(required)} \quad \text{(ACI 10.5.3)}$$

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

$$A_s \geq 0.025 b_w d \quad \text{(ACI 21.3.2.1)}$$

- At any end (support) of the beam, the beam positive moment capacity (i.e. associated with the bottom steel) would not be less than 1/2 of the beam negative moment capacity (i.e. associated with the top steel) at that end (ACI 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 1/4 of the maximum of positive or negative moment capacities of any of the beam end (support) stations (ACI 21.3.2.2).

For Intermediate 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 1/3 of the beam negative moment capacity at that end (ACI 21.10.4.1).
- Neither the negative moment capacity nor the positive moment capacity at any of the sections within the beam would be less than 1/5 of the maximum of positive or negative moment capacities of any of the beam end (support) stations (ACI 21.10.4.1).

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 following steps are involved:

- Determine the factored shear force, V_u .
- Determine the shear force, V_c , that can be resisted by the concrete.
- Determine the reinforcement steel required to carry the balance.

For Special and Intermediate moment resisting frames (ductile frames), the shear design of the beams is also based upon the probable and nominal moment capacities of the members, respectively, in addition to the factored load design.

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

Determine Shear Force and Moment

- In the design of the beam shear reinforcement of an **Ordinary moment resisting 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 **Special 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 at the ends and the factored gravity load. This check is done in addition to the design check required for Ordinary moment resisting frames. The shear force, V_u , is calculated from the probable moment capacities of each end of the beam and the gravity shear forces. The procedure for calculating the design shear force in a beam from probable moment capacity is the same as that described for a column in section “Column Design” on page 27. See also Table III-2 for details.

The design shear force V_u is then given by (ACI 21.3.4.1)

$$V_u = V_p + V_{D+L} \quad (\text{ACI 21.3.4.1})$$

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_{p_1} and V_{p_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

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 ($\phi = 1.0$),

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 ($\phi = 1.0$),

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 ($\phi = 1.0$),

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 ($\phi = 1.0$), and

L = Clear span of beam.

For Special moment resisting frames α is taken as 1.25 (ACI R21.3.4.1). V_{D+L} is the contribution of shear force from the in-span distribution of gravity loads.

- For **Intermediate moment resisting frames**, the shear capacity of the beam is also checked for the design nominal shear based on the nominal moment capacities at the ends and the factored gravity loads, in addition to the check required for Ordinary moment resisting frames. The design shear force in beams is taken to be the minimum of that based on the nominal moment capacity and factored shear force. The procedure for calculating nominal ($\phi = 1.0$) moment capacity is the same as that for computing the probable moment capacity for Special moment resisting frames, except that α is taken equal to 1 rather than 1.25 (ACI R21.10). The factored shear forces are based on the specified load factors except the earthquake load factors are doubled (ACI 21.10.3). The computation of the design shear force in a beam of an **Intermediate moment resisting frame**, is also the same as that for columns, which is described earlier on page 28. See also Table III-2 for details.

Determine Concrete Shear Capacity

The allowable concrete shear capacity is given by

$$V_c = 2\sqrt{f'_c} b_w d . \quad (\text{ACI 11.3.1.1})$$

For Special moment resisting frame concrete design, V_c is set to zero if both the factored axial compressive force including the earthquake effect P_u is less than $f'_c A_g / 20$ and the shear force contribution from earthquake V_E is more than half of the total maximum shear force over the length of the member V_u (i.e. $V_E \geq 0.5V_u$) (ACI 21.3.4.2).

Determine Required Shear Reinforcement

Given V_u and V_c , the required shear reinforcement in area/unit length is calculated as

$$A_v = \frac{(V_u/\phi - V_c) s}{f_{ys} d} . \quad (\text{ACI 11.5.6.2})$$

The shear force resisted by steel is limited by

$$(V_u/\phi - V_c) \leq 8\sqrt{f'_c} bd , \quad (\text{ACI 11.5.6.9})$$

where, ϕ , the strength reduction factor, is 0.85 (ACI 9.3.2.3). The maximum of all the calculated A_v values, obtained from each load combination, is reported along with the controlling shear force and associated load combination number.

The beam shear reinforcement requirements displayed by the program are based purely upon shear strength considerations. Any minimum stirrup requirements to satisfy spacing and volumetric considerations must be investigated independently of the program by the user.

Type of Check/ Design	Ordinary Moment Resisting Frames (non-Seismic)	Intermediate Moment Resisting Frames (Seismic)	Special Moment Resisting Frames (Seismic)
Column Check (interaction)	NLD ^a Combinations	NLD ^a Combinations	NLD ^a Combinations
Column Design (Interaction)	NLD ^a Combinations 1% < ρ < 8%	NLD ^a Combinations 1% < ρ < 8%	NLD ^a Combinations α = 1.0 1% < ρ < 6%
Column Shears	NLD ^a Combinations	Modified NLD ^a Combinations (earthquake loads doubled) Column Capacity φ = 1.0 and α = 1.0	NLD ^a Combinations and Column shear capacity φ = 1.0 and α = 1.25
Beam Design Flexure	NLD ^a Combinations	NLD ^a Combinations	NLD ^a Combinations ρ ≤ 0.025 $\rho \geq \frac{3\sqrt{f_c}}{f_y}, \rho \geq \frac{200}{f_y}$
Beam Min. Moment Override Check	No Requirement	$M_{uEND}^+ \geq \frac{1}{3} M_{uEND}^-$ $M_{uSPAN}^+ \geq \frac{1}{5} \max\{M_u^+, M_u^-\}_{END}$ $M_{uSPAN}^- \geq \frac{1}{5} \max\{M_u^+, M_u^-\}_{END}$	$M_{uEND}^+ \geq \frac{1}{2} M_{uEND}^-$ $M_{uSPAN}^+ \geq \frac{1}{4} \max\{M_u^+, M_u^-\}_{END}$ $M_{uSPAN}^- \geq \frac{1}{4} \max\{M_u^+, M_u^-\}_{END}$
Beam Design Shear	NLD ^a Combinations	Modified NLD ^a Combinations (earthquake loads doubled) Beam Capacity Shear (V _p) with α = 1.0 and φ = 1.0 plus V _{D+L}	NLD ^a Combinations Beam Capacity Shear (V _p) with α = 1.25 and φ = 1.0 plus V _{D+L} V _c = 0

^a NLD = Number of specified loading

Table III-2
Design Criteria Table

Chapter IV

Design for AASHTO LRFD 1997

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the AASHTO LRFD 1997 Design Code (AASHTO 1997). Various notations used in this chapter are listed in Table IV-1.

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

SAP2000 provides options to design or check moment resisting frames of Zones 1 (low seismic activity), 2, 3, and 4 (high seismic activity) as required for seismic design provisions. The details of the design criteria used for the different seismic zones are described in the following sections.

English as well as SI and MKS metric units can be used for input. But the code is based on Inch-Kip-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Inch-Kip-Second** units unless otherwise noted.

A_{cv}	Area of concrete used to determine shear stress, sq-in
A_g	Gross area of concrete, sq-in
A_s	Area of tension reinforcement, sq-in
A_s'	Area of compression reinforcement, sq-in
$A_{s(required)}$	Area of steel required for tension reinforcement, sq-in
A_{st}	Total area of column longitudinal reinforcement, sq-in
A_v	Area of shear reinforcement, sq-in
a	Depth of compression block, in
a_b	Depth of compression block at balanced condition, in
a_{max}	Maximum allowed depth of compression block, in
b	Width of member, in
b_f	Effective width of flange (T-Beam section), in
b_w	Width of web (T-Beam section), in
C_m	Coefficient, dependent upon column curvature, used to calculate moment magnification factor
c	Depth to neutral axis, in
c_b	Depth to neutral axis at balanced conditions, in
d	Distance from compression face to tension reinforcement, in
d'	Concrete cover to center of reinforcing, in
d_s	Thickness of slab (T-Beam section), in
E_c	Modulus of elasticity of concrete, psi
E_s	Modulus of elasticity of reinforcement, assumed as 29,000 ksi
f_c'	Specified compressive strength of concrete, ksi
f_y	Specified yield strength of flexural reinforcement, ksi
f_{yh}	Specified yield strength of shear reinforcement, ksi
h	Dimension of column, in
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in ⁴
I_{se}	Moment of inertia of reinforcement about centroidal axis of member cross section, in ⁴
k	Effective length factor
L	Clear unsupported length, in

Table IV-1
List of Symbols Used in the AASHTO code

M_1	Smaller factored end moment in a column, kip-in
M_2	Larger factored end moment in a column, kip-in
M_c	Factored moment to be used in design, kip-in
M_b	Nonsway component of factored end moment, kip-in
M_s	Sway component of factored end moment, kip-in
M_u	Factored moment at section, kip-in
M_{ux}	Factored moment at section about X-axis, kip-in
M_{uy}	Factored moment at section about Y-axis, kip-in
P_b	Axial load capacity at balanced strain conditions, kip
P_e	Euler buckling strength of column, kip
P_{max}	Maximum axial load strength allowed, kip
P_0	Axial load capacity at zero eccentricity, kip
P_u	Factored axial load at section, kip
r	Radius of gyration of column section, in
V_c	Shear resisted by concrete, kip
V_{D+L}	Shear force from span loading, kip
V_u	Factored shear force at a section, kip
V_p	Shear force computed from probable moment capacity, kip
α	Reinforcing steel overstrength factor
β	is a factor indicating the ability of diagonally cracked concrete to transmit tension
β_1	Factor for obtaining depth of compression block in concrete
β_d	Absolute value of ratio of maximum factored axial dead load to maximum factored axial total load
θ	An angle of inclination of diagonal compressive stresses with the longitudinal axis of beam or column
δ_s	Moment magnification factor for sway moments
δ_b	Moment magnification factor for nonsway (braced) moments
ϵ_c	Strain in concrete
ϵ_s	Strain in reinforcing steel
ϕ	Strength reduction factor

Table IV-1*List of Symbols Used in the AASHTO code (continued)*

Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs to be checked. There are more different types of loads specified in the code than are considered in the current implementation of the default load combinations. However, the user has full control of the definition of loads and load combinations.

There are six types of dead loads: dead load of structural components and nonstructural attachments (DC), downdrag (DD), dead load of wearing surface and utilities (DW), horizontal earth pressure load (EH), vertical earth pressure load (EV), earth surcharge load (ES). Each type of dead load case requires a separate load factor.

There are six types of live loads: vehicular live load (LL), vehicular dynamic load allowance (IM), vehicular centrifugal force (CE), vehicular braking force (BR), pedestrian live load (PL), and live load surcharge (LS). All these load cases require the same factor and do not need to be treated separately.

If the structure is subjected to structural dead load (DL), live load (LL), wind load (WL), and earthquake loads (EL), and considering that wind and earthquake forces are reversible, the following default load combinations have been considered for Strength and Extreme Event limit states (AASHTO 3.4.1).

1.50 DL	(Strength-IV)
1.25 DL + 1.75 LL	(Strength-I)
0.90 DL ± 1.4 WL	(Strength-III)
1.25 DL ± 1.4 WL	(Strength-III)
1.25 DL + 1.35 LL ± 0.40 WL	(Strength-V)
0.90 DL ± 1.0 EL	(Extreme-I)
1.25 DL + 0.5 LL ± 1.0 EL	(Extreme-I)

These are also the default design load combinations in SAP2000 whenever the AASHTO LRFD 1997 code is used. The user is expected to define the other load combinations as necessary.

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.

Strength Reduction Factors

The strength reduction factors, ϕ , are applied on the nominal strength to obtain the design strength provided by a member. The ϕ factors for flexure, axial force, shear, and torsion are as follows:

$$\phi = 0.90 \text{ for flexure,} \quad (\text{AASHTO 5.5.4.2.1})$$

$$\phi = 0.90 \text{ for axial tension,} \quad (\text{AASHTO 5.5.4.2.1})$$

$$\phi = 0.90 \text{ for axial tension and flexure,} \quad (\text{AASHTO 5.5.4.2.1})$$

$$\phi = 0.90 \text{ for shear and torsion, and} \quad (\text{AASHTO 5.5.4.2.1})$$

$$\phi = 0.75 \text{ for axial compression only, or axial compression and flexure.} \quad (\text{AASHTO 5.5.4.2.1})$$

$$\phi = 0.50 \text{ for axial compression only, or axial compression and flexure in} \\ \text{seismic zone 3 and 4.} \quad (\text{AASHTO 5.5.4.2.3, 5.10.11.4.1b})$$

The value of ϕ involving axial compression and flexure varies from 0.75 to 0.9 based on the axial load. For low values of axial load, ϕ is increased linearly from 0.75 to 0.9 as the axial load decreases from $0.1f'_cA_g$ to zero (AASHTO 5.5.4.2.1). For seismic design in Zones 3 and 4, the value of ϕ involving axial compression and flexure varies from 0.5 to 0.9 based on the axial load. For low values of axial load, ϕ is increased linearly from 0.5 to 0.9 as the axial load decreases from $0.2f'_cA_g$ to zero (AASHTO 5.10.11.4.1b). In cases involving axial tension, ϕ is always 0.9 (AASHTO 5.5.4.2.1).

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 II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement ratio (A_{st}/A_g)—

$0.135 f'_c / f_y$ to 0.08 for moment resisting frames (AASHTO 5.7.4.2) and 0.01 to 0.06 for ductile moment resisting frames in seismic Zones 3 and 4 (AASHTO 5.10.11.4.1a).

- 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 1.0 when calculating the required reinforcing area.
- Design the column shear reinforcement.

The following three subsections describe in detail the algorithms associated with the above-mentioned steps.

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 diagram is shown in Figure II-1.

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 II-2. The linear strain diagram limits the maximum concrete strain, ϵ_c , at the extremity of the section to 0.003 (AASHTO 5.7.2.1).

The formulation is based consistently upon the general principles of ultimate strength design (AASHTO 5.7), and allows for any doubly symmetric rectangular, square, or circular column section.

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\epsilon_s E_s$, and is limited to the yield stress of the steel, f_y (AASHTO 5.7.2.1). 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. See Figure IV-1.

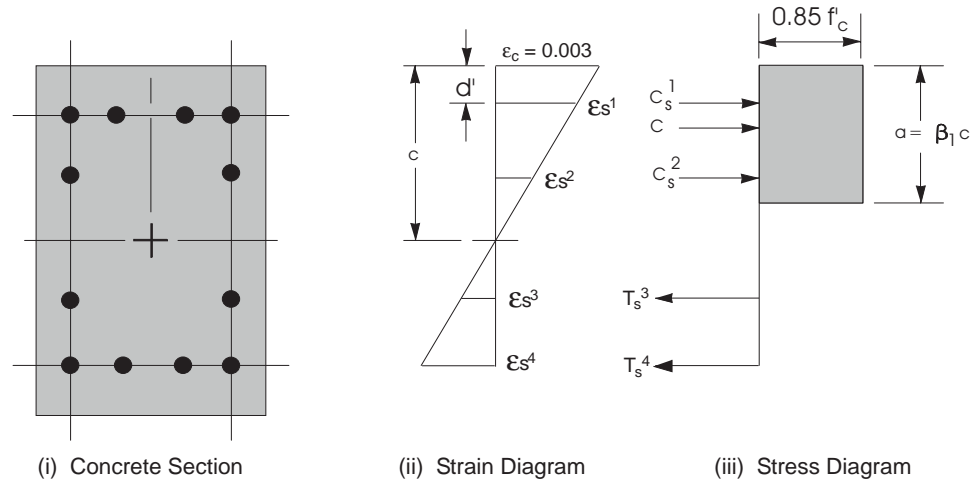


Figure IV-1
Idealization of Stress and Strain Distribution in a Column Section

The concrete compression stress block is assumed to be rectangular (AASHTO 5.7.2.1), with a stress value of $0.85f'_c$ (AASHTO 5.7.2.2). See Figure IV-1. The depth of the stress block of $\beta_1 c$, where

$$\beta_1 = 0.85 - 0.05(f'_c - 4), \quad (\text{AASHTO 5.7.2.2})$$

$$0.65 \leq \beta_1 \leq 0.85, \text{ and} \quad (\text{AASHTO 5.7.2.2})$$

The limit of f'_c is taken to be 10 ksi for all seismic regions:

$$f'_c \leq 10 \text{ ksi.} \quad (\text{AASHTO 5.1, 5.4.2.1})$$

The limit of f_y is taken to be 75 ksi for all frames:

$$f_y \leq 75 \text{ ksi.} \quad (\text{AASHTO 5.4.3.1})$$

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 factor, ϕ , are included in the generation of the interaction surfaces. The maximum compressive axial load is limited to P_{max} , where

$$P_{max} = 0.85 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ spiral column, (AASHTO 5.7.4.4)}$$

$$P_{max} = 0.80 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ tied column. (AASHTO 5.7.4.4)}$$

The value of ϕ involving axial compression and flexure varies from 0.75 to 0.9 based on the axial load. For low values of axial load, ϕ is increased linearly from 0.75 to 0.9 as the axial load decreases from $0.1 f'_c A_g$ to zero (AASHTO 5.5.4.2.1). For seismic design in Zones 3 and 4, the value of ϕ involving axial compression and flexure varies from 0.5 to 0.9 based on the axial load. For low values of axial load, ϕ is increased linearly from 0.5 to 0.9 as the axial load decreases from $0.2 f'_c A_g$ to zero (AASHTO 5.10.11.4.1b). In cases involving axial tension, ϕ is always 0.9 (AASHTO 5.5.4.2.1).

Check Column Capacity

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 station, the following steps are involved:

- Determine the factored moments and forces from the analysis load cases and the specified load combination factors to give P_u , M_{ux} , and M_{uy} .
- Determine the moment magnification factors for the column moments.
- Apply the moment magnification factors to the factored moments. Determine whether the point, defined by the resulting axial load and biaxial moment set, lies within the interaction volume.

The factored moments and corresponding magnification factors depend on the identification of the individual column as either “braced” or “unbraced”.

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

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 cases, giving P_u , M_{ux} , and M_{uy} . The computed moments are further amplified by using “Moment Magnification Factors” to allow for stability effects.

Determine Moment Magnification Factors

The moment magnification factors are calculated separately for sway (overall stability effect), δ_s , and for nonsway or braced (individual column stability effect), δ_{ns} . Also the moment magnification factors in the major and minor directions are in general different.

The program assumes that a P- Δ analysis has been performed in SAP2000 and, therefore, moment magnification factors for moments causing sidesway are taken as unity (AASHTO 4.5.3). For the P- Δ analysis the load should correspond to a load combination of (1.25 dead load + 1.35 live load) / ϕ , where ϕ is the resistance factor for axial compression which is taken as 0.75 for seismic zones 1 and 2, and as 0.5 for seismic zones 3 and 4 by default (AASHTO 5.5.4.2.1). See also White and Hajjar (1991).

The moment obtained from analysis is separated into two components: the sway (M_s) and the nonsway (M_b) components. The non-sway or braced components which are identified by “b” 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 side sway.

For individual columns or column-members in a floor, the magnified moments about two axes at any station of a column can be obtained as

$$M = \delta_b M_b + \delta_s M_s . \quad (\text{AASHTO 4.5.3.2.2b})$$

The factor δ_s is the moment magnification factor for moments causing side sway. This factor is taken as 1 because the component moments M_s and M_b are obtained from a “second order elastic (P- Δ) analysis”.

The nonsway moment magnification factor, δ_b , associated with the major or minor direction of the column is given by (AASHTO 4.5.3.2.2b),

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi P_e}} \geq 1.0, \quad \text{where} \quad (\text{AASHTO 4.5.3.2.2b})$$

$$P_e = \frac{\pi^2 EI}{(kL)^2}, \quad (\text{AASHTO 4.5.3.2.2b})$$

k is taken as 1, however SAP2000 allows the user to override this value (AASHTO 4.6.2.5, 5.7.4.3), and

EI is associated with a particular column direction given by:

$$EI = \frac{E_c I_g / 2.5}{1 + \beta_d}, \quad (\text{AASHTO 5.7.4.3})$$

$$\beta_d = \left| \frac{\text{maximum factored dead load moment}}{\text{maximum factored total load moment}} \right|, \quad \text{and} \quad (\text{AASHTO 5.7.4.3})$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4. \quad (\text{AASHTO 4.5.3.2.2b})$$

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 above expression of C_m is valid if there is no transverse load applied between the supports and the member is braced against side-sway. If transverse load is present on the span, or the length is overwritten, or for any other case, $C_m = 1$. C_m can be overwritten by the user on an element by element basis.

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

The above calculations use the unsupported lengths 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. See Figure II-4. 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 .

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_u , M_{ux} , and M_{uy} . The point (P_u, M_{ux}, M_{uy}) is then placed in the interaction space shown as point L in Figure II-3. 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.

This capacity ratio is achieved by plotting the point L and determining the location of point C. 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 II-3.

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_u , M_{ux} , and M_{uy} set and associated load combination number.

If the reinforcing area is not defined, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

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 following steps are involved:

- Determine the factored forces acting on the section, M_u , P_u and V_u . Note that M_u and P_u 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 moment resisting frames in seismic Zones 3 and 4, the shear design of the columns is also based upon the overstrength moment capacities of the members in addition to the factored moments (AASHTO 3.10.9.4.3). 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 the above-mentioned steps.

Determine Section Forces

- In the design of the column shear reinforcement of **moment resisting concrete frames in seismic Zones 1 and 2**, the forces for a particular load combination, namely, the column axial force, P_u , and the column shear force, V_u , in a particular direction are obtained by factoring the SAP2000 analysis load cases with the corresponding load combination factors.
- In the shear design of **moment resisting frames in seismic Zones 3 and 4 (seismic design)**, the following are checked in addition to the requirement for the moment resisting frames in Zones 1 and 2. In the design of moment resisting concrete frames in seismic Zones 3 and 4, the design shear force in a column, V_u , in a particular direction is also calculated from the overstrength moment capacities of the column associated with the factored axial force acting on the column (AASHTO 3.10.9.4.3).

For each load combination, the factored axial load, P_u , is calculated. Then, the positive and negative moment capacities, M_u^+ and M_u^- , of the column in a particular direction under the influence of the axial force P_u is calculated using the uniaxial interaction diagram in the corresponding direction. The design shear force, V_u , is then given by (AASHTO 3.10.9.4.3)

$$V_u = V_p + V_{D+L} \quad (\text{AASHTO 3.10.9.4.3})$$

where, V_p is the shear force obtained by applying the calculated overstrength ultimate moment capacities at the two ends of the column acting in two opposite directions. Therefore, V_p is the maximum of V_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

M_I^+, M_I^- = Positive and negative moment capacities at end I of the column obtained by multiplying the nominal resistance by 1.3,

M_J^+, M_J^- = Positive and negative moment capacities at end J of the column obtained by multiplying the nominal resistance by 1.3,
and

L = Clear span of column.

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. See also Table IV-2 for details.

Determine Concrete Shear Capacity

Given the design force set M_u , P_u and V_u , the shear capacity provided by the concrete alone, v_c , is calculated as follows:

- For designing **moment resisting concrete frames in any seismic zone**, v_c is set to:

$$v_c = 0.0316 \beta \sqrt{f'_c}, \quad (\text{AASHTO 5.8.3.3})$$

where, β is a factor indicating the ability of diagonally cracked concrete to transmit tension. It is a function of stress condition and its approximate value is 2.0 (AASHTO 5.8.3.4.1). It is computed from AASHTO Table 5.8.3.4.2-1 through an iterative procedure.

- For designing **moment resisting concrete frames in seismic Zones 3 and 4**, v_c is taken as that given earlier (AASHTO 5.8.3.3) if compressive axial force is high ($P_u > 0.1 f'_c A_g$) (AASHTO 5.10.11.4.1c). v_c is taken as zero if axial force is tensile. v_c is linearly interpolated between zero and that given by AASHTO 5.8.3.3, if the factored axial compressive force, P_u , including the earthquake effect is small ($P_u \leq 0.1 f'_c A_g$) (AASHTO 5.10.11.4.1c). This provision is applied to all locations of the column irrespective of whether it is in the end region or not.

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular section as,

$$v = \frac{V_u}{b_w d}.$$

For other types of sections $b_w d$ is replaced by A_{cv} , the effective shear area which is shown in Figure IV-2.

- The average shear stress, v , is limited to a maximum limit, v_{max} , given by

$$v_{max} = 0.25 f'_c. \quad (\text{AASHTO 5.8.3.3})$$

- The shear reinforcement per unit spacing is computed as follows:

If $v \leq \phi (v_c / 2)$,

$$\frac{A_v}{s} = 0, \quad (\text{AASHTO 5.8.2.4})$$

else if $\phi (v_c / 2) < v \leq \phi (v_c + 0.0316\sqrt{f'_c} \cot \theta)$,

$$\frac{A_v}{s} = \frac{0.0316\sqrt{f'_c} b_w}{f_{yh}}, \quad (\text{AASHTO 5.8.2.5})$$

else if $\phi (v_c + 0.0316\sqrt{f'_c} \cot \theta) < v \leq \phi v_{max}$,

$$\frac{A_v}{s} = \frac{(v - \phi v_c) b_w}{\phi f_{yh} \cot \theta}, \quad (\text{AASHTO 5.8.3.3})$$

else if $v > v_{max}$,

a failure condition is declared. (AASHTO 5.8.3.3)

Here θ is an angle of inclination of diagonal compressive stresses. It is a function of current stress condition and its approximate value is 45 degree (AASHTO 5.8.3.4.1). It is computed from AASHTO Table 5.8.3.4.2-1 through an iterative procedure. Here the default value of ϕ , the strength reduction factor, is 0.90 (AASHTO 5.5.4.2.1).

The limit of f_{yh} is taken to be 60 ksi for all frames:

$$f_{yh} \leq 60 \text{ ksi.} \quad (\text{AASHTO 5.8.2.8})$$

The limit of f'_c is taken to be 10 ksi for all seismic regions:

$$f'_c \leq 10 \text{ ksi.} \quad (\text{AASHTO 5.1, 5.4.2.1})$$

In calculating the shear reinforcement for seismic moment resisting frames in Zone 3 and 4, a limit was imposed on the f_{ys} as

$$f_{ys} \leq f_y. \quad (\text{AASHTO 5.10.11.4.1d})$$

The maximum of all 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:

$$\frac{A_v}{s} \geq 0.45 \left[\frac{A_g}{A_c} - 1 \right] \frac{f'_c}{f_{yh}} \frac{h_{core}}{4} \quad (\text{AASHTO 5.7.4.6})$$

In potential plastic hinge locations, as described later, of Seismic moment resisting frames in Zones 2, 3 and 4, the minimum area of circular hoops and transverse stirrups is imposed as follows:

$$\frac{A_v}{s} \geq 0.16 \left[0.5 + \frac{1.25 P_u}{f'_c A_g} \right] \frac{f'_c}{f_{yh}} \frac{h_{core}}{4} \quad (\text{Hoops}) \quad (\text{AASHTO 5.10.11.4.1d})$$

$$\frac{A_v}{s} \geq 0.30 \left[\frac{A_g}{A_c} - 1 \right] \frac{f'_c}{f_{yh}} h_{core} \quad (\text{Stirrups}) \quad (\text{AASHTO 5.10.11.4.1d})$$

$$\frac{A_v}{s} \geq 0.12 \left[0.5 + \frac{1.25 P_u}{f'_c A_g} \right] \frac{f'_c}{f_{yh}} h_{core} \quad (\text{Stirrups}) \quad (\text{AASHTO 5.10.11.4.1d})$$

In the above equations, the following limits are imposed:

$$\left[0.5 + \frac{1.25 P_u}{f'_c A_g} \right] \geq 1.0 \quad (\text{AASHTO 5.10.11.4.1d})$$

$$f_{yh} \leq f_y \quad (\text{AASHTO 5.10.11.4.1d})$$

For the definition of the potential plastic hinge, it is assumed in the current version of the program that any beam and column segment near the joint is a potential plastic hinge. The length of the plastic hinge, L_{hinge} , in a column is taken as follows:

$$L_{hinge} = \max\{h, b, l/6, 18''\} \quad (\text{AASHTO 5.10.11.4.1c})$$

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

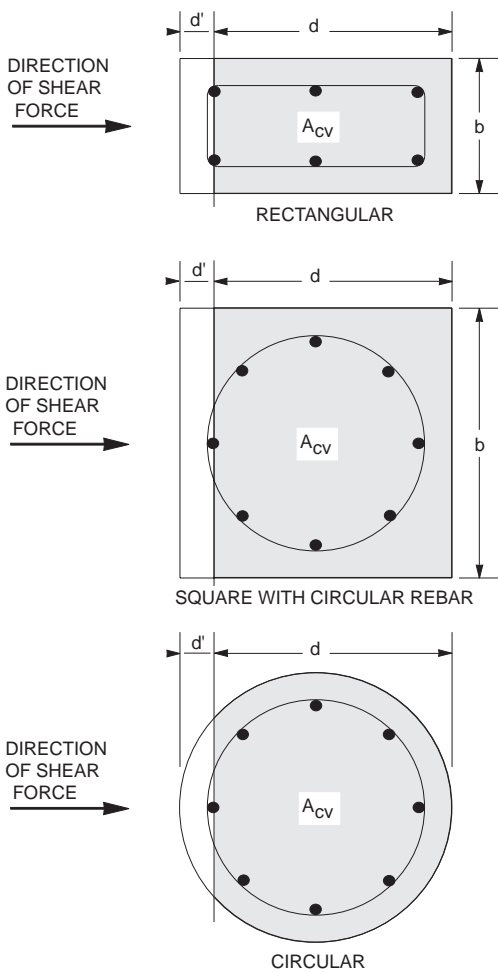


Figure IV-2
Shear Stress Area, A_{cv}

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, shears, load combination factors, and other criteria described below. The reinforcement requirements are calculated at a user defined number of check/design stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

Design Beam Flexural Reinforcement

The beam top and bottom flexural steel is designed at check/design stations along the beam span. In designing the flexural reinforcement for the major moment for a particular beam for a particular section, the following steps are involved:

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement of moment resisting concrete beams, the factored moments for each load combination at a particular beam section 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_u^+ and maximum negative M_u^- factored moments 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- or a T-beam.

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 IV-3 (AASHTO 5.7). Furthermore it is assumed that the maximum depth of compression block is limited to $0.42d$ (AASHTO 5.7.3.3.1). When the applied moment exceeds the moment capacity at this limiting condition, the area of compression reinforcement is calculated on the assumption that the additional moment will be carried by compression and additional tension reinforcement.

In designing the beam flexural reinforcement, the following limits are imposed on the steel tensile strength and the concrete compressive strength:

$$f'_c \leq 10 \text{ ksi.} \quad (\text{AASHTO 5.1, 5.4.2.1})$$

$$f_y \leq 75 \text{ ksi.} \quad (\text{AASHTO 5.4.3.1})$$

The design procedure used by SAP2000, for both rectangular and flanged sections (L- and T-beams) is summarized below. All the beams are designed for major direction flexure and shear only.

Design for Rectangular Beam

In designing for a factored negative or positive moment, M_u , (i.e. designing top or bottom steel) the depth of the compression block is given by a (see Figure IV-3), is computed as,

$$a = d - \sqrt{d^2 - \frac{2|M_u|}{0.85f'_c\phi b}}, \quad (\text{AASHTO 5.7.2.1})$$

where, the value of ϕ is 0.90 (AASHTO 5.5.4.2.1) in the above and the following equations. Also β_1 and c_b are calculated as follows:

$$\beta_1 = 0.85 - 0.05(f'_c - 4), \quad 0.65 \leq \beta_1 \leq 0.85, \quad (\text{AASHTO 5.7.2.2})$$

$$c_b = \frac{\varepsilon_c E_s}{\varepsilon_c E_s + f_y} d = \frac{87}{87 + f_y} d. \quad (\text{AASHTO 5.7.2.1})$$

The maximum allowed depth of the compression block is given by

$$a_{max} = \min\{\beta_1 c_b, 0.42d\}. \quad (\text{AASHTO 5.7.2.2, 5.7.3.3.1})$$

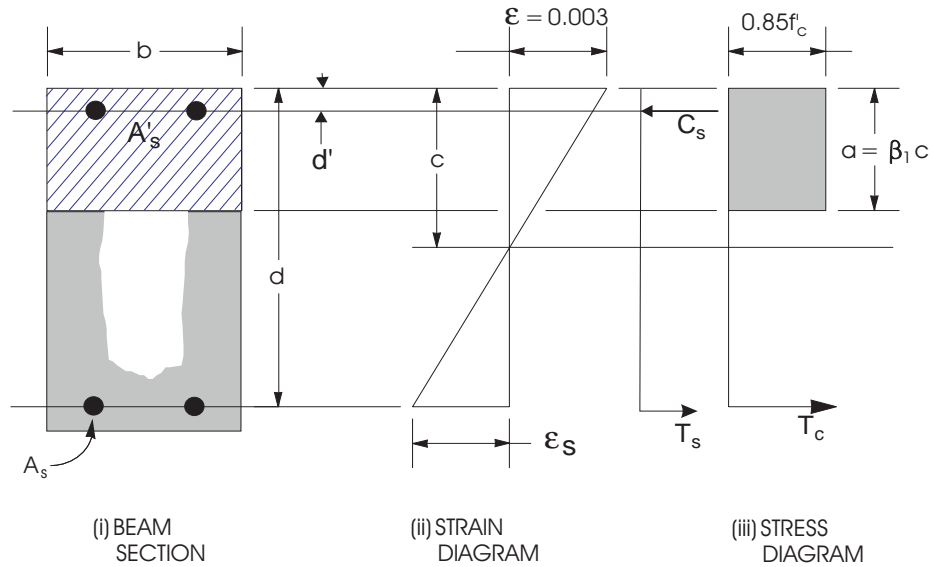


Figure IV-3
Design of Rectangular Beam Section

- If $a \leq a_{max}$ (AASHTO 5.7.3.3.1), the area of tensile steel reinforcement is then given by

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2} \right)}$$

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

- If $a > a_{max}$ (AASHTO 5.7.3.3.1), compression reinforcement is required and is calculated as follows:

- The compressive force developed in concrete alone is given by

$$C = 0.85 f'_c b a_{max}, \text{ and} \quad (\text{AASHTO 5.7.2.2})$$

the moment resisted by concrete compression and bottom steel is

$$M_{uc} = C \left(d - \frac{a_{max}}{2} \right) \phi .$$

- Therefore the moment resisted by compression steel and tensile steel is

$$M_{us} = M_u - M_{uc} .$$

- So the required compression steel is given by

$$A'_s = \frac{M_{us}}{(f'_s - \alpha_1 f'_c)(d - d')\phi} , \text{ where}$$

$$f'_s = 0.003 E_s \left[\frac{c - d'}{c} \right] . \quad (\text{AASHTO 5.7.2.1})$$

- The required tensile steel for balancing the compression in concrete is

$$A_{s1} = \frac{M_{uc}}{f_y \left(d - \frac{a_{max}}{2} \right) \phi} , \text{ and}$$

the tensile steel for balancing the compression in steel is given by

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

- 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 bottom and A'_s is to be placed at top if M_u is positive, and vice versa.

Design for Flexure of a T-Beam

(i) Flanged Beam Under Negative Moment

In designing for a factored negative moment, M_u , (i.e. designing top steel), the calculation of the steel area is exactly the same as above, i.e., no T-Beam data is to be used.

(ii) Flanged Beam Under Positive Moment

If $M_u > 0$, the depth of the compression block is given by (see Figure IV-4).

$$a = d - \sqrt{d^2 - \frac{2 M_u}{0.85 f'_c \phi b_f}}, \quad (\text{AASHTO 5.7.2.1})$$

where, the value of ϕ is 0.90 (AASHTO 5.5.4.2.1) in the above and the following equations. Also β_1 and c_b are calculated as follows:

$$\beta_1 = 0.85 - 0.05 (f'_c - 4), \quad 0.65 \leq \beta_1 \leq 0.85, \quad (\text{AASHTO 5.7.2.1})$$

$$c_b = \frac{87}{87 + f_y} d. \quad (\text{AASHTO 5.7.2.1})$$

The maximum allowed depth of the compression block is given by

$$a_{max} = \min\{\beta_1 c_b, 0.42d\}. \quad (\text{AASHTO 5.7.2.2, 5.7.3.3.1})$$

- If $a \leq \beta_1 d_s$ (AASHTO 5.7.3.2.2), the subsequent calculations for A_s are exactly the same as previously done for the rectangular section design. However, in this case b_f is taken as the width of the beam. Whether compression reinforcement is required depends on whether $a > a_{max}$.
- If $a > \beta_1 d_s$ (AASHTO 5.7.3.2.2), calculation for A_s is done 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 Figure IV-4,

$$C_f = 0.85 f'_c (b_f - b_w) \beta_1 d_s. \quad (\text{AASHTO 5.7.3.2.2})$$

Therefore, $A_{s1} = \frac{C_f}{f_y}$ and the portion of M_u that is resisted by the flange is

given by

$$M_{uf} = C_f \left(d - \frac{\beta_1 d_s}{2} \right) \phi.$$

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

$$M_{uw} = M_u - M_{uf}.$$

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

$$a_1 = d - \sqrt{d^2 - \frac{2 M_{uw}}{0.85 f'_c \phi b_w}}.$$

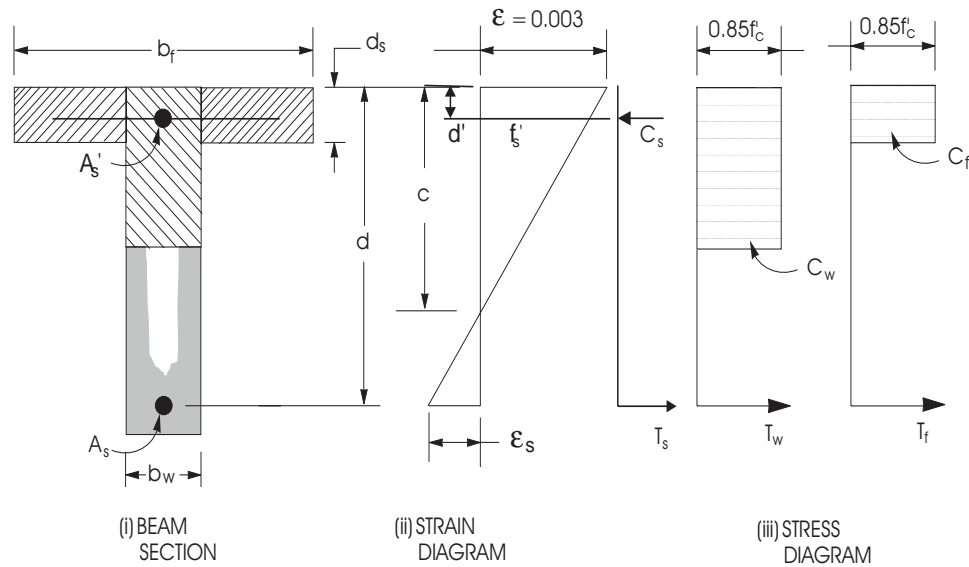


Figure IV-4
Design of a T-Beam Section

- If $a_1 \leq a_{max}$ (AASHTO 5.7.3.3.1), the area of tensile steel reinforcement is then given by

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

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

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

- If $a_1 > a_{max}$ (AASHTO 5.7.3.3.1), compression reinforcement is required and is calculated as follows:

- The compressive force in web concrete alone is given by

$$C = 0.85 f_c' b a_{max} . \quad (\text{AASHTO 5.7.2.2})$$

- Therefore the moment resisted by concrete web and tensile steel is

$$M_{uc} = C \left(d - \frac{a_{max}}{2} \right) \phi , \text{ and}$$

the moment resisted by compression steel and tensile steel is

$$M_{us} = M_{uw} - M_{uc} .$$

– Therefore, the compression steel is computed as

$$A'_s = \frac{M_{us}}{(f'_s - \alpha_1 f'_c)(d - d')\phi} , \text{ where}$$

$$f'_s = 0.003 E_s \left[\frac{c - d'}{c} \right] \leq f_y . \quad (\text{AASHTO 5.7.2.1})$$

– The tensile steel for balancing compression in web concrete is

$$A_{s2} = \frac{M_{uc}}{f_y \left(d - \frac{a_{max}}{2} \right) \phi} , \text{ and}$$

the tensile steel for balancing compression in steel is

$$A_{s3} = \frac{M_{us}}{f_y (d - d')\phi} .$$

– The 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 bottom and A'_s is to be placed at top.

Minimum and Maximum Tensile Reinforcement

The minimum temperature and shrinkage flexural tensile steel provided in a rectangular section is given by:

$$A_s \geq \min \left\{ \frac{0.11}{f_y} b_w d \quad \text{and} \quad 0.0015 b_w d \right\} \text{ or} \quad (\text{AASHTO 5.10.8.2})$$

The minimum flexural tensile steel provided to prevent premature flexural failure in a rectangular section is given by:

$$A_s \geq \min \left\{ 0.03 \frac{f'_c}{f_y} b_w d \quad \text{and} \quad \frac{4}{3} A_{s(\text{required})} \right\} \text{ or} \quad (\text{AASHTO 5.7.3.3.2})$$

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

$$A_s \leq \begin{cases} 0.04 b d & \text{Rectangular beam} \\ 0.04 b_w d & \text{T-beam} \end{cases}$$
$$A'_s \leq \begin{cases} 0.04 b d & \text{Rectangular beam} \\ 0.04 b_w d & \text{T-beam} \end{cases}$$

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 following steps are involved:

- Determine the factored forces acting on the section, M_u and V_u . Note that M_u 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 moment resisting frames in seismic zones 3 and 4, the shear design of the beams is also based upon the overstrength moment capacities of the members in addition to the factored moments (AASHTO 3.10.9.4.3).

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

Determine Shear Force and Moment

- In the design of the beam shear reinforcement of **moment resisting concrete frames in seismic Zones 1 and 2**, 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 shear design of **moment resisting frames in seismic Zones 3 and 4 (seismic design)**, the following are checked in addition to the requirement for the moment resisting frames in Zones 1 and 2. In the design of moment resisting concrete frames in seismic Zones 3 and 4, the design shear force in a beam, V_u , is also calculated from the overstrength moment capacities of the beam (AASHTO 3.10.9.4.3). The design shear force V_u is then given by

$$V_u = V_p + V_{D+L} \quad (\text{AASHTO 3.10.9.4.3})$$

where V_p is the shear force obtained by applying the calculated overstrength ultimate moment capacities at the two ends of the beams acting in two opposite directions. Therefore, V_p is the maximum of V_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

M_I^+, M_I^- = Positive and negative moment capacities at end I of the beam obtained by multiplying the nominal resistance by 1.3,

M_J^+, M_J^- = Positive and negative moment capacities at end J of the beam obtained by multiplying the nominal resistance by 1.3, and

L = Clear span of beam.

V_{D+L} is the contribution of shear force from the in-span distribution of gravity loads. See also Table IV-2 for details.

Determine Concrete Shear Capacity

Given the design force set M_u and V_u , the shear capacity provided by the concrete alone, v_c , is calculated as follows:

- For designing **moment resisting concrete frames in any seismic zone**, v_c is set to:

$$v_c = 0.0316 \beta \sqrt{f'_c}, \quad (\text{AASHTO 5.8.3.3})$$

where, β is a factor indicating the ability of diagonally cracked concrete to transmit tension. It is a function of stress condition and its approximate value is 2.0 (AASHTO 5.8.3.4.1). It is computed from AASHTO Table 5.8.3.4.2-1 through an iterative procedure.

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular section as,

$$v = \frac{V_u}{b_w d}.$$

For other types of sections $b_w d$ is replaced by A_{cv} , the effective shear area which is shown in Figure IV-2.

- The average shear stress, v , is limited to a maximum limit, v_{max} , given by

$$v_{max} = 0.25 f_c' \quad (\text{AASHTO 5.8.3.3})$$

- The shear reinforcement per unit spacing is computed as follows:

If $v \leq \phi (v_c / 2)$,

$$\frac{A_v}{s} = 0, \quad (\text{AASHTO 5.8.2.4})$$

else if $\phi (v_c / 2) < v \leq \phi (v_c + 0.0316 \sqrt{f_c'} \cot \theta)$,

$$\frac{A_v}{s} = \frac{0.0316 \sqrt{f_c'} b_w}{f_{yh}}, \quad (\text{AASHTO 5.8.2.5})$$

else if $\phi (v_c + 0.0316 \sqrt{f_c'} \cot \theta) < v \leq \phi v_{max}$,

$$\frac{A_v}{s} = \frac{(v - \phi v_c) b_w}{\phi f_{yh} \cot \theta}, \quad (\text{AASHTO 5.8.3.3})$$

else if $v > v_{max}$,

a failure condition is declared. (AASHTO 5.8.3.3)

Here θ is an angle of inclination of diagonal compressive stresses. It is a function of the current stress condition and its approximate value is 45 degrees (AASHTO 5.8.3.4.1). It is computed from AASHTO Table 5.8.3.4.2-1 through an iterative procedure. Here the default value of ϕ , the strength reduction factor, is 0.90 (AASHTO 5.5.4.2.1).

The limit of f_{yh} is taken to be 60 ksi for all frames:

$$f_{yh} \leq 60 \text{ ksi.} \quad (\text{AASHTO 5.8.2.8})$$

The limit of f_c' is taken to be 10 ksi for all seismic regions:

$$f_c' \leq 10 \text{ ksi.} \quad (\text{AASHTO 5.1, 5.4.2.1})$$

The maximum of all 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.

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

Type of Check/Design	Moment Resisting Frames in Zones 1 and 2 (non-Seismic)	Moment Resisting Frames in Zones 3 and 4 (Seismic)
φ In Compression	$\phi = 0.75$, if $P_u > 0.10 f'_c$	$\phi = 0.50$, if $P_u > 0.20 f'_c$
Column Check (Interaction)	NLD ^a Combinations	NLD ^a Combinations
Column Design (Interaction)	NLD ^a Combinations $0.135 f'_c / f_y < \rho < 0.08$	NLD ^a Combinations $0.01 < \rho < 0.06$
Column Shears	NLD ^a Combinations $v_c = 0.0316 \beta \sqrt{f'_c}$ Minimum volumetric shear reinforcement in potential plastic hinge only for Zone 2	NLD ^a Combinations and Column capacity shear (V_p) with overstrength factor 1.3 $v_c = 0.0316 \beta \sqrt{f'_c}$ if $P_u \geq 0.1 f'_c A_g$ $v_c = 0.0316 \beta \sqrt{f'_c} \frac{P_u}{0.1 f'_c A_g} \geq 0$ otherwise Minimum volumetric shear reinforcement in potential plastic hinge for both Zone 3 and 4
Beam Design (Flexure)	NLD ^a Combinations	NLD ^a Combinations $\rho \leq 0.025$ $\rho \geq \frac{3\sqrt{f'_c}}{f_y}, \rho \geq \frac{200}{f_y}$
Beam Minimum Flexural Reinforcement	$\frac{0.11}{f_y} b_w d$, $0.0015 b_w d$, $0.03 \frac{f'_c}{f_f}, \frac{4}{3} A_{s,required}$	$\frac{0.11}{f_y} b_w d$, $0.0015 b_w d$, $0.03 \frac{f'_c}{f_f}, \frac{4}{3} A_{s,required}$
Beam Design (Shear)	NLD ^a Combinations $v_c = 0.0316 \beta \sqrt{f'_c}$	NLD ^a Combinations Beam capacity shear (V_p) with overstrength factor 1.3 $v_c = 0.0316 \beta \sqrt{f'_c}$

^a NLD = Number of specified loading

Table IV-2
Design Criteria Table

Chapter V

Design for CSA-A23.3-94

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the Canadian code, **CSA-A23.3-94** (CSA 1994). Various notations used in this chapter are listed in Table V-1.

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

SAP2000 provides options to design or check Ordinary, Nominal (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 the following sections.

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Newton-Millimeter-Second** units unless otherwise noted.

A_{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

Table V-1
List of Symbols Used in the Canadian code

M_2	Larger factored end moment in a column, N-mm
M_c	Factored moment to be used in design, N-mm
M_{ns}	Nonsway 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
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_0	Axial load capacity at zero eccentricity, N
P_f	Factored axial load at section, 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	is a factor indicating the ability of diagonally cracked concrete to transmit tension
θ	An 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 nonsway 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

Table V-1*List of Symbols Used in the Canadian code (continued)*

Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs 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, then the following load combinations may have to be considered (CSA 8.3):

$$1.25 \text{ DL} \\ 1.25 \text{ DL} + 1.50 \text{ LL} \quad (\text{CSA 8.3.2})$$

$$1.25 \text{ DL} \pm 1.50 \text{ WL} \\ 0.85 \text{ DL} \pm 1.50 \text{ WL} \\ 1.25 \text{ DL} + 0.7 (1.50 \text{ LL} \pm 1.50 \text{ WL}) \quad (\text{CSA 8.3.2})$$

$$1.00 \text{ DL} \pm 1.50 \text{ EL} \\ 1.00 \text{ DL} + (0.50 \text{ LL} \pm 1.00 \text{ EL}) \quad (\text{CSA 8.3.2})$$

These are also the default design load combinations in SAP2000 whenever the Canadian Code is used.

In generating the above default loading combinations, the importance factor is taken as 1. The user should use other appropriate loading combinations if roof live load is separately treated, other types of loads are present, or pattern live loads are to be considered.

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

Strength Reduction Factors

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

$$\phi_c = 0.60 \text{ for concrete and} \quad (\text{CSA 8.4.2})$$

$$\phi_s = 0.85 \text{ for steel.} \quad (\text{CSA 8.4.3})$$

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).

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 II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement — 1 to 8 percent for Ordinary and Nominal moment 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 the above-mentioned steps.

Generation of Biaxial Interaction Surfaces

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

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 II-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 upon the general principles of ultimate strength design (CSA 10.1), and allows for any doubly symmetric rectangular, square, or circular column section.

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\epsilon_s E_s$, 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). See Figure V-1.

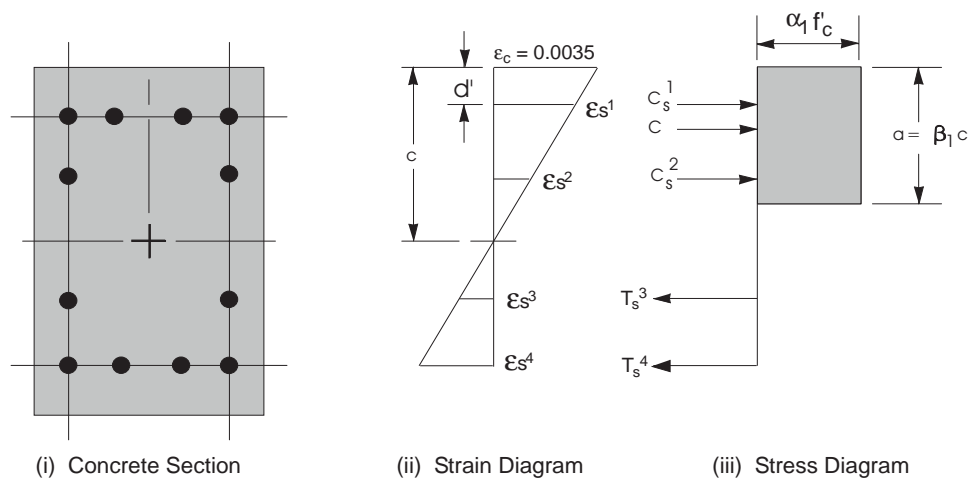


Figure V-1
Idealization of Stress and Strain Distribution in a Column Section

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 V-1. The depth of the stress block is $\beta_1 c$, where

$$\alpha_1 = 0.85 - 0.0015 f'_c, \quad \alpha_1 \geq 0.67. \quad (\text{CSA 10.1.7})$$

$$\beta_1 = 0.97 - 0.0025 f'_c, \quad \beta_1 \geq 0.67. \quad (\text{CSA 10.1.7})$$

The limit of f'_c is taken to be 80 MPa for Ordinary moment resisting frames and 55 MPa for Nominal and Ductile moment resisting frames:

$$f'_c \leq 80 \text{ MPa} \quad (\text{Ordinary}) \quad (\text{CSA 8.6.1.1})$$

$$f'_c \leq 55 \text{ MPa} \quad (\text{Nominal and Ductile}) \quad (\text{CSA 21.2.3.1})$$

The limit of f_y is taken to be 500 MPa for all frames:

$$f_y \leq 500 \text{ MPa.} \quad (\text{CSA 8.5.1})$$

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

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

$$P_{r,max} = 0.85 [\varphi_c \alpha_1 f'_c (A_g - A_{st}) + \varphi_s f_y A_{st}] (\text{spiral column}). \quad (\text{CSA 10.10.4})$$

Check Column Capacity

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 following steps are involved:

- Determine the factored moments and forces from the analysis 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 the above-mentioned steps.

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”.

Determine Moment Magnification Factors

The moment magnification factors are applied in two different 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, SAP2000 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

$$\frac{kL}{r} \geq \frac{25 - 10 M_1/M_2}{\sqrt{P_f/f'_c A_g}}, \quad \text{for braced frames, and} \quad (\text{CSA 10.15.2})$$

$$\frac{kL}{r} \geq \frac{35}{\sqrt{P_f/f'_c A_g}}, \quad \text{for unbraced frames.} \quad (\text{CSA 10.16.4})$$

However, SAP2000 considers individual member stability effect for all compression column elements.

Lateral Drift Effect

For all frames, the moment magnification factor for lateral drift effect is applied only to the “sway” moment in SAP2000.

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

The moment obtained from analysis is separated into two components: the sway (M_s) and the nonsway (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 side sway.

The moment magnification factors in the major and minor directions can, in general, be different. The moment magnification factors for moments causing sidesway, δ_{xx} and δ_{yy} , can be taken as 1.0 if a P- Δ analysis is carried out (CSA 10.16.3.1). **The program assumes that a P- Δ analysis has been performed in SAP2000 and, therefore, moment magnification factors for moments causing sidesway are taken as unity.** For the P- Δ analysis the load should correspond to a load combination of $(1.25 \text{ DL} + 1.50 \text{ LL})/\phi_m$ (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). SAP2000 analysis uses EI of gross cross-sectional area in conjunction with a multiplication factor. The multipli-

ation factors are defined in a section-by-section basis, so that 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} .

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,

$$M_c = \delta_b M_2 \geq M_2, \quad (\text{CSA 10.15.3})$$

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 either 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

$$\delta_b = \frac{C_m}{1 - \frac{P_f}{\Phi_m P_c}} \geq 1.0, \text{ where} \quad (\text{CSA 10.15.3})$$

$$\Phi_m = 0.75, \quad (\text{CSA 10.15.3})$$

$$P_c = \frac{\pi^2 EI}{(kL)^2}, \quad (\text{CSA 10.15.3})$$

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

EI is associated with a particular column direction given by

$$EI = 0.25 E_c I_g, \text{ and} \quad (\text{CSA 10.15.3})$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4, \quad (\text{CSA 10.15.3.1})$$

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 above 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 10.15.3.2). 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 $C_m M / P_f$ is at least $(15 + 0.03 h)$ mm about each axis separately, where h is the dimension of the column in mm unit in the corresponding direction (CSA 10.15.3).

$$C_m M \geq P_f (15 + 0.03 h) \quad (\text{CSA 10.15.3})$$

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

The above 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. See Figure II-4. 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 .

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 which 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 II-3. 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.

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 II-3. 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, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

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 following steps are involved:

- 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 Nominal moment resisting concrete frames, the shear design of the columns is also based upon the probable and nominal moment capacities, respectively, of the members in addition to the factored moments (CSA 21.7.2.2, 21.9.2.3). 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 the above-mentioned steps.

Determine Section Forces

- In the design of the column shear reinforcement of an **Ordinary 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 SAP2000 analysis 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 the Ordinary 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 21.7.2.2).

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

$$V_f = V_p + V_{D+L} \quad (\text{CSA 21.7.2.2})$$

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_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L}, \text{ where}$$

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 ($\phi_s = \phi_c = 1.0$),

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 ($\phi_s = \phi_c = 1.0$), and

L = Clear span of column.

For Ductile moment resisting frames α is taken as 1.25 (CSA 21.1). 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.

- In the design of **Nominal moment resisting frames** (seismic), the shear capacity of the column is also checked for the nominal shear due to the nominal ($\phi_s = \phi_c = 1.0$) moment capacities and the factored gravity load (CSA 21.9.2.3), in addition to the design checks required for Ordinary 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 21.0) rather than 1.25. The design shear strength is enforced not to be greater than the factored shear force with the seismic load doubled (CSA 21.9.2.3, 21.9.1). See also Table V-2 for details.

Determine Concrete Shear Capacity

Given the design force set M_f , P_f and V_f , the shear capacity provided by the concrete alone, v_c , is calculated using the “General Method” or “Compression Field Theory” as follows (CSA 11.4):

- For designing **Ordinary and Nominal moment resisting concrete frames**, v_c is set to:

$$v_c = 1.3 \lambda \beta \sqrt{f'_c}, \quad (\text{CSA 11.4.3.1})$$

where, β is a factor indicating the ability of diagonally cracked concrete to resist shear. It is a function of stress condition and its average value is 0.2 usually with a range of 0.1 to 0.4 (CSA 11.4.4). It is computed from CSA Table 11-1 through an iterative procedure. In computing the β factor, the following two parameters are used:

$$\varepsilon_x = \frac{|M_f|/d_v + 0.5 N_f + 0.5 V_f \cot \theta}{E_s A_s}, \quad 0 \leq \varepsilon_x \leq 0.002 \quad (\text{CSA 11.4.6})$$

$$v_{ratio} = \frac{V_f / b_w d_v}{\lambda \phi_c f'_c} \quad (\text{CSA 11.4.5})$$

In the above equations d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.9d$.

$$d_v = 0.9d \quad (\text{CSA 11.0})$$

Here θ is an angle of inclination of diagonal compressive stresses. It is a function of the current stress condition and its approximate value is 34 degrees with a range of 27 degrees to 45 degrees (CSA 11.4.4). In a similar procedure to calculate the β factor, θ is also computed from CSA Table 11-1 using the parameters ε_x and v_{ratio} through an iterative procedure.

The shear strength factor, λ , is taken to be 1, assuming normal density concrete (CSA 11.0, 8.6.5).

- For designing **Ductile moment resisting concrete frames**, v_c is computed based on the assumption that $\varepsilon_x = 0.002$ (CSA 21.7.3.1b, 11.4.3.1). Otherwise the procedure for computing v_c is the same as that for Ordinary and Nominal moment resisting frames.

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular section as,

$$v = V_f / (b_w d_v) . \quad (\text{CSA 11.4.5})$$

In the above equation d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.9d$ for rectangular sections.

$$d_v = 0.9d \quad (\text{CSA 11.0})$$

For other types of sections $b_w d_v$ is replaced by 0.9 times A_{cv} , the effective shear area, which is shown in Figure V-2.

- The average shear stress, v , is limited to a maximum limit, v_{max} , given by

$$v_{max} = 0.25 f'_c . \quad (\text{CSA 11.4.3})$$

- The shear reinforcement per unit spacing is computed as follows:

$$\text{If } v \leq \phi_c (v_c / 2) ,$$

$$\frac{A_v}{s} = 0 , \quad (\text{CSA 11.2.8.1})$$

$$\text{else if } \phi_c (v_c / 2) < v \leq \left(\phi_c v_c + \phi_s 0.06 \sqrt{f'_c} \cot \theta \right) ,$$

$$\frac{A_v}{s} = \frac{0.06 \sqrt{f'_c} b_w}{f_{yh}} , \quad (\text{CSA 11.2.8.4})$$

$$\text{else if } \left(\phi_c v_c + \phi_s 0.06 \sqrt{f'_c} \cot \theta \right) < v \leq \phi_c v_{max} ,$$

$$\frac{A_v}{s} = \frac{(v - \phi_c v_c) b_w}{\phi_s f_{yh} \cot \theta} , \quad (\text{CSA 11.4.3.2})$$

$$\text{else if } v > \phi_c v_{max} ,$$

$$\text{a failure condition is declared.} \quad (\text{CSA 11.3.4})$$

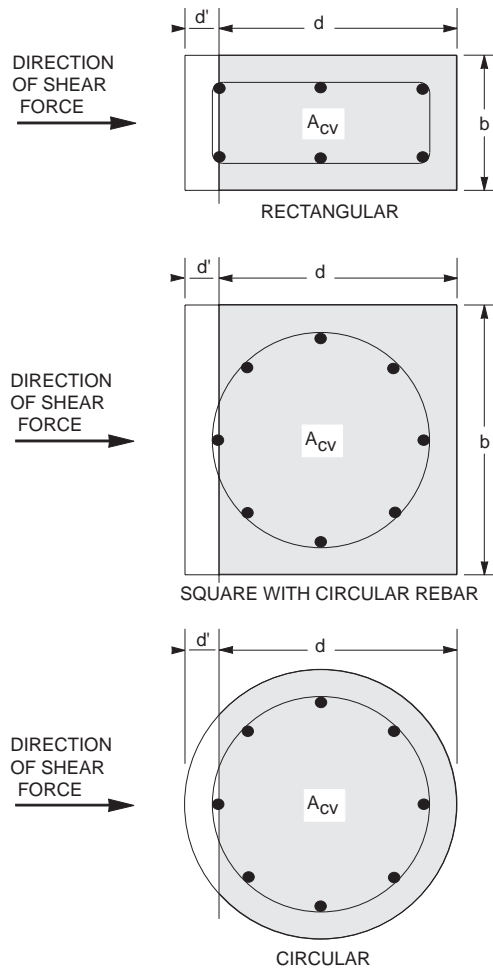


Figure V-2
Shear Stress Area, A_{cv}

The limit of f'_c is taken to be 80 MPa for Ordinary moment resisting frames and 55 MPa for Nominal and Ductile moment resisting frames:

$$f'_c \leq 80 \text{ MPa} \quad (\text{Ordinary}) \quad (\text{CSA 8.6.1.1})$$

$$f'_c \leq 55 \text{ MPa} \quad (\text{Nominal and Ductile}) \quad (\text{CSA 21.2.3.1})$$

The limit of f_{yh} is taken to be 500 MPa for all frames:

$$f_{yh} \leq 500 \text{ MPa.} \quad (\text{CSA 8.5.1})$$

The maximum of all 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:

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

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

$$\frac{A_v}{s} \geq 0.12 \frac{f'_c}{f_{yh}} \frac{h_{core}}{4} \quad (\text{Hoops}) \quad (\text{CSA 21.4.4.2})$$

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

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

$$\frac{A_v}{s} \geq 0.30 \left[\frac{A_g}{A_{core}} - 1 \right] \frac{f'_c}{f_{yh}} h_{core} \quad (\text{Stirrups}) \quad (\text{CSA 21.4.4.2})$$

For the definition of the potential plastic hinge, it is assumed in the current version of the program that any beam and column segment near the joint is a potential plastic hinge. The length of the plastic hinge, L_{hinge} , in a column is taken as follows:

$$L_{hinge} = \max\{h, b, l/6, 450 \text{ mm}\} \quad (\text{CSA 21.4.4.5})$$

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

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, shears, load combination factors and other criteria described below. The reinforcement requirements are calculated at a user defined number of check stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

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 following steps are involved:

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement of Ductile, Nominal, or Ordinary 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.

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 V-3 (CSA 10.1.7). 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 additional moment will be carried by compression and additional tension reinforcement.

In designing the beam flexural reinforcement, the limit of f'_c is taken to be 80 MPa for Ordinary moment resisting frames and 55 MPa for Nominal and Ductile moment resisting frames:

$$f'_c \leq 80 \text{ MPa} \quad (\text{Ordinary}) \quad (\text{CSA 8.6.1.1})$$

$$f'_c \leq 55 \text{ MPa} \quad (\text{Nominal and Ductile}) \quad (\text{CSA 21.2.3.1})$$

The limit of f_y is taken to be 500 MPa for all frames:

$$f_y \leq 500 \text{ MPa.} \quad (\text{CSA 8.5.1})$$

The design procedure used by SAP2000 for both rectangular and flanged sections (L- and T-beams) is summarized below. It is assumed that the design ultimate axial force in a beam is negligible, hence all the beams are designed for major direction flexure and shear only.

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 V-3, where,

$$a = d - \sqrt{d^2 - \frac{2|M_f|}{\alpha_1 f'_c \phi_c b}}, \quad (\text{CSA 10.1})$$

where the value of ϕ_c is 0.60 (CSA 8.4.2) in the above and following equations. Also α_1 , β_1 , and c_b are calculated as follows:

$$\alpha_1 = 0.85 - 0.0015f'_c \geq 0.67, \quad (\text{CSA 10.1.7})$$

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

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

The balanced depth of the compression block is given by

$$a_b = \beta_1 c_b. \quad (\text{CSA 10.1.7})$$

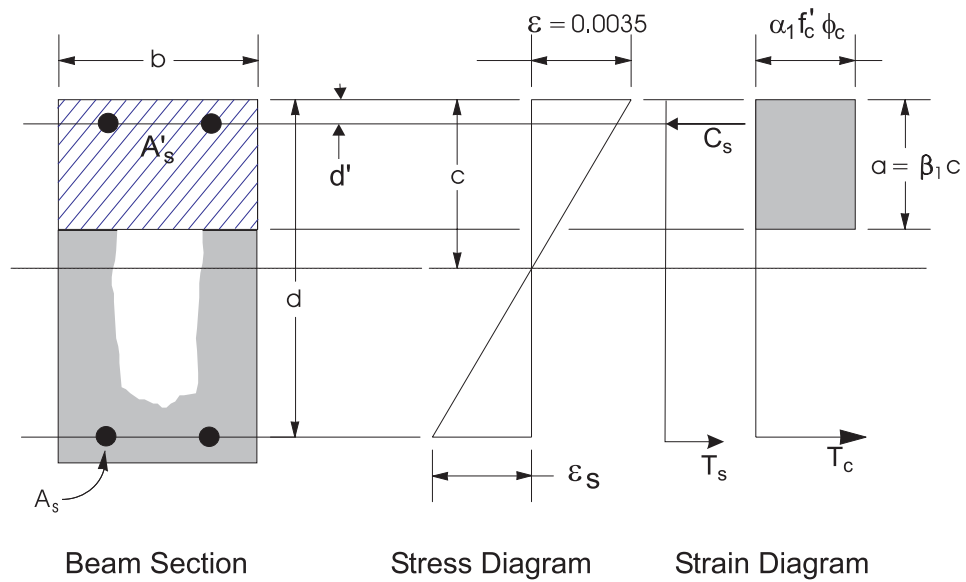


Figure V-3
Design of a Rectangular Beam Section

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

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

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

$$C = \varphi_c \alpha_1 f'_c b a_b, \text{ and} \quad (\text{CSA 10.1.7})$$

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

$$A'_s = \frac{M_{fs}}{(\varphi_s f'_s - \varphi_c \alpha_1 f'_c)(d - d')}, \text{ where}$$

$$f'_s = 0.0035 E_s \left[\frac{c - d'}{c} \right] \leq f_y. \quad (\text{CSA 10.1.2 and CSA 10.1.3})$$

- 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.

Design for Flexure of a T-Beam**(i) 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_w .

(ii) Flanged Beam Under Positive Moment

If $M_f > 0$, the depth of the compression block is given by (see Figure V-4).

$$a = d - \sqrt{d^2 - \frac{2 M_f}{\alpha_1 f'_c \phi_c b_f}} \quad (\text{CSA 10.1})$$

where the value of ϕ_c is 0.60 (CSA 8.4.2) in the above and following equations. See Figure V-4. Also α_1 , β_1 , and c_b are calculated as follows:

$$\alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67, \quad (\text{CSA 10.1.7})$$

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

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

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

$$a_b = \beta_1 c_b. \quad (\text{CSA 10.1.7})$$

- If $a \leq d_s$, the subsequent calculations for A_s are exactly the same as previously done 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 whether $a > a_b$.
- If $a > d_s$, calculation for A_s is done 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 Figure V-4,

$$C_f = \alpha_1 f'_c (b_f - b_w) d_s. \quad (\text{CSA 10.1.7})$$

Therefore, $A_{s1} = \frac{C_f \phi_c}{f_y \phi_s}$ and the portion of M_f that is resisted by the flange is

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

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

$$a_1 = d - \sqrt{d^2 - \frac{2M_{fw}}{\alpha_1 f'_c \Phi_c b_w}} . \quad (\text{CSA 10.1})$$

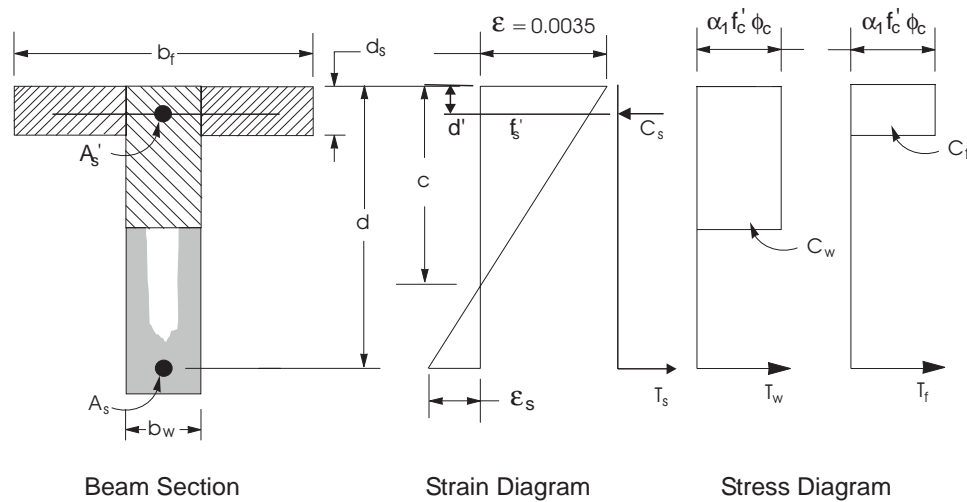


Figure V-4
Design of a T-Beam Section

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

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

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

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

$$C = \phi_c \alpha_1 f'_c b a_b , \text{ and} \quad (\text{CSA 10.1.7})$$

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

$$A'_s = \frac{M_{fs}}{(\phi_s f'_s - \phi_c \alpha_1 f'_c)(d - d')} , \text{ where}$$

$$f'_s = 0.0035 E_s \left[\frac{c - d'}{c} \right] \leq f_y . \quad (\text{CSA 10.1.2 and CSA 10.1.3})$$

- The tensile steel for balancing compression in web concrete is

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

the tensile steel for balancing compression in steel is

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

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.

Minimum and Maximum Tensile Reinforcement

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

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

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

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

$$A_s \geq 0.004(b - b_w) d_s. \quad (\text{CSA 10.5.3.1})$$

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:

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

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

Special Consideration for Seismic Design

For Ductile moment resisting concrete frames (seismic design), the beam design satisfies the following additional conditions (see also Table V-2 for 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 $A_{s(\text{min})}$.

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

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

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

- At any end (support) of the beam, the beam positive moment capacity (i.e. associated with the bottom steel) would not be less than $\frac{1}{2}$ 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 $\frac{1}{4}$ of the maximum of positive or negative moment capacities of any of the beam end (support) stations (CSA 21.3.2.2).

For Nominal 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 $\frac{1}{3}$ of the beam negative moment capacity at that end (CSA 21.9.2.1.1).
- Neither the negative moment capacity nor the positive moment capacity at any of the sections within the beam would be less than $\frac{1}{5}$ of the maximum of positive or negative moment capacities of any of the beam end (support) stations (CSA 21.9.2.1.1).

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 following steps are involved:

- 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 Nominal moment resisting frames, the shear design of the beams is also based on the probable and nominal moment capacities of the members, respectively (CSA 21.7.2.1, 21.9.2.3).

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

Determine Shear Force and Moment

- In the design of the beam shear reinforcement of an **Ordinary moment resisting 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 Ordinary moment resisting frames. The procedure for calculating the design shear force in a beam from probable moment capacities is the same as that described for a column in section “Column Design” on page 82. See also Table V-2 for more details. The design shear force V_f is then given by

$$V_f = V_p + V_{D+L} \quad (\text{CSA 21.7.2.1})$$

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_{P_1} and V_{P_2} , where

$$V_{P_1} = (M_I^- + M_J^+) / L, \text{ and}$$

$$V_{P_2} = (M_I^+ + M_J^-) / L, \text{ where}$$

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 ($\phi_s = \phi_c = 1.0$),

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 ($\phi_s = \phi_c = 1.0$),

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 ($\phi_s = \phi_c = 1.0$),

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 ($\phi_s = \phi_c = 1.0$), and

L = Clear span of beam.

The overstrength factor α is always taken as 1.25 for Ductile moment resisting frames (CSA 21.1). V_{D+L} is the contribution of shear force from the in-span distribution of gravity loads.

- In the design of **Nominal moment resisting frames** (seismic), the shear capacity of the beam is also checked for the nominal shear due to the nominal ($\phi_s = \phi_c = 1.0$) moment capacities and the factored gravity load (CSA 21.9.2.3), in addition to the design checks required for Ordinary 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 21.1) rather than 1.25. The design shear strength is enforced not to be greater than the factored shear force with the seismic load doubled (CSA 21.9.2.3, 21.9.1). See also Table V-2 for details.

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 using the “General Method” or “Compression Field Theory” as follows (CSA 11.4):

- For designing **Ordinary and Nominal moment resisting concrete frames**, v_c is set to:

$$v_c = 1.3 \lambda \beta \sqrt{f'_c}, \quad (\text{CSA 11.4.3.1})$$

where, β is a factor indicating the ability of diagonally cracked concrete to resist shear. It is a function of the stress condition and its average value is 0.2 usually with a range of 0.1 to 0.4 (CSA 11.4.4). It is computed from CSA Table 11-1 through an iterative procedure. In computing the β factor, the following two parameters are used:

$$\varepsilon_x = \frac{|M_f|/d_v + 0.5 V_f \cot \theta}{E_s A_s}, \quad 0 \leq \varepsilon_x \leq 0.002 \quad (\text{CSA 11.4.6})$$

$$v_{ratio} = \frac{V_f / b_w d_v}{\lambda \phi_c f'_c} \quad (\text{CSA 11.4.5})$$

In the above equations d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.9d$.

$$d_v = 0.9d \quad (\text{CSA 11.0})$$

Here θ is an angle of inclination of diagonal compressive stresses. It is a function of the current stress condition and its approximate value is 34 degrees with a range of 27 degrees to 45 degrees (CSA 11.4.4). In a similar procedure to cal-

culate the β factor, θ is also computed from CSA Table 11-1 using the parameters ε_x and v_{ratio} through an iterative procedure.

The shear strength factor, λ , is taken to be 1, assuming normal density concrete (CSA 11.0, 8.6.5).

- For designing **Ductile moment resisting concrete beams**, v_c is taken as zero, β is taken as zero, and θ is taken as 45 degrees (CSA 21.7.3.1a, 11.4.3.1).

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular section as,

$$v = \frac{V_f}{b_w d_v} \quad (\text{CSA 11.4.5})$$

In the above equation d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.9d$.

$$d_v = 0.9d \quad (\text{CSA 11.0})$$

For other types of sections $b_w d_v$ is replaced by 0.9 times A_{cv} , the effective shear area, which is shown in Figure V-2.

- The average shear stress, v , is limited to a maximum limit, v_{max} , given by

$$v_{max} = 0.25 f'_c \quad (\text{CSA 11.4.3})$$

- The shear reinforcement per unit spacing is computed as follows:

If $v \leq \phi_c (v_c/2)$,

$$\frac{A_v}{s} = 0, \quad (\text{CSA 11.2.8.1})$$

else if $\phi_c (v_c/2) < v \leq (\phi_c v_c + \phi_s 0.06 \sqrt{f'_c} \cot \theta)$,

$$\frac{A_v}{s} = \frac{0.06 \sqrt{f'_c} b_w}{f_{yh}}, \quad (\text{CSA 11.2.8.4})$$

else if $\left(\phi_c v_c + \phi_s 0.06 \sqrt{f'_c} \cot \theta \right) < v \leq \phi_c v_{max}$,

$$\frac{A_v}{s} = \frac{(v - \phi_c v_c) b_w}{\phi_s f_{yh} \cot \theta}, \quad (\text{CSA 11.4.3.2})$$

else if $v > \phi_c v_{max}$,

a failure condition is declared. (CSA 11.3.4)

The limit of f'_c is taken to be 80 MPa for Ordinary moment resisting frames and 55 MPa for Nominal and Ductile moment resisting frames:

$$f'_c \leq 80 \text{ MPa} \quad (\text{Ordinary}) \quad (\text{CSA 8.6.1.1})$$

$$f'_c \leq 55 \text{ MPa} \quad (\text{Nominal and Ductile}) \quad (\text{CSA 21.2.3.1})$$

The limit of f_{yh} is taken to be 500 MPa for all frames:

$$f_{yh} \leq 500 \text{ MPa}. \quad (\text{CSA 8.5.1})$$

The maximum of all 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 upon the above considerations. Any other minimum stirrup requirements to satisfy spacing and/or volumetric requirements must be investigated independently of the program by the user.

Type of Check/ Design	Ordinary Moment Resisting Frames (non-Seismic)	Nominal Moment Resisting Frames (Seismic)	Ductile Moment Resisting Frames (Seismic)
Strength of materials	$f'_c \leq 80$ MPa $f_y \leq 500$ MPa	$f'_c \leq 55$ MPa $f_y \leq 500$ MPa	$f'_c \leq 55$ MPa $f_y \leq 500$ 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 Minimum hoop	Modified NLD ^a Combinations or Nominal Capacity Shear (V_p) $\phi = 1.0$ and $\alpha = 1.0$ Minimum hoop reinforcement	NLD ^a Combinations and Probable Capacity Shear (V_p) $\phi = 1.0$ and $\alpha = 1.25$ Strength based on $\epsilon_x = 0.002$ 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 $\rho \leq 0.025$ $\rho \geq 0.2\sqrt{f'_c}/f_y, \rho \geq 1.4/f_y$
Beam Min. Moment Override Check	No Requirement	$M_{uEND}^+ \geq \frac{1}{3} M_{uEND}^-$ $M_{uSPAN}^+ \geq \frac{1}{5} \max\{M_u^+, M_u^-\}_{END}$ $M_{uSPAN}^- \geq \frac{1}{5} \max\{M_u^+, M_u^-\}_{END}$	$M_{uEND}^+ \geq \frac{1}{2} M_{uEND}^-$ $M_{uSPAN}^+ \geq \frac{1}{4} \max\{M_u^+, M_u^-\}_{END}$ $M_{uSPAN}^- \geq \frac{1}{4} \max\{M_u^+, M_u^-\}_{END}$
Beam Design Shear	NLD ^a Combinations	Modified NLD ^a Combinations or Nominal Capacity Shear (V_p) with $\alpha = 1.0$ and $\phi = 1.0$ plus V_{D+L}	NLD ^a Combinations and Probable Capacity Shear (V_p) with $\alpha = 1.25$ and $\phi = 1.0$ plus V_{D+L} Strength based on $\beta = 0$, $\theta = 45^\circ$

^a NLD = Number of specified loading

Table V-2
Comparison of Ordinary, Ductile, and Nominal Moment Resisting Frame Design

Chapter VI

Design for BS 8110-85 R1989

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the British limit state design code **BS 8110 R1989** (BSI 1989). Various notations used in this chapter are listed in Table VI-1.

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

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Newton-Millimeter-Second** units unless otherwise noted.

Design Load Combinations

The design loading combinations define the various factored combinations of the load cases for which the structure is to be checked. The design loading combinations are obtained by multiplying the characteristic loads by appropriate partial factors of safety, γ_f (BS 2.4.1.3). If a structure is subjected to dead load (DL) and live load (LL) only, the design will need only one loading combination, namely 1.4 DL + 1.6 LL. However, in addition to the dead load and live load, if the structure is sub-

A_{cv}	Area of section for shear resistance, mm ²
A_s	Area of tension reinforcement, mm ²
A'_s	Area of compression reinforcement, mm ²
A_{sc}	Total area of column longitudinal reinforcement, mm ²
A_{sv}	Total cross-sectional area of links at the neutral axis, mm ²
a	Depth of compression block, mm
b	Width or effective width of the section in the compression zone, mm
b'	Shorter section dimension, mm
	Shorter effective depth of biaxially bent column, mm
b_f	Width or effective width of flange, mm
b_w	Average web width of a flanged beam, mm
C	Compression force, N
d	Effective depth of tension reinforcement, mm
d'	Depth to center of compression reinforcement, mm
E_c	Modulus of elasticity of concrete, MPa
E_s	Modulus of elasticity of reinforcement, assumed as 200000 MPa (BS 2.4.2.3)
e_{min}	Minimum or nominal eccentricity, mm
f_{cu}	Characteristic cube strength at 28 days, MPa
f'_s	Compressive stress in a beam compression steel, MPa
f_y	Characteristic strength of reinforcement, MPa
f_{yv}	Characteristic strength of link reinforcement, MPa (< 460 MPa)
h	Overall depth of a section in the plane of bending, mm
h_f	Flange thickness, mm
K'	Maximum $\frac{M_u}{bd^2 f_{cu}}$ for a singly reinforced concrete section taken as 0.156 by assuming that moment redistribution is limited to 10%
k_1	Shear strength enhancement factor
k_2	Concrete shear strength factor, $[f_{cu}/25]^{1/3}$
l_e	Effective height of a column, mm

Table VI-1
List of Symbols Used in the BS code

l_0	Clear height between end restraints, mm
M	Design moment at a section, MPA
M_1, M_2	Smaller and larger end moments in a slender column, N-mm
M_{add}	Maximum additional moment column, N-mm
M_i	Initial moment at the point of maximum additional moment, N-mm
M_x, M_y	Applied moments about the major and minor axes of a column, N-mm
N	Ultimate axial load, N
s_v	Spacing of links, mm
T	Tension force, N
V	Shear force at ultimate design load, N
v	Shear stress, MPA
v_c	Design ultimate shear stress resistance of a concrete beam, MPA
v_c'	Design concrete shear stress corrected for axial forces, MPA
v_x, v_y	Design ultimate shear stress of a concrete section, MPA
x	Neutral axis depth, mm
x_{bal}	Depth of neutral axis in a balanced section, mm
z	Lever arm, mm
β	Effective length factor
β_b	Moment redistribution factor in a member
γ_f	Partial safety factor for load
γ_m	Partial safety factor for material strength
ϵ_c	Concrete strain
ϵ_s	Strain in tension steel
ϵ_s'	Strain in compression steel

Table VI-1*List of Symbols Used in the BS code (continued)*

jected to wind (WL) and/or earthquake (EL) loads, and considering that those loads are subject to reversals, the following load combinations for ultimate limit state might have to be considered (BS 2.4.3):

$$\begin{aligned} &1.4 \text{ DL} \\ &1.4 \text{ DL} + 1.6 \text{ LL} \end{aligned} \quad (\text{BS 2.4.3})$$

$$\begin{aligned} &1.0 \text{ DL} \pm 1.4 \text{ WL} \\ &1.4 \text{ DL} \pm 1.4 \text{ WL} \\ &1.2 \text{ DL} + 1.2 \text{ LL} \pm 1.2 \text{ WL} \end{aligned} \quad (\text{BS 2.4.3})$$

$$\begin{aligned} &1.0 \text{ DL} \pm 1.4 \text{ EL} \\ &1.4 \text{ DL} \pm 1.4 \text{ EL} \\ &1.2 \text{ DL} + 1.2 \text{ LL} \pm 1.2 \text{ EL} \end{aligned}$$

These are the default load combinations. In addition to the above load combinations, the code requires that all buildings should be capable of resisting a notional design ultimate horizontal load applied at each floor or roof level. The notional load should be equal to 0.015 times the dead load (BS 3.1.4.2). It is recommended that the user define additional load cases for considering the notional load in SAP2000.

Live load reduction factors, as allowed by some design codes, can be applied to the member forces of the live load case on a member-by-member basis to reduce the contribution of the live load to the factored loading.

Design Strength

The design strength for concrete and steel are obtained by dividing the characteristic strength of the material by a partial factor of safety, γ_m . The values of γ_m used in the program are listed below (BS 2.4.4.1).

$$\gamma_m = \begin{cases} 1.15, & \text{for reinforcement,} \\ 1.50, & \text{for concrete in flexure and axial load, and} \\ 1.25, & \text{for shear strength without shear reinforcement.} \end{cases} \quad (\text{BS 2.4.4.1})$$

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 II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement from 0.4 to 6 per cent (BS 3.12.5.3 and BS 3.12.6.2).
- 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 the above-mentioned steps.

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 (BS 3.8.4.1). A typical interaction diagram is shown in Figure II-1.

The coordinates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column (BS 3.4.4.1). See Figure II-2. The linear strain diagram limits the maximum concrete strain, ϵ_c , at the extremity of the section, to 0.0035 (BS 3.4.4.1).

The formulation is based consistently upon the basic principles of ultimate strength design and allows for any doubly symmetric rectangular, square, or circular column section (BS 3.8.4).

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\epsilon_s E_s$ and is limited to the design strength the steel, $f_y/1.15$ ($0.87f_y$). The area associated with each reinforcing bar is placed at the actual location of the center of the bar and the algorithm does not assume any 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). See Figure VI-1.

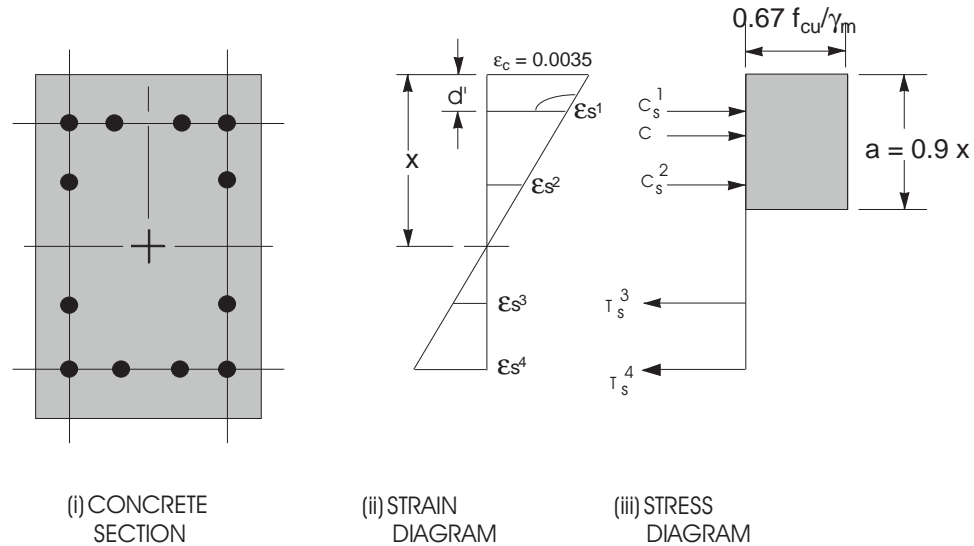


Figure VI-1
Idealized Stress and Strain Distribution in a Column Section

The concrete compression stress block is assumed to be rectangular, with a stress value of $0.67 f_{cu} / \gamma_m = 0.45 f_{cu}$ (BS 3.4.4.1). See Figure VI-1. The interaction algorithm provides corrections to account for the concrete area that is displaced by the reinforcement in the compression zone.

Check Column Capacity

The column capacity is checked for each loading combination at each output station of each column. In checking a particular column for a particular loading combination at a particular location, the following steps are involved:

- Determine the factored moments and forces from the analysis load cases and the specified load combination factors to give $N, V_x, V_y, M_x,$ and M_y .
- Determine the additional moments due to slender column effect. Compute moments due to minimum eccentricity.
- Determine total design moments by adding the corresponding additional moments to the factored moments obtained from the analysis. Determine whether the point, defined by the resulting axial load and biaxial moment set, lies within the interaction volume.

The following three subsections describe in detail the algorithms associated with the above-mentioned steps.

Determine Factored Moments and Forces

Each load combination is defined with a set of load factors corresponding to the load cases. The factored loads for a particular load combination are obtained by applying the corresponding load factors to the load cases, giving N, V_x, V_y, M_x and M_y .

Determine Additional Moments

The determination of additional moments depends on whether the frame is “braced” or “unbraced” against side-sway (BS 3.8.1.5). For “unbraced” columns additional moment is automatically considered in the P- Δ analysis. But for “braced” columns, further calculation is required for stability of individual column members.

Braced Column

The additional moment in a braced column in a particular plane is the product of the axial load and the lateral deflection of the column in that plane (BS 3.8.3),

$$M_{add} = N a_u, \quad (\text{BS 3.8.3.1})$$

where, a_u is the deflection at the ultimate limit state which is obtained from

$$a_u = \beta_e K h \quad \text{and} \quad (\text{BS 3.8.3.1})$$

$$\beta_e = \frac{1}{2000} \left(\frac{l_e}{b'} \right)^2. \quad (\text{BS 3.8.3.1})$$

In the above equations,

- l_e is the effective length in the plane under consideration. It is obtained from

$$l_e = \beta l_0, \quad (\text{BS 3.8.1.6.1})$$

where β is the effective length factor, and l_0 the unsupported length corresponding to instability in the major or minor direction of the element, l_x or l_y in Figure II-4. In calculating the value of the effective length, the β factor is conservatively taken as 1. However, SAP2000 allows the user to override this default value.

- b' is the dimension of the column in the plane of bending considered,

- h is also the dimension of the column in the plane of bending considered, and
- K is the correction factor to the deflection to take care of the influence of the axial force and K is conservatively taken as 1.

SAP2000 then calculates the total design moments by combining the factored moments obtained from analysis and the additional moments. If M_1 and M_2 ($M_2 > M_1$) are the initial end moments in a column member in a particular plane, then the maximum design moment for the column is taken as the greatest of the following:

$$M_2 \quad (\text{BS 3.8.3.2})$$

$$M_1 + M_{add} \quad (\text{BS 3.8.3.2})$$

$$M_i + M_{add}/2 \quad (\text{BS 3.8.3.2})$$

$$N e_{min} \quad (\text{BS 3.8.3.2})$$

where,

M_i is the initial moment in a column due to design ultimate loads at the point of maximum additional moment and is given by

$$M_i = 0.4 M_1 + 0.6 M_2 \geq 0.4 M_2 . \quad (\text{BS 3.8.3.2})$$

M_1 and M_2 are the smaller and the larger end moments respectively. Both moments are assumed to be positive if the column is in single curvature. If the column is in double curvature, M_1 is assumed to be negative.

e_{min} is the minimum eccentricity which is taken as 0.05 times the overall dimension of the column in the plane of bending considered but not more than 20 mm (BS 3.8.2.4).

$$e_{min} = \frac{h}{20} \leq 20 \text{ mm} \quad (\text{BS 3.8.2.4})$$

Unbraced Column

In the case of the unbraced column, it is assumed that the SAP2000 analysis includes P- Δ effects so that the analysis results include the effects of the additional moments. Therefore, no additional computation is required. That means moment magnification factors for moments causing sidesway are taken as unity. However, it is recommended that for P- Δ analysis a factor be used to obtain a P equivalent to 1.2 DL + 1.2 LL (White and Hajjar 1991).

Also, the minimum eccentricity requirements are satisfied so the design moment should at least be

$$M_u \geq N e_{min}, \quad (\text{BS 3.8.3.2})$$

where, e_{min} is the minimum eccentricity which is described in the previous section. In biaxial bending the algorithm ensures that the eccentricity exceeds the minimum about both the axes simultaneously.

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 design forces N , M_x , and M_y are obtained according to the previous subsections. The point (N, M_x, M_y) is then placed in the interaction space shown as point L in Figure II-3. 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.

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 II-3. 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 N , M_x , and M_y set and associated load combination number.

If the reinforcing area is not defined, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

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 reinforcement for a particular column for a particular loading combination due to shear forces in a particular direction, the following steps are involved (BS 3.8.4.6):

- Calculate the design shear stress from

$$v = \frac{V}{A_{cv}}, A_{cv} = bd, \text{ where} \quad (\text{BS 3.4.5.2})$$

$$v \leq 0.8\sqrt{f_{cu}}, \text{ and} \quad (\text{BS 3.4.5.12})$$

$$v \leq 5 \text{ N/mm}^2. \quad (\text{BS 3.4.5.12})$$

If v exceeds either $0.8\sqrt{f_{cu}}$ or 5 N/mm^2 , the section area should be increased.

- Calculate the design concrete shear stress from (BS 3.8.4.6)

$$v'_c = v_c + 0.75 \frac{N}{A_c} \frac{Vd}{M}, \text{ with} \quad (\text{BS 3.4.5.12})$$

$$v_c = \frac{0.79 k_1 k_2}{\gamma_m} \left(\frac{100 A_s}{bd} \right)^{1/3} \left(\frac{400}{d} \right)^{1/4}, \quad (\text{BS 3.4.5.4})$$

where,

k_1 is the enhancement factor for support compression and taken conservatively as 1, (BS 3.4.5.8)

$$k_2 = \left(\frac{f_{cu}}{25} \right)^{1/3}, \quad (\text{BS 3.4.5.4})$$

$$\gamma_m = 1.25, \quad (\text{BS 2.4.4.1})$$

$$0.15 \leq \frac{100 A_s}{bd} \leq 3, \quad (\text{BS 3.4.5.4})$$

$$\frac{400}{d} \geq 1, \quad (\text{BS 3.4.5.4})$$

$$\frac{Vd}{M} \leq 1, \quad (\text{BS 3.4.5.12})$$

$$f_{cu} \leq 40 \text{ N/mm}^2, \text{ and} \quad (\text{BS 3.4.5.4})$$

A_s is the area of tensile steel.

- If $v \leq v'_c + 0.4$, provide minimum links defined by

$$\frac{A_{sv}}{s_v} \geq \frac{0.4 b}{0.87 f_{yv}}, \quad (\text{BS 3.4.5.3})$$

else if $v > v'_c + 0.4$, provide links given by

$$\frac{A_{sv}}{s_v} \geq \frac{(v - v'_c) b}{0.87 f_{yv}}, \quad (\text{BS 3.4.5.3})$$

where f_{yv} can not be greater than 460 MPA (BS 3.4.5.1).

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, and shears, load combination factors, and other criteria described below. The reinforcement requirements are calculated at a user defined number of check stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

Design Beam Flexural Reinforcement

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

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement of 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 and maximum negative factored moments obtained from all of the load combinations at that section.

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 section, or T-Beam effects may be included.

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 VI-2 (BS 3.4.4.1). Furthermore it is assumed that moment redistribution in the member does not exceed 10% (i.e. $\beta_b \geq 0.9$) (BS 3.4.4.4). The code also places a limitation on the neutral axis depth, $x/d \leq 0.5$, to safeguard against non-ductile failures (BS 3.4.4.4). In addition, the area of compression reinforcement is calculated on the assumption that the neutral axis depth remains at the maximum permitted value.

The design procedure used by SAP2000, for both rectangular and flanged sections (L- and T-beams) is summarized below. It is assumed that the design ultimate axial force does not exceed $0.1 f_{cu} A_g$ (BS 3.4.4.1), hence all the beams are designed for major direction flexure and shear only.

Design of a Rectangular beam

For rectangular beams, the moment capacity as a singly reinforced beam, M_{single} , is obtained first for a section. The reinforcing steel area is determined based on whether M is greater than, less than, or equal to M_{single} . See Figure VI-2.

- Calculate the ultimate moment of resistance of the section as singly reinforced.

$$M_{single} = K' f_{cu} b d^2, \text{ where} \quad (\text{BS 3.4.4.4})$$

$$K' = 0.156.$$

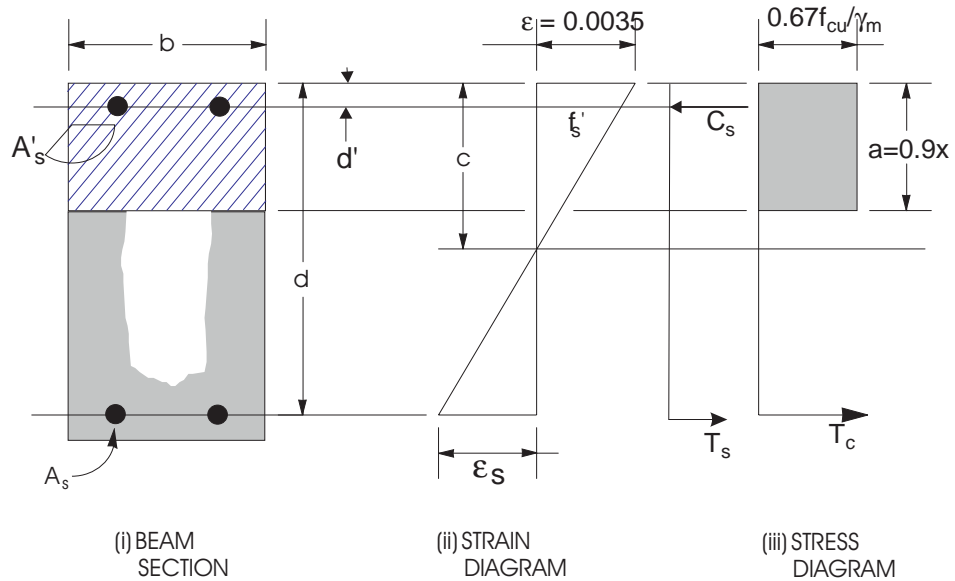


Figure VI-2
Design of Rectangular Beam Section

- If $M \leq M_{single}$ the area of tension reinforcement, A_s , is obtained from

$$A_s = \frac{M}{(0.87 f_y) z}, \text{ where} \quad (\text{BS 3.4.4.4})$$

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\} \leq 0.95d, \text{ and}$$

$$K = \frac{M}{f_{cu} b d^2}.$$

This is the top steel if the section is under negative moment and the bottom steel if the section is under positive moment.

- If $M > M_{single}$, the area of compression reinforcement, A'_s , is given by

$$A'_s = \frac{M - M_{single}}{f'_s (d - d')},$$

where d' is the depth of the compression steel from the concrete compression face, and

$$f'_s = 0.87 f_y \quad \text{if} \quad \frac{d'}{d} \leq \frac{1}{2} \left[1 - \frac{f_y}{800} \right],$$

$$f'_s = 700 \left[1 - \frac{2d'}{d} \right] \quad \text{if} \quad \frac{d'}{d} > \frac{1}{2} \left[1 - \frac{f_y}{800} \right].$$

This is the bottom steel if the section is under negative moment. From equilibrium, the area of tension reinforcement is calculated as

$$A_s = \frac{M_{single}}{(0.87 f_y) z} + \frac{M - M_{single}}{f_y (d - d')}, \quad \text{where} \quad (\text{BS 3.4.4.4})$$

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K'}{0.9}} \right\} = 0.776887 d.$$

Design as a T-Beam

(i) Flanged beam under negative moment

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_w . See Figure VI-3.

(ii) Flanged beam under positive moment

With the flange in compression, the program analyzes the section by considering alternative locations of the neutral axis. Initially the neutral axis is assumed to be located in the flange. Based on this assumption, the program calculates the exact depth of the neutral axis. If the stress block does not extend beyond the flange thickness the section is designed as a rectangular beam of width b_f . If the stress block extends beyond the flange width, then the contribution of the web to the flexural strength of the beam is taken into account. See Figure VI-3.

Assuming the neutral axis to lie in the flange, the normalized moment is computed as

$$K = \frac{M}{f_{cu} b_f d^2}.$$

Then the moment arm is computed as

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\} \leq 0.95d,$$

the depth of neutral axis is computed as

$$x = \frac{1}{0.45} (d - z), \text{ and}$$

the depth of compression block is given by

$$a = 0.9x.$$

- If $a \leq h_f$, the subsequent calculations for A_s are exactly the same as previously defined for the rectangular section design. However, in this case the width of the compression flange, b_f , is taken as the width of the beam, b , for analysis. Whether compression reinforcement is required depends on whether $K > K'$.
- If $a > h_f$, calculation for A_s is done 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 Figure VI-3.

In this case, the ultimate resistance moment of the flange is given by

$$M_f = 0.45 f_{cu} (b_f - b_w) h_f (d - 0.5 h_f),$$

the balance of moment taken by the web is computed as

$$M_w = M - M_f, \text{ and}$$

the normalized moment resisted by the web is given by

$$K_w = \frac{M_w}{f_{cu} b_w d^2}.$$

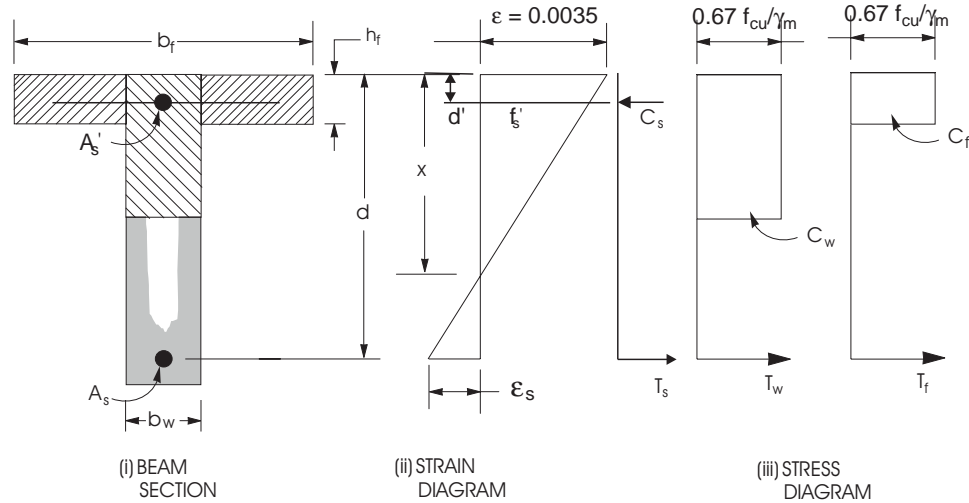


Figure VI-3
Design of a T-Beam Section

- If $K_w \leq 0.156$, the beam is designed as a singly reinforced concrete beam. The area of steel is calculated as the sum of two parts, one to balance compression in the flange and one to balance compression in the web.

$$A_s = \frac{M_f}{0.87 f_y (d - 0.5 h_f)} + \frac{M_w}{0.87 f_y z}, \text{ where}$$

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K_w}{0.9}} \right\} \leq 0.95d .$$

- If $K_w > K'$, compression reinforcement is required and is calculated as follows:

The ultimate moment of resistance of the web only is given by

$$M_{uw} = K' f_{cu} b_w d^2 .$$

The compression reinforcement is required to resist a moment of magnitude $M_w - M_{uw}$. The compression reinforcement is computed as

$$A'_s = \frac{M_w - M_{uw}}{f'_s (d - d')},$$

where, d' is the depth of the compression steel from the concrete compression face, and

$$f'_s = 0.87f_y, \quad \text{if } \frac{d'}{d} \leq \frac{1}{2} \left[1 - \frac{f_y}{800} \right], \text{ and}$$

$$f'_s = 700 \left[1 - \frac{2d'}{d} \right], \quad \text{if } \frac{d'}{d} > \frac{1}{2} \left[1 - \frac{f_y}{800} \right].$$

The area of tension reinforcement is obtained from equilibrium

$$A_s = \frac{1}{0.87f_y} \left[\frac{M_f}{d - 0.5h_f} + \frac{M_{uw}}{0.777d} + \frac{M_w - M_{uw}}{d - d'} \right].$$

Design Beam Shear Reinforcement

The shear reinforcement is designed for each loading combination in the major and minor directions of the column. In designing the shear reinforcement for a particular beam for a particular loading combination due to shear forces in a particular direction, the following steps are involved (BS 3.4.5):

- Calculate the design shear stress as

$$v = \frac{V}{A_{cv}}, \quad A_{cv} = bd, \quad \text{where} \quad (\text{BS 3.4.5.2})$$

$$v \leq 0.8\sqrt{f_{cu}}, \quad \text{and} \quad (\text{BS 3.4.5.2})$$

$$v \leq 5 \text{ N/mm}^2. \quad (\text{BS 3.4.5.2})$$

- Calculate the design concrete shear stress from

$$v_c = \frac{0.79 k_1 k_2}{\gamma_m} \left(\frac{100 A_s}{bd} \right)^{1/3} \left(\frac{400}{d} \right)^{1/4}, \quad (\text{BS 3.4.5.4})$$

where,

k_1 is the enhancement factor for support compression, and is conservatively taken as 1, (BS 3.4.5.8)

$$k_2 = \left(\frac{f_{cu}}{25} \right)^{1/3} \geq 1, \text{ and} \quad (\text{BS 3.4.5.4})$$

$$\gamma_m = 1.25. \quad (\text{BS 2.4.4.1})$$

However, the following limitations also apply:

$$0.15 \leq \frac{100 A_s}{bd} \leq 3, \quad (\text{BS 3.4.5.4})$$

$$\frac{400}{d} \geq 1, \text{ and} \quad (\text{BS 3.4.5.4})$$

$$f_{cu} \leq 40 \text{ N/mm}^2 \text{ (for calculation purpose only)}. \quad (\text{BS 3.4.5.4})$$

A_s is the area of tensile steel.

- If $v \leq v_c + 0.4$, provide minimum links defined by

$$\frac{A_{sv}}{s_v} \geq \frac{0.4 b}{0.87 f_{yv}}, \quad (\text{BS 3.4.5.3})$$

else if $v > v_c + 0.4$, provide links given by

$$\frac{A_{sv}}{s_v} \geq \frac{(v - v_c) b}{0.87 f_{yv}}, \quad (\text{BS 3.4.5.3})$$

where f_{yv} can not be greater than 460 MPA (BS 3.4.5.1).

Chapter VII

Design for Eurocode 2

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the **1992 Eurocode 2** (CEN 1992). Various notations used in this chapter are listed in Table VII-1.

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

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Newton-Millimeter-Second** units unless otherwise noted.

Design Load Combinations

The design loading combinations define the various factored combinations of the load cases for which the structure is to be checked. The design loading combinations are obtained by multiplying the characteristic loads by appropriate partial factors of safety. If a structure is subjected to dead load (DL) and live load (LL) only, the design will need only one loading combination, namely $1.35 \text{ DL} + 1.5 \text{ LL}$.

A_c	Gross cross-sectional area of a frame member, mm ²
A_{cv}	Area of section for shear resistance, mm ²
A_s	Area of tension reinforcement in a beam, mm ² Area of total longitudinal reinforcement in a column, mm ²
A'_s	Area of compression reinforcement, mm ²
A_{sw}	Total cross-sectional area of links at the neutral axis, mm ²
a	Depth of compression block, mm
b	Width or effective width of the section in the compression zone, mm
b_f	Width or effective width of flange, mm
b_w	Average web width of a flanged beam, mm
d	Effective depth of tension reinforcement, mm
d'	Effective depth of compression reinforcement, mm
E_c	Modulus of elasticity of concrete, MPA
E_s	Modulus of elasticity of reinforcement, assumed as 200000 MPa (EC2 4.2.2.3.2)
e	Eccentricity of axial load in a column, mm
e_{min}	Minimum or nominal eccentricity, mm
e_{tot}	Total eccentricity for a braced column, mm
f_{cd}	Design concrete compressive strength = f_{ck} / γ_c , MPA
f_{ck}, f'_c	Characteristic compressive cylinder strength of concrete at 28 days, MPa
f_{cwd}	Design concrete compressive strength for shear design = f_{cwk} / γ_c , MPA
f_{cwk}, f'_{cs}	Characteristic compressive cylinder strength for shear design, MPA
f_{yd}	Design tensile yield strength of reinforcing steel = f_{yk} / γ_s , MPA
f_{yk}, f_y	Characteristic tensile yield strength of reinforcement, MPA
f'_s	Stress in a beam compression steel, MPA
f_{ywd}	Design tensile strength of shear reinforcing steel = f_{ywk} / γ_s , MPA
f_{ywk}, f_{ys}	Characteristic tensile strength of shear reinforcement, MPA
h	Overall depth of a section in the plane of bending, mm
h_f	Flange thickness, mm
l_0	Effective height of a column, mm
l_{col}	Clear height between end restraints, mm

Table VII-1

List of Symbols Used in the Eurocode 2

M	Design moment at a section, N-mm
M_1, M_2	Smaller and larger end moments in a slender column, N-mm
M_x, M_y	Applied moments about the major and minor axes of a column, N-mm
M_{Rd}	Design moment of resistance of a section N-mm
M_{Sd}	Moment at a section obtained from analysis, N-mm
m	Normalized design moment, $M/bd^2\alpha f_{cd}$
N	Ultimate axial load, N
s_v	Spacing of links, mm
V_{Rd1}	Design shear resistance from concrete alone, N
V_{Rd2}	Design limiting shear resistance of a cross-section, N
V_{Sd}	Shear force at ultimate design load, N
V_x, V_y	Shear force at ultimate design load in two directions, N
V_{wd}	Shear force from reinforcement, N
α	Concrete strength reduction factor for sustained loading
β	Effective length factor, Enhancement factor of shear resistance for concentrated load
γ_f	Partial safety factor for load
γ_c	Partial safety factor for concrete strength, 1.5 by default
γ_m	Partial safety factor for material strength
γ_s	Partial safety factor for steel strength, 1.15 by default
δ	Redistribution factor
ϵ_c	Concrete strain
ϵ_s	Strain in tension steel
ϵ_s'	Strain in compression steel
ν	Effectiveness factor for shear resistance without concrete crushing, Out of plumbness factor
ρ	Tension reinforcement ratio, A_s/bd
σ_{cp}	Effective average compressive stress in concrete column, MPA
ω	Normalized tensile steel ratio, $A_s f_{yd} / \alpha f_{cd} bd$
ω'	Normalized compression steel ratio, $A_s' f_{yd} / \alpha f_{cd} bd$
ω_{lim}	Normalized limiting tensile steel ratio

Table VII-1*List of Symbols Used in the Eurocode 2 (continued)*

However, in addition to the dead load and live load, if the structure is subjected to wind (WL) and earthquake (EL) forces, and considering that wind and earthquake forces are subject to reversals, the following load combinations might have to be considered (EC2 2.3.3):

$$\begin{aligned} &1.35 \text{ DL} \\ &1.35 \text{ DL} + 1.50 \text{ LL} \end{aligned} \quad (\text{EC2 2.3.3})$$

$$\begin{aligned} &1.35 \text{ DL} \pm 1.50 \text{ WL} \\ &1.00 \text{ DL} \pm 1.50 \text{ WL} \\ &1.35 \text{ DL} + 1.35 \text{ LL} \pm 1.35 \text{ WL} \end{aligned} \quad (\text{EC2 2.3.3})$$

$$\begin{aligned} &1.00 \text{ DL} \pm 1.00 \text{ EL} \\ &1.00 \text{ DL} + 1.5*0.3 \text{ LL} \pm 1.0 \text{ EL} \end{aligned} \quad (\text{EC2 2.3.3})$$

These are the default load combinations. These default loading combinations are produced for persistent and transient design situations (EC2 2.2.1.2) by combining load due to dead, live, wind, and earthquake loads according to the simplified formula (EC2 2.3.3.1) for ultimate limit states.

In addition to the above load combinations, the code requires that all buildings should be capable of resisting a notional design ultimate horizontal load applied at each floor or roof level (EC2 2.5.1.3). It is recommended that the user define additional load cases for considering the notional load in SAP2000.

Live load reduction factors, as allowed by some design codes, can be applied to the member forces of the live load condition on a member-by-member basis to reduce the contribution of the live load to the factored loading.

Design Strength

The design strength for concrete and steel are obtained by dividing the characteristic strength of the materials by a corresponding partial factor of safety as shown below.

$$f_{cd} = f_{ck} / \gamma_c, \quad (\text{EC2 2.3.3.2})$$

$$f_{cwd} = f_{cwk} / \gamma_c, \quad (\text{EC2 2.3.3.2})$$

$$f_{yd} = f_{yk} / \gamma_s, \quad (\text{EC2 2.3.3.2})$$

$$f_{ywd} = f_{ywk} / \gamma_s, \text{ where} \quad (\text{EC2 2.3.3.2})$$

$$\gamma_s = \text{Partial safety factor for steel} = 1.15, \text{ and} \quad (\text{EC2 2.3.3.2})$$

$$\gamma_c = \text{Partial safety factor for concrete} = 1.5. \quad (\text{EC2 2.3.3.2})$$

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 sections types of the model (EC2 4.3.1.2). A typical biaxial interaction surface is shown in Figure II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement. The minimum reinforcement is taken to be

$$A_{s,min} = \max \begin{cases} 0.15 \frac{N_{sd}}{f_{yd}} \\ 0.003A_c \end{cases} \quad (\text{EC2 5.4.1.2.1})$$

and the maximum reinforcement is taken to be 8 percent (EC2 5.4.1.2.1).

- 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 reinforcing.

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

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 as shown in Figure II-1. The coordi-

nates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column. See Figure II-2.

The formulation is based consistently upon the basic principles of ultimate strength design and allows for any doubly symmetric rectangular, square, or circular column section. The linear strain diagram limits the maximum concrete strain, ϵ_c , at the extremity of the section to 0.0035 and at a depth of $\frac{3}{7}d$ from the most compressed face to 0.0020 (EC2 4.3.1.2). See Figure VII-1.

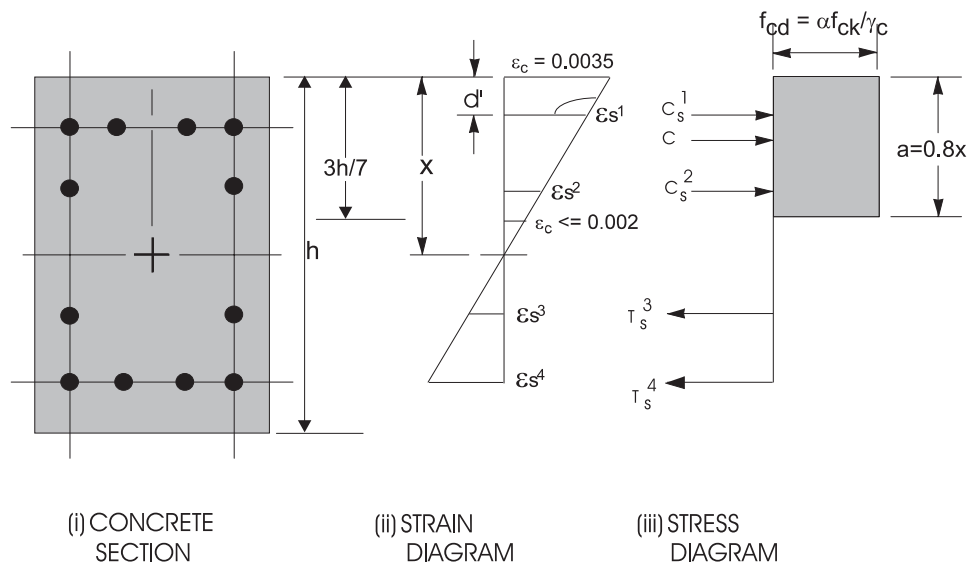


Figure VII-1
Idealized Stress and Strain Distribution in a Column Section

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\epsilon_s E_s$, and is limited to the design yield strength the steel, f_{yd} (EC2 4.2.3.3.3). The area associated with each reinforcing bar is placed at the actual location of the center of the bar and the algorithm does not assume any 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).

The concrete compression stress block is assumed to be rectangular, with a stress value of αf_{cd} , where f_{cd} is the design value of concrete cylinder compressive strength and α is the reduction factor to account for sustained compression. α is generally assumed to be 0.80 (EC2 4.2.1.3). See Figure VII-1. The interaction algo-

rithm provides corrections to account for the concrete area that is displaced by the reinforcement in the compression zone.

Check Column Capacity

The column capacity is checked for each loading combination of each column. In checking a particular column for a particular loading combination at a particular location, the following steps are involved:

- Determine the factored moments and forces from the analysis load cases and the specified load combination factors to give N , V_x , V_y , M_x , and M_y .
- Determine the code total moments due to slender column effect. Compute moments due to minimum eccentricity.
- Check the column capacity ratio or compute the reinforcement for the column for resisting the factored moments, the code total moments, and the moments from minimum eccentricity.

The following three subsections describe in detail the algorithms associated with the above-mentioned steps.

Determine Factored Moments and Forces

Each load combination is defined with a set of load factors corresponding to the load cases. The factored loads for a particular load combination are obtained by applying the corresponding load factors to the load cases, giving N , V_x , V_y , M_x , and M_y .

Determine Code Total Moments

The determination of code total moments depends on whether the frame is “braced” or “unbraced” against side-sway.

Braced Column

Eurocode specifies that for braced columns the total moment should be computed from a set of eccentricities, such that

$$e_{tot} = e_0 + e_a + e_2, \text{ where} \quad (\text{EC2 4.3.5.6.2})$$

$$e_0 = 0.4 \frac{M_1}{N} + 0.6 \frac{M_2}{N} \geq 0.4 \frac{M_2}{N}, \text{ where } |M_1| \leq |M_2|, \quad (\text{EC2 4.3.5.6.2})$$

$$e_a = \frac{\nu l_0}{2}, \quad (\text{EC2 4.3.5.4})$$

ν is taken as 1/100, however the user can override this value (EC2 2.5.1.3),

l_0 is the effective length of a column in a given plane and is obtained from

$$l_0 = \beta l_{col}, \quad (\text{EC2 4.3.5.3.5})$$

where β is the effective length factor depending on the end conditions and resistance against side-sway, β is conservatively taken as 1 for braced frames, and l_{col} is the unsupported length corresponding to instability in the major or minor direction of the element, l_x and l_y in Figure II-4.

$$e_2 = \frac{k_1 k_2 l_0^2 f_{yd}}{4.5 E_s d}, \quad (\text{EC2 4.3.5.6.3})$$

$$k_1 = \begin{cases} 0 & \text{for } \lambda \leq 15, \\ \frac{\lambda}{20} - 0.75, & \text{for } 15 < \lambda \leq 35, \\ 1 & \text{for } \lambda > 35, \end{cases}$$

$$\lambda = \beta l_0 / r_G,$$

r_G = the radius of gyration about the axis of bending, and

$$k_2 = \frac{N_{ud} - N_{sd}}{N_{ud} - N_{bal}} \leq 1, \quad k_2 \text{ is taken as 1.0.}$$

However, the minimum eccentricity requirement is satisfied such that

$$M_{Rd} > N_{Sd} e_{min}, \quad \text{where} \quad (\text{EC2 4.3.5.5.3})$$

M_{Rd} = Design moment resistance of the section,

N_{Sd} = The axial force obtained from analysis, and

e_{min} is the minimum eccentricity which is taken as 0.05 times the overall dimension of the column in the plane of bending and is given by

$$e_{min} = h/20. \quad (\text{EC2 4.3.5.5.3})$$

Finally the design moments are computed from the maximum of the three,

$$M_{Rd} = \max(N_{Rd} e_{tot}, N_{Rd} e_{min}, M_{factored}). \quad (\text{EC2 4.3.5.6})$$

In biaxial bending, the program calculates the design moments at any station about two axes.

Unbraced Column

In the case of the unbraced column, it is assumed that the SAP2000 analysis includes P- Δ effects so that the analysis results include the effects of the additional moments. Therefore, any additional computation is not required. That means, the moment magnification factors for moments causing sidesway are taken as unity. However, it is recommended that a factor be used to obtain a axial force P equivalent to $1.35 \text{ DL} + 1.35 \text{ LL}$ for P- Δ analysis (White and Hajjar 1991).

In addition, the minimum eccentricity requirement needs to be satisfied so that the design moment should at least be

$$M_{Rd} \geq N e_{min} , \quad (\text{EC2 4.3.5.5.3})$$

where, e_{min} is the minimum eccentricity which is described in the previous section. In biaxial bending the algorithm ensures that the eccentricity exceeds the minimum about both the axes simultaneously.

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 design forces N , M_x , and M_y are obtained according to the previous subsections. The point (N, M_x, M_y) is then placed in the interaction space shown as point L in Figure II-3. 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.

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 II-3. 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 N , M_x , and M_y set and associated load combination number.

If the reinforcing area is not defined, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

Design Column Shear Reinforcement

The shear reinforcement is designed for each loading combination in the major and minor directions of the column. The assumptions in designing the shear reinforcement are as follows:

- The column sections are assumed to be prismatic. The effect of any variation of width in the column section on the concrete shear capacity is neglected.
- The effect on the concrete shear capacity of any concentrated or distributed load in the span of the column between two beams is ignored. Also, the effect of the direct support on the columns provided by the beams is ignored.
- All shear reinforcement is provided through shear reinforcements which are perpendicular to the longitudinal reinforcement.
- The effect of any torsion is neglected for the design of shear reinforcement.

In designing the shear reinforcement for a particular column for a particular loading combination due to shear forces in a particular direction, the following steps of the standard method are involved (EC2 4.3.2.1):

- Obtain the design value of the applied shear force V_{sd} from the SAP2000 analysis results.

$$V_{sd} = V_2 \quad \text{or} \quad V_3$$

- Calculate the design shear resistance of the member without shear reinforcement.

$$V_{Rd1} = \beta \left[\tau_{Rd} k (1.2 + 40\rho_1) + 0.15\sigma_{cp} \right] A_{cv}, \quad \text{where} \quad (\text{EC2 4.3.2.3(1)})$$

β = enhancement factor for shear resistance for members with concentrated loads located near the face of the support.
 β is taken as 1. (EC2 4.3.2.1(9))

$$\tau_{Rd} = \frac{0.25f_{ctk0.05}}{\gamma_c}, \text{ basic design shear strength of concrete, (EC2 4.3.2.3)}$$

$$f_{ctk0.05} = 0.7f_{ctm}, \text{ (EC2 3.1.2.3)}$$

$$f_{ctm} = 0.3f_{cwk}^{2/3}, \text{ (EC2 3.1.2.3)}$$

k = strength magnification factor for curtailment of longitudinal reinforcement and depth of the section, and is considered to be 1, (EC2 4.3.2.3)

$$\rho_1 = \text{tension reinforcement ratio} = \frac{A_{s1}}{A_{cv}} \leq 0.02, \text{ (EC2 4.3.2.3)}$$

A_{s1} = area of tension reinforcement, taken as half of total longitudinal reinforcement, $A_s/2$, in a column, (EC2 4.3.2.3)

$$\sigma_{cp} = \text{average stress in concrete due to axial force} = \frac{N_{sd}}{A_c}, \text{ (EC2 4.3.2.3)}$$

N_{sd} = the design value of the applied axial force in section, N_{sd} is taken as positive for compression and negative for tension, and (EC2 4.3.2.3)

A_c = the total gross area of concrete cross-section, (EC2 4.3.2.3)

A_{cv} = the total area of concrete cross-section, it is taken as A_{cv} which is shown in Figure VII-2. For a rectangular section A_{cv} equals $b_w d$.

- Calculate the maximum design shear force that can be carried without crushing of the notional concrete compressive struts, $V_{Rd2,red}$.

$$V_{Rd2,red} = 1.67 V_{Rd2} \left(1 - \frac{\sigma_{cp,eff}}{f_{cd}} \right), \text{ where (EC2 4.3.2.2(4))}$$

$$0 \leq V_{Rd2,red} \leq V_{Rd2}, \text{ (EC2 4.3.2.2(4))}$$

$$V_{Rd2} = \frac{1}{2} v f_{cwd} (0.9A_{cv}), \text{ (EC2 4.3.2.3(3))}$$

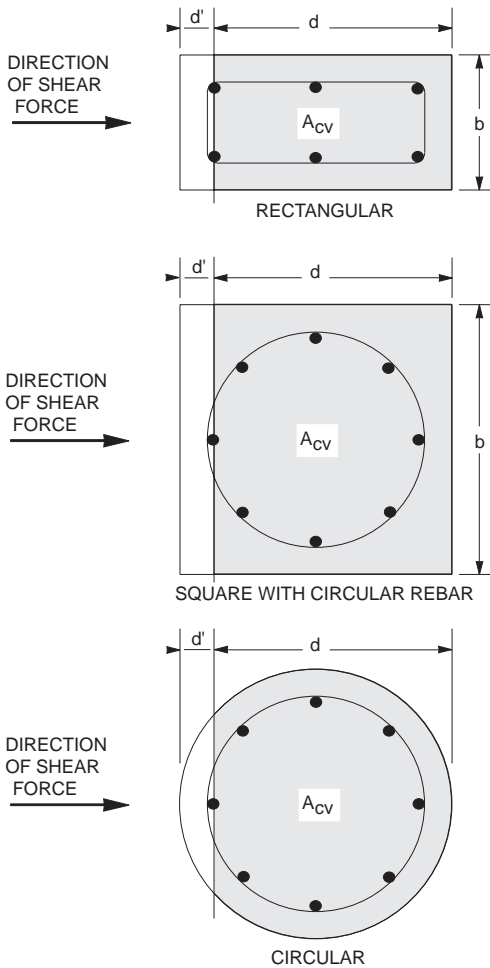


Figure VII-2
Shear Stress Area, A_{cv}

$$v = 0.7 - \frac{f_{ck}}{200} \geq 0.5, \text{ the effectiveness factor,} \quad (\text{EC2 4.3.2.3})$$

$$\sigma_{cp,eff} = \frac{N_{sd} - f_{yd} A_{s2}}{A_c}, \text{ effective average stress in concrete, (EC2 4.3.2.2)}$$

N_{Sd} = the design axial force, it is taken positive for compression and negative for tension,
(EC2 4.3.2.2(4))

f_{yd} = the design yield stress of compression steel,
(EC2 4.3.2.2(4))

$f_{yd} \leq 400$ MPa,
(EC2 4.3.2.2(4))

A_{s2} is the area of reinforcement in the compression zone, A_{s2} is taken as half of total longitudinal reinforcement, $A_s/2$, in a column, and
(EC2 4.3.2.2(4))

A_c is the gross total area of concrete cross-section.

- The shear reinforcement per unit spacing is computed following the Standard method as follows (EC2 4.3.2.4.3):

If $V_{Sd} \leq V_{Rd1}$,

$$\frac{A_{sw}}{s} = 0, \quad (\text{EC2 4.3.2.4.3})$$

else if $V_{Rd1} < V_{Sd} < V_{Rd2,red}$,

$$\frac{A_{sw}}{s} = \frac{(V_{Sd} - V_{Rd1})/A_{cv}}{0.9 f_{ywd}} b_w, \text{ where} \quad (\text{EC2 4.3.2.4.3})$$

f_{ywd} is the design yield strength of the shear reinforcement.

else if $V_{Sd} \geq V_{Rd2,red}$,

a failure condition is declared.
(EC2 4.3.2.2(4))

The maximum of all the calculated A_{sw}/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 shear reinforcement is imposed as follows:

$$\frac{A_{sw}}{s} \geq \rho_w b_w \quad (\text{EC2 5.4.2.2(5)})$$

In the above equation ρ_w is the shear reinforcement ratio. It is taken based on the concrete compression strength for shear use f'_{cs} which is the same as f_{cwk} and the shear rebar yield strength f_{ys} which is the same as f_{ywk} . ρ_w is determined from the following table (EC2 Table 5.5).

Concrete Strength (MPA)	Steel Strength (MPA)		
	$f_{ywk} \leq 220$	$220 < f_{ywk} \leq 400$	$f_{ywk} > 400$
$f_{cwk} \leq 20$	0.0016	0.0009	0.0007
$20 < f_{cwk} \leq 35$	0.0024	0.0013	0.0011
$f_{cwk} > 35$	0.0030	0.0016	0.0013

Table VII.2
Minimum shear reinforcement ratio, ρ_w

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, shears, load combination factors, and other criteria described below. The reinforcement requirements are calculated at a user defined number of check stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

Design Beam Flexural Reinforcement

The beam top and bottom flexural steel is designed at the design stations along the beam span. In designing the flexural reinforcement for a particular beam for a particular section, for the beam major moment, the following steps are involved:

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement, 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_u^+ and maximum negative M_u^- factored moments 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 section, or T-Beam effects may be included.

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 VII-3 (EC2 4.3.1.2). Furthermore, it is assumed that moment redistribution in the member does not exceed the code specified limiting value. The code also places a limitation on the neutral axis depth, to safeguard against non-ductile failures (EC2 2.5.3.4.2). When the applied moment exceeds M_u , the area of compression reinforcement is calculated on the assumption that the neutral axis depth remains at the maximum permitted value.

The design procedure used by SAP2000, for both rectangular and flanged sections (L- and T-beams) is summarized below. It is assumed that the design ultimate axial force does not exceed $0.08 f_{ck} A_g$ (EC2 4.3.1.2), hence all the beams are designed for major direction flexure and shear only.

Design as a Rectangular Beam

For rectangular beams, the normalized moment, m , and the normalized section capacity as a singly reinforced beam, m_{lim} , are obtained first. The reinforcing steel area is determined based on whether m is greater than, less than, or equal to m_{lim} .

- Calculate the normalized design moment, m .

$$m = \frac{M}{bd^2\alpha f_{cd}}, \text{ where}$$

α is the reduction factor to account for sustained compression. α is generally assumed to be 0.80 for assumed rectangular stress block, (EC2 4.2.1.3). See also page 124 for α . The concrete compression stress block is assumed to be rectangular, with a stress value of αf_{cd} , where f_{cd} is the design concrete strength and is equal to $\frac{f_{ck}}{\gamma_c}$. See Figure VII-3.

- Calculate the normalized concrete moment capacity as a singly reinforced beam, m_{lim} .

$$m_{lim} = \left(\frac{x}{d}\right)_{lim} \left[1 - 0.4 \left(\frac{x}{d}\right)_{lim} \right],$$

where the limiting value of the ratio, $\frac{x}{d}$, of the neutral axis depth at the ultimate limit state after redistribution to the effective depth, is expressed as a function of the ratio of the redistributed moment to the moment before redistribution, δ , as follows:

$$\left(\frac{x}{d}\right)_{lim} = \frac{\delta - 0.44}{1.25}, \text{ if } f_{ck} \leq 35, \quad (\text{EC2 2.5.3.4.1})$$

$$\left(\frac{x}{d}\right)_{lim} = \frac{\delta - 0.56}{1.25}, \text{ if } f_{ck} > 35, \quad (\text{EC2 2.5.3.4.1})$$

δ is assumed to be 1.

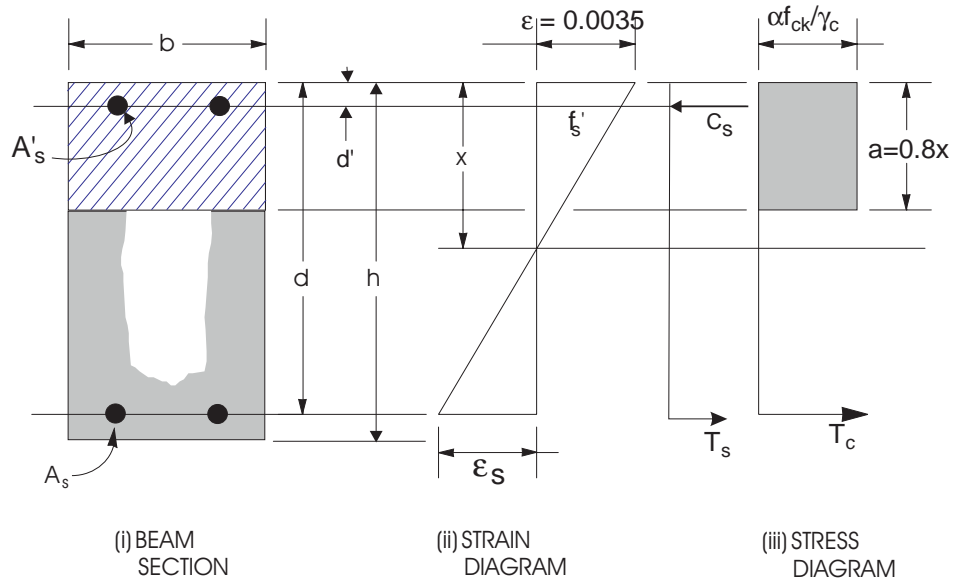


Figure VII-3
Design of a Rectangular Beam

- If $m \leq m_{lim}$, a singly reinforced beam will suffice. Calculate the normalized steel ratio,

$$\omega = 1 - \sqrt{1 - 2m}.$$

Calculate the area of tension reinforcement, A_s , from

$$A_s = \omega \left[\frac{\alpha f_{cd} b d}{f_{yd}} \right].$$

This is the top steel if the section is under negative moment and the bottom steel if the section is under positive moment.

- If $m > m_{lim}$, the beam will not suffice as a singly reinforced beam. Both top and bottom steel are required.
 - Calculate the normalized steel ratios ω' , ω_{lim} , and ω .

$$\omega_{lim} = 0.807 \left(\frac{x}{d} \right)_{lim},$$

$$\omega' = \frac{m - m_{lim}}{1 - d'/d}, \text{ and}$$

$$\omega = \omega_{lim} + \omega'.$$

- Calculate the area of compression and tension reinforcement, A'_s and A_s , as follows:

$$A'_s = \omega' \left[\frac{\alpha f_{cd} b d}{f'_s} \right], \text{ and}$$

$$A_s = \omega \left[\frac{\alpha f_{cd} b d}{f_{yd}} \right] \text{ where}$$

$$f'_s = E_s \varepsilon_{c,max} \left[1 - \frac{2d'}{d} \right] \leq \frac{f_y}{\gamma_s}.$$

Design as a T-Beam

(i) Flanged beam under negative moment

The contribution of the flange to the strength of the beam is ignored if the flange is in the tension side. See Figure VII-4. The design procedure is therefore identical to the one used for rectangular beams. However, the width of the web, b_w , is taken as the width of the beam.

(ii) Flanged beam under positive moment

With the flange in compression, the program analyzes the section by considering alternative locations of the neutral axis. Initially the neutral axis is assumed to be located within the flange. Based on this assumption, the program calculates the depth of the neutral axis. If the stress block does not extend beyond the flange thickness the section is designed as a rectangular beam of width b_f . If the stress block extends beyond the flange, additional calculation is required. See Figure VII-4.

- Calculate the normalized design moment, m .

$$m = \frac{M}{b_f d^2 \alpha f_{cd}}, \text{ where}$$

α is the reduction factor to account for sustained compression. α is generally assumed to be 0.80 for assumed rectangular stress block, (EC2 4.2.1.3). See

also page 124 for α . The concrete compression stress block is assumed to be rectangular, with a stress value of αf_{cd} .

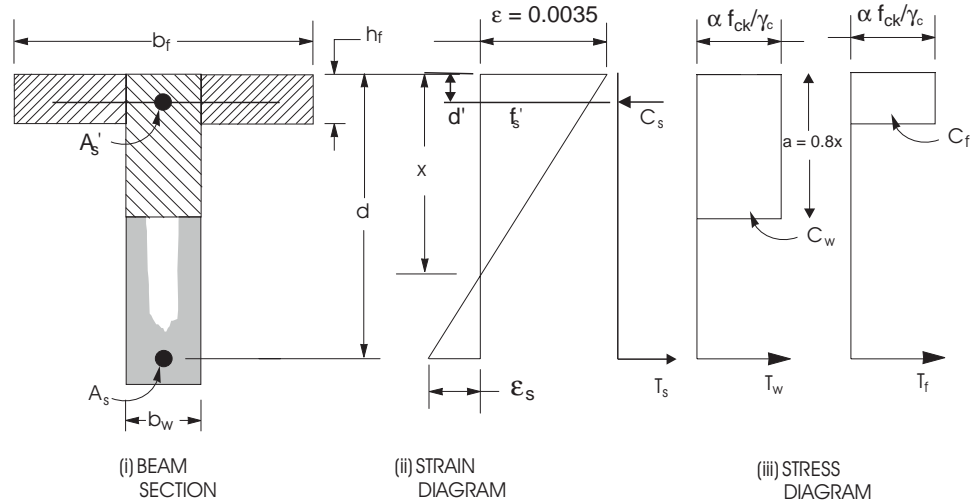


Figure VII-4
Design of a T-Beam Section

- Calculate the limiting value of the ratio, $\left(\frac{x}{d}\right)_{lim}$, of the neutral axis depth at the ultimate limit state after redistribution to the effective depth, which is expressed as a function of the ratio of the redistributed moment to the moment before redistribution, δ , as follows:

$$\left(\frac{x}{d}\right)_{lim} = \frac{\delta - 0.44}{1.25}, \quad \text{if } f_{ck} \leq 35, \quad (\text{EC2 2.5.3.4.1})$$

$$\left(\frac{x}{d}\right)_{lim} = \frac{\delta - 0.56}{1.25}, \quad \text{if } f_{ck} > 35, \quad (\text{EC2 2.5.3.4.1})$$

δ is assumed to be 1.

- Calculate the normalized steel ratio,

$$\omega = 1 - \sqrt{1 - 2m}.$$

- Calculate the ratio, $\frac{x}{d}$, as follows:

$$\frac{x}{d} = \frac{\omega}{0.807}.$$

- If $\left(\frac{x}{d}\right) \leq \left(\frac{h_f}{d}\right)$, the neutral axis lies within the flange. Calculate the area of tension reinforcement, A_s , as follows:

$$A_s = \omega \left[\frac{\alpha f_{cd} b_f d}{f_{yd}} \right].$$

- If $\left(\frac{x}{d}\right) > \left(\frac{h_f}{d}\right)$, the neutral axis lies below the flange.

Calculate steel area required for equilibrating the flange compression, A_{s2} .

$$A_{s2} = \frac{(b_f - b_w) h_f \alpha f_{cd}}{f_{yd}},$$

and the corresponding resistive moment is given by

$$M_2 = A_{s2} f_{yd} \left(d - \frac{h_f}{2} \right).$$

Calculate steel area required for rectangular section of width b_w to resist moment, $M_1 = M - M_2$, as follows:

$$m_1 = \frac{M_1}{b_w d^2 \alpha f_{cd}}, \text{ and}$$

$$m_{lim} = \left(\frac{x}{d} \right)_{lim} \left[1 - 0.4 \left(\frac{x}{d} \right)_{lim} \right],$$

- If $m_1 \leq m_{lim}$,

$$\omega_1 = 1 - \sqrt{1 - 2m_1}, \text{ and}$$

$$A_{s1} = \omega_1 \left[\frac{\alpha f_{cd} b_w d}{f_{yd}} \right].$$

– If $m_1 > m_{lim}$,

$$\omega' = \frac{m_1 - m_{lim}}{1 - d'/d},$$

$$\omega_{lim} = 0.807 \left(\frac{x}{d} \right)_{lim},$$

$$\omega_1 = \omega_{lim} + \omega',$$

$$A'_s = \omega' \left[\frac{\alpha f_{cd} b d}{f'_s} \right], \text{ and}$$

$$A_{s1} = \omega_1 \left[\frac{\alpha f_{cd} b_w d}{f_{yd}} \right] \text{ where}$$

$$f'_s = E_s \varepsilon_{c,max} \left[1 - \frac{2d'}{d} \right] \leq \frac{f_y}{\gamma_s}.$$

– Calculate total steel area required for the tension side.

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

Design Beam Shear Reinforcement

The shear reinforcement is designed for each loading combination at various check stations along the beam span. The assumptions in designing the shear reinforcements are as follows:

- The beam sections are assumed to be prismatic. The effect of any variation of width in the beam section on the concrete shear capacity is neglected.
- The effect on the concrete shear capacity of any concentrated or distributed load in the span of the beam between two columns is ignored. Also, the effect of the direct support on the beams provided by the columns is ignored.
- All shear reinforcements are assumed to be perpendicular to the longitudinal reinforcement.
- The effect of any torsion is neglected for the design of shear reinforcement.

In designing the shear reinforcement for a particular beam for a particular loading combination due to shear forces in a particular direction, the following steps of the standard method are involved (EC2 4.3.2.1).

- Obtain the design value of the applied shear force V_{Sd} from the SAP2000 analysis results.

$$V_{Sd} = V_2$$

- Calculate the design shear resistance of the member without shear reinforcement.

$$V_{Rd1} = \beta \left[\tau_{Rd} k (1.2 + 40\rho_1) + 0.15\sigma_{cp} \right] (b_w d), \text{ where (EC2 4.3.2.3(1))}$$

β = enhancement factor for shear resistance for members with concentrated loads located near the face of the support.

β is taken as 1. (EC2 4.3.2.1(9))

$$\tau_{Rd} = \frac{0.25f_{ctk\ 0.05}}{\gamma_c}, \text{ basic design shear strength of concrete, (EC2 4.3.2.3)}$$

$$f_{ctk\ 0.05} = 0.7f_{ctm}, \text{ (EC2 3.1.2.3)}$$

$$f_{ctm} = 0.3f_{cwk}^{\frac{2}{3}}, \text{ (EC2 3.1.2.3)}$$

k = strength magnification factor for curtailment of longitudinal reinforcement and depth of the section, and is considered to be 1, (EC2 4.3.2.3)

$$\rho_1 = \text{tension reinforcement ratio} = \frac{A_{s1}}{b_w d} \leq 0.02, \text{ (EC2 4.3.2.3)}$$

$$A_{s1} = \text{area of tension reinforcement in the beam, (EC2 4.3.2.3)}$$

$$\sigma_{cp} = \text{average stress in concrete due to axial force} = \frac{N_{Sd}}{A_c}, \text{ (EC2 4.3.2.3)}$$

N_{Sd} = the design value of the applied axial force in section, N_{Sd} is taken as positive for compression and negative for tension, and (EC2 4.3.2.3)

$$A_c = \text{the total gross area of concrete cross-section, (EC2 4.3.2.3)}$$

- Calculate the maximum design shear force that can be carried without crushing of the notional concrete compressive struts, $V_{Rd2,red}$.

$$V_{Rd2,red} = 1.67 V_{Rd2} \left(1 - \frac{\sigma_{cp,eff}}{f_{cd}} \right), \text{ where (EC2 4.3.2.2(4))}$$

$$0 \leq V_{Rd2,red} \leq V_{Rd2} , \quad (\text{EC2 4.3.2.2(4)})$$

$$V_{Rd2} = \frac{1}{2} v f_{c wd} (0.9 A_{cv}) , \quad (\text{EC2 4.3.2.3(3)})$$

$$v = 0.7 - \frac{f_{ck}}{200} \geq 0.5 , \text{ the effectiveness factor,} \quad (\text{EC2 4.3.2.3})$$

$$\sigma_{cp,eff} = \frac{N_{Sd} - f_{yd} A_{s2}}{A_c} , \text{ effective average stress in concrete,} (\text{EC2 4.3.2.2})$$

N_{Sd} = the design axial force, it is taken positive for compression and negative for tension, (EC2 4.3.2.2(4))

f_{yd} = the design yield stress of compression steel, (EC2 4.3.2.2(4))

$f_{yd} \leq 400$ MPa, (EC2 4.3.2.2(4))

A_{s2} is the area of the compression reinforcement, and (EC2 4.3.2.2(4))

A_c is the gross total area of concrete cross-section.

- The shear reinforcement per unit spacing is computed following the Standard method as follows (EC2 4.3.2.4.3):

If $V_{Sd} \leq V_{Rd1}$,

$$\frac{A_{sw}}{s} = 0 , \quad (\text{EC2 4.3.2.4.3})$$

else if $V_{Rd1} < V_{Sd} < V_{Rd2,red}$,

$$\frac{A_{sw}}{s} = \frac{(V_{Sd} - V_{Rd1}) / A_{cv}}{0.9 f_{ywd}} b_w , \text{ where} \quad (\text{EC2 4.3.2.4.3})$$

f_{ywd} is the design yield strength of the shear reinforcement.

else if $V_{Sd} \geq V_{Rd2,red}$,

a failure condition is declared. (EC2 4.3.2.2(4))

The maximum of all the calculated A_{sw} / 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.

- For all beams and at any station, the minimum area of shear reinforcement is imposed as follows:

$$\frac{A_{sw}}{s} \geq \rho_w b_w \quad (\text{EC2 5.4.2.2(5)})$$

In the above equation ρ_w is the shear reinforcement ratio. It is taken based on the concrete compression strength for shear use f'_{cs} which is the same as f_{cwk} and the shear rebar yield strength f_{ys} which is the same as f_{ywk} . ρ_w is determined from Table VII.2 according to the EC2 Table 5.5.

Chapter VIII

Design for NZS 3101-95

This chapter describes in detail the various aspects of the concrete design procedure that is used by SAP2000 when the user selects the New Zealand code, **NZS 3101-95** (NZS 1995). Various notations used in this chapter are listed in Table VIII-1.

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

SAP2000 provides options to design or check all types of moment resisting frames as required for regular and seismic design. For regular design, the frame should be identified as Ordinary. For Seismic design, the frame has to be identified as either Ductile, Limited, or Elastic to represent Ductile moment resisting frames, frames with Limited ductility, and Elastically responding frames, respectively. The details of the design criteria used for the different framing systems are described in the following sections.

English as well as SI and MKS metric units can be used for input. But the code is based on Newton-Millimeter-Second units. For simplicity, all equations and descriptions presented in this chapter correspond to **Newton-Millimeter-Second** units unless otherwise noted.

A_{cv}	Area of concrete used to determine shear stress, 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
a_{max}	Maximum depth of compression block, 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 (NZS 3.8.3.3)
f'_c	Specified compressive strength of concrete, MPA
f_y	Specified yield strength of flexural reinforcement, MPA
f_{yt}	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
k	Effective length factor
L	Clear unsupported length, mm

Table VIII-1

List of Symbols Used in the New Zealand code

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}	Nonsway component of factored end moment, N-mm
M_s	Sway component of factored end moment, N-mm
M^*	Factored moment at section, N-mm
M_x^*	Factored moment at section about X-axis, N-mm
M_y^*	Factored moment at section about Y-axis, N-mm
N_b	Axial load capacity at balanced strain conditions, N
N_c	Critical buckling strength of column, N
N_{max}	Maximum axial load strength allowed, N
N_0	Axial load capacity at zero eccentricity, N
N^*	Factored axial load at section, N
v_b	Basic shear stress resisted by concrete, MPA
v_c	Shear stress resisted by concrete, MPA
V_c	Shear force resisted by concrete, N
V_{D+L}	Shear force from span loading, N
V_p	Shear force computed from probable moment capacity, N
V^*	Factored shear force at a section, N
V_s	Shear force at a section resisted by steel, N
α	Reinforcing steel overstrength factor
α_1	Average stress factor in equivalent stress block
β_1	Factor for obtaining depth of compression block in concrete
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 nonsway moments
δ_s	Moment magnification factor for sway moments
ϵ_c	Strain in concrete
ϵ_s	Strain in reinforcing steel
ϕ_b	Strength reduction factor for bending

Table VIII-1*List of Symbols Used in the New Zealand code (continued)*

Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs 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, then the following load combinations may have to be considered for design of concrete frames (NZS 4203-92 2.4.3):

$$1.4 \text{ DL} \\ 1.2 \text{ DL} + 1.6 \text{ LL} \quad (\text{NZS 4203-92 2.4.3.3})$$

$$1.2 \text{ DL} \pm 1.0 \text{ WL} \\ 0.9 \text{ DL} \pm 1.0 \text{ WL} \\ 1.2 \text{ DL} + 0.4 \text{ LL} \pm 1.0 \text{ WL} \quad (\text{NZS 4203-92 2.4.3.3})$$

$$1.0 \text{ DL} \pm 1.0 \text{ EL} \\ 1.0 \text{ DL} + 0.4 \text{ LL} \pm 1.0 \text{ EL} \quad (\text{NZS 4203-92 2.4.3.3})$$

These are also the default design load combinations in SAP2000 whenever the NZS 3101-95 code is used. The user should use other appropriate loading combinations if roof live load is separately treated, other types of loads are present, or pattern live loads are to be considered.

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

Strength Reduction Factors

The default strength reduction factor, ϕ , is taken as

$$\phi_b = 0.85 \text{ for bending and} \quad (\text{NZS 3.4.2.2})$$

$$\phi_c = 0.85 \text{ for compression and} \quad (\text{NZS 3.4.2.2})$$

$$\phi_s = 0.75 \text{ for shear.} \quad (\text{NZS 3.4.2.2})$$

The user can, however, overwrite them.

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 II-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement ratios — 0.008 to 0.08 for Ordinary moment resisting frames (NZS 8.4.6.1) and 0.008 to $18/f_y$ for Seismic (Ductile, Limited, and Elastic) moment resisting frames (NZS 8.5.4.2).
- 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 the above-mentioned steps.

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 diagram is shown in Figure II-1.

The coordinates of these points of the interaction diagram are determined by rotating a plane of linear strain in three dimensions on the section of the column. See Figure II-2. The linear strain diagram limits the maximum concrete strain, ϵ_c , at the extremity of the section, to 0.003 (NZS 8.3.1.3).

The formulation is based consistently upon the general principles of ultimate strength design (NZS 8.3), and allows for any doubly symmetric rectangular, square, or circular column section.

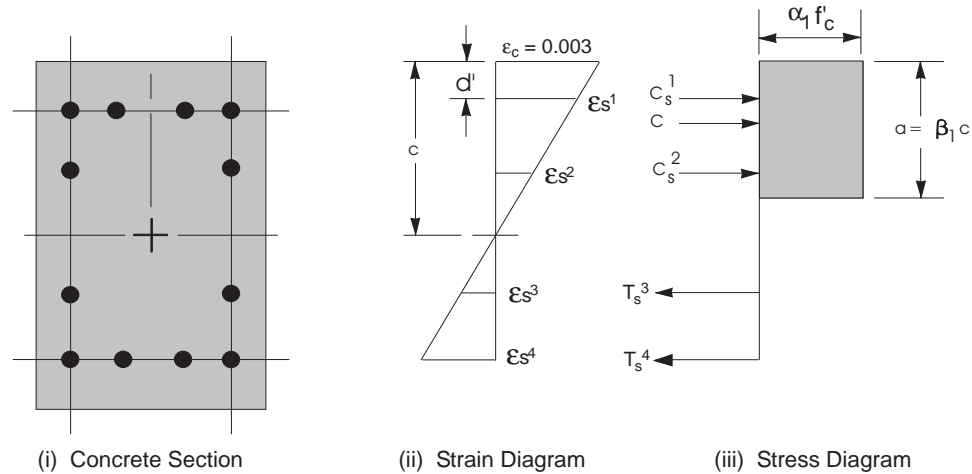


Figure VIII-1
Idealization of Stress and Strain Distribution in a Column Section

The stress in the steel is given by the product of the steel strain, ϵ_s , and the steel modulus of elasticity, E_s , and is limited to the yield stress of the steel, f_y (NZS 8.3.1.4). 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). See Figure VIII-1.

The concrete compression stress block is assumed to be rectangular, with a stress value of $\alpha_1 f'_c$ (NZS 8.3.1.7) and a depth of the stress block of $\beta_1 c$, where

$$\alpha_1 = 0.85 - 0.004(f'_c - 55), \quad (\text{NZS 8.3.1.7})$$

$$\beta_1 = 0.85 - 0.008(f'_c - 30), \quad (\text{NZS 8.3.1.7})$$

$$0.75 \leq \alpha_1 \leq 0.85, \text{ and} \quad (\text{NZS 8.3.1.7})$$

$$0.65 \leq \beta_1 \leq 0.85, \text{ and} \quad (\text{NZS 8.3.1.7})$$

In designing the column longitudinal reinforcement, the following limits are imposed on the steel tensile strength and the concrete compressive strength:

$$f_y \leq 500 \text{ MPA} \quad (\text{NZS 3.8.2.1})$$

$$f'_c \leq 100 \text{ MPA (Ordinary and Elastic)} \quad (\text{NZS 3.8.1.1})$$

$$f'_c \leq 70 \text{ MPa (Ductile and Limited)} \quad (\text{NZS 3.8.4.4})$$

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

The effects of the strength reduction factor, ϕ , are included in the generation of the interaction surfaces. The maximum compressive axial load is limited to N_{max} , where the maximum factored axial load resistance is given by:

$$N_{max} = 0.85 \phi_c [\alpha_1 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ (Ordinary, Elastic),} \quad (\text{NZS 8.4.1.5})$$

$$N_{max} = 0.70 \phi_c [\alpha_1 f'_c (A_g - A_{st}) + f_y A_{st}] \text{ (Ductile, Limited).} \quad (\text{NZS 8.5.1.4})$$

Check Column Capacity

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 station, the following steps are involved:

- Determine the factored moments and forces from the analysis load cases and the specified load combination factors to give N^* , M_x^* , and M_y^* .
- Determine the moment magnification factors for stability.
- Determine the dynamic moment magnification effect.
- 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 the above-mentioned steps.

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 N^* , M_x^* , and M_y^* . The factored moments are further increased, if required, to obtain minimum eccentricities of $(15 + 0.03h)$ mm, where h is the dimension of the column in the corresponding direction (NZS 8.4.1.5). The computed moments are further amplified by us-

ing “Moment Magnification Factors” to allow for “Lateral Drift Effect” and “Member Stability Effect”.

Determine Moment Magnification Factors

The moment magnification factors are applied in two different stages. First the moments are separated into their “sway” and “non-sway” components. The non-sway components are amplified for lateral drift effect. Although this amplification may be avoided for “braced” frames according to the code, SAP2000 treats all frames uniformly to amplify non-sway components of moments. These amplified moments are further amplified for individual member stability effect.

Lateral Drift Effect

For all frames, the moment magnification factor for lateral drift effect is applied only to the “sway” moment in SAP2000.

$$M = M_{ns} + \delta_s M_s$$

The moment magnification factors for moments causing sidesway in the major and minor directions, δ_{sx} and δ_{sy} , can be different. The moment magnification factors, δ_{sx} and δ_{sy} , can be taken as 1.0 if a P- Δ analysis is carried out. **The program assumes that the SAP2000 analysis models P- Δ effects, therefore, δ_{sx} and δ_{sy} are taken as 1.0.**

It is suggested that the P- Δ analysis be done at the factored load level (White and Hajjar 1991). The necessary factors for a P- Δ analysis for the NZS 3101-95 code should be $(1.0 \text{ DL} + 0.4 \text{ LL})/\phi_c$ with the loading standard NZS 4203, where ϕ_c is the strength reduction factor for compression and is equal to 0.85.

The user is reminded of the special analysis requirements, especially those related to the value of EI used in analysis (NZS 8.4.11.5). In SAP2000, the EI values are computed based on gross cross-section areas. The user has the option to reduce the EI values for analysis purposes using a scale factor on a section-by-section basis. If the program assumptions are not satisfactory for a particular member, the user can explicitly specify values of δ_{sx} and δ_{sy} .

Member Stability Effects

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

$$M_c = \delta_b M_2, \quad (\text{NZS 8.4.11.5})$$

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 associated with the major or minor direction of the column occurring either 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 is given by

$$\delta_b = \frac{C_m}{1 - \frac{N^*}{\phi_c N_c}} \geq 1.0, \text{ where} \quad (\text{NZS 8.4.11.5})$$

$$N_c = \frac{\pi^2 EI}{(kL)^2},$$

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

EI is associated with a particular column direction given by

$$EI = \frac{0.40 E_c I_g}{1 + \beta_d}, \text{ and} \quad (\text{NZS 8.4.11.5})$$

$$\beta_d = \frac{\text{Maximum factored axial dead load}}{\text{Maximum factored total axial load}}, \quad (\text{NZS 8.1})$$

$$C_m = 0.6 + 0.4 \frac{M_a}{M_b} \geq 0.4, \quad (\text{NZS 8.4.11.5})$$

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 above 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, $C_m = 1$ (NZS 8.4.11.5). C_m can be overwritten by the user on an element by element basis.

The magnification factor, δ_b , must be a positive number and greater than one. Therefore N^* must be less than $\phi_c N_c$. If N^* is found to be greater than or equal to $\phi_c N_c$, a failure condition is declared.

The above calculations use the unsupported lengths of the column. The two unsupported lengths are l_{22} and l_{33} corresponding to instability in the minor and major di-

rections of the element, respectively. See Figure II-4. These are the lengths between the support points of the element in the corresponding directions.

Dynamic Moment Magnification

For seismic design of Ductile frames and frames with Limited ductility, the moment is further amplified for dynamic effects of higher modes as follows (NZS 8.5.1.2, NZS 8.5.1.3):

$$M^* = R_m \left[\phi_0 \omega M_{col, joint, elastic} - 0.3 h_b V_{col}^* \right] \quad (\text{NZS A7.2})$$

where,

M^* = the design moment for column,

$M_{col, joint, elastic}$ = the column moment at the center of the joint obtained from linear elastic analysis,

V_{col}^* = the design shear for column,

R_m = the moment reduction factor which is taken as a function of axial force and the dynamic magnification factor, ω , for Ductile moment resisting frames (NZS Table A3). It is taken as 1 for frames with Limited ductility. The user can overwrite this,

ϕ_0 = an overstrength factor, and is taken as $1.25/\phi_b$ (NZS A4.1.6), and the user can overwrite this,

ω = the dynamic magnification factor. It is taken as 1, and the user can overwrite this, and

h_b = the overall dimension of the beam at the beam-column joint.

However, in the current implementation, the reduction due to the design shear force is taken as zero, which is based on a conservative assumption that h_b is equal to zero. The user can model this using end-offsets in SAP2000. In the current implementation of SAP2000, the three parameters R_m , ϕ_0 , and ω can be overwritten.

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 N^* , M_x^* , and M_y^* . The point (N^*, M_x^*, M_y^*) is then placed in the interaction space, shown as point L in Figure II-3. 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.

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 II-3. 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 N^* , M_x^* , and M_y^* set and associated load combination number.

If the reinforcing area is not defined, SAP2000 computes the reinforcement that will give an interaction ratio of unity.

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 following steps are involved:

- Determine the factored forces acting on the section, N^* and V^* . Note that N^* is needed for the calculation of v_c .
- Determine the shear stress, v_c , that can be resisted by concrete alone.
- Determine the reinforcing steel required to carry the balance.

For Seismic moment resisting Ductile frames and frames with Limited ductility, the shear design of the columns is based on the overstrength moment capacities of the column (NZS 4.4, 17.3.7.1).

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

Determine Section Forces

- In the design of the column shear reinforcement of an **Ordinary moment resisting concrete frame**, the forces for a particular load combination, namely, the column axial force, N^* , and the column shear force, V^* , in a particular direction are obtained by factoring the SAP2000 analysis load cases with the corresponding load combination factors.
- In the shear design of **Seismic moment resisting Ductile frames and frames of Limited ductility** (NOT Elastically responding frames) the following are checked in addition to the requirement for the Ordinary moment resisting frames (NZS 4.4.1.1). The design shear force, V^* , in a particular direction is also calculated from the overstrength moment capacities of the column associated with the factored axial force acting on the column (NZS 8.5.1.3).

For each load combination, the factored axial load, N^* , is calculated. Then, the positive and negative moment capacities, M_u^+ and M_u^- , of the column in a particular direction under the influence of the axial force N^* is calculated using the uniaxial interaction diagram in the corresponding direction. The design shear force, V^* , is then given by (NZS 4.4, NZS 17.3.7.1)

$$V^* = V_p + V_{D+L} \quad (\text{NZS 4.4.5.7, NZS 17.3.7.1})$$

where V_p is the shear force obtained by applying the calculated overstrength ultimate moment capacities at the two ends of the column acting in two opposite directions. Therefore, V_p is the maximum of V_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L},$$

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 ($\phi = 1.0$),

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 ($\phi = 1.0$), and

L = Clear span of column.

For Seismic moment resisting frames, α , is taken as 1.25 for both Ductile moment resisting frames and frames with Limited ductility (NZS A4.1.6). ϕ is taken as 1 (NZS 4.4.3.3). 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. See also for more details.

- For **Ductile moment resisting frames**, the shear capacity of the column is also checked for additional factored loads, in addition to the checks required for capacity design and factored loads. The factored shear force is based on the specified load combinations which are regular load combinations except the earthquake load factor is taken to be 1.7 (NZS 4.4.5.8).

Determine Concrete Shear Capacity

The shear capacity provided by the concrete alone, v_c , is calculated as follows:

- The basic shear strength for a rectangular section is computed as,

$$v_b = \left[0.07 + 10 \frac{A_{st}}{b_w d} \right] \sqrt{f'_c} \quad (\text{NZS 9.3.2.1})$$

where,

$$\sqrt{f'_c} \leq \sqrt{70}, \text{ and} \quad (\text{NZS 9.3.2.1})$$

$$0.08\sqrt{f'_c} \leq v_b \leq 0.2\sqrt{f'_c}. \quad (\text{NZS 9.3.2.1})$$

For other types of sections $b_w d$ is replaced by A_{cv} , the effective shear area, which is shown in Figure VIII-2.

- For Ordinary and Elastically responding moment resisting frames, the concrete shear capacity for columns is taken as follows:

$$v_c = \begin{cases} \left(1 + 12 \frac{N^*}{f_c' A_g}\right) v_b & \text{if under axial tension, } N^* < 0, \\ v_b & \text{if under flexure only, } N^* = 0, \\ \left(1 + 3 \frac{N^*}{f_c' A_g}\right) v_b & \text{if under compression, } N^* > 0. \end{cases} \quad (\text{NZS 9.3.2.1})$$

N^* is negative for tension. In any case, v_c is not taken less than zero.

- For Seismic moment resisting Ductile frames, the shear strength of a column section is taken as zero if the axial force is tensile or compression is very small. This is given as

$$v_c = 0 \quad \text{if } N^* \leq 0.10 f_c' A_g . \quad (\text{NZS 9.4.2.2})$$

If the axial compression is higher than the above limit, the concrete shear capacity for columns is taken as follows:

$$v_c = 4v_b \sqrt{\frac{N^*}{A_g f_c'} - 0.1} \quad \text{if } N^* > 0.10 f_c' A_g . \quad (\text{NZS 9.4.2.2})$$

In any case, v_c is not taken less than zero.

- For Seismic moment resisting frames with Limited ductility, the concrete shear capacity for columns is taken as follows (NZS 17.3.7.3 and NZS 17.3.7.4):

$$v_c = \begin{cases} \left(0.5 + 6 \frac{N^*}{f_c' A_g}\right) v_b & \text{if under axial tension, } N^* < 0, \\ 0.5 v_b & \text{if under flexure only, } N^* = 0, \\ \left(0.5 + 1.5 \frac{N^*}{f_c' A_g}\right) v_b & \text{if under compression, } N^* > 0. \end{cases} \quad (\text{NZS 17.3.7.4})$$

N^* is negative for tension. In any case, v_c is not taken less than zero (NZS 17.3.7.4).

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular section as,

$$v^* = \frac{V^*}{b_w d} . \quad (\text{NZS } 9.3.1.1)$$

For other types of sections $b_w d$ is replaced by A_{cv} , the effective shear area which is shown in .

- The average shear stress, v^* , is limited to a maximum limit, v_{max} , which is given for Ordinary frames and Elastically responding seismic frames as,

$$v_{max} = \min \left\{ 1.1\sqrt{f'_c}, 0.2f'_c, 9 \text{ MPa} \right\} \varphi_s \quad (\text{NZS } 9.3.1.8)$$

and for Ductile frames and frames with Limited ductility as,

$$v_{max} = \min \left\{ 0.85\sqrt{f'_c}, 0.16f'_c, 9 \text{ MPa} \right\} \varphi_s . \quad (\text{NZS } 9.4.4.2)$$

- The shear reinforcement per unit spacing is computed as follows:

$$\text{If } v^* \leq \varphi_s (v_c / 2) ,$$

$$\frac{A_v}{s} = 0 , \quad (\text{NZS } 9.3.4.1)$$

$$\text{else if } \varphi_s (v_c / 2) < v^* \leq \varphi_s (v_c + 0.35) ,$$

$$\frac{A_v}{s} = \frac{0.35b_w}{f_{yt}} , \quad f_{yt} \leq 500 \text{ MPa} , \quad (\text{NZS } 9.3.4.3)$$

$$\text{else if } \varphi_s (v_c + 0.35) < v^* \leq \varphi_s v_{max} ,$$

$$\frac{A_v}{s} = \frac{(v^* - \varphi_s v_c) b_w}{\varphi_s f_{yt}} , \quad f_{yt} \leq 500 \text{ MPa} , \quad (\text{NZS } 9.3.6.3)$$

$$\text{else if } v^* > v_{max} ,$$

$$\text{a failure condition is declared.} \quad (\text{NZS } 9.3.1.8)$$

In calculating the design shear reinforcement, a limit is imposed on the f_{yt} as

$$f_{yt} \leq 500 \text{ Mpa.} \quad (\text{NZS } 3.8.2.1 \text{ and NZS } 9.3.6.1)$$

The maximum of all 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.

In designing the column shear reinforcement, the following limits are imposed on the concrete compressive strength:

$$f'_c \leq 100 \text{ MPA (Ordinary and Elastic)} \quad (\text{NZS 3.8.1.1})$$

$$f'_c \leq 70 \text{ MPa (Ductile and Limited)} \quad (\text{NZS 3.8.4.4})$$

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

$$\frac{A_v}{s} \geq \left\{ \frac{1 - \rho_t m A_g}{3.3} \frac{f'_c}{A_c f_{yt}} \frac{N^*}{\phi_c f'_c A_g} - 0.0065 \right\} h_{core} \quad (\text{Stirrups}) \quad (\text{NZS 8.4.7.2})$$

$$\frac{A_v}{s} \geq \left\{ \frac{1 - \rho_t m A_g}{2.4} \frac{f'_c}{A_c f_{yt}} \frac{N^*}{\phi_c f'_c A_g} - 0.0084 \right\} \frac{h_{core}}{4} \quad (\text{Hoops}) \quad (\text{NZS 8.4.7.1})$$

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

$$\frac{A_v}{s} \geq \left\{ \frac{1.3 - \rho_t m A_g}{3.3} \frac{f'_c}{A_c f_{yt}} \frac{N^*}{\phi_c f'_c A_g} - 0.0060 \right\} h_{core} \quad (\text{Stirrups}) \quad (\text{NZS 8.5.4.3})$$

$$\frac{A_v}{s} \geq \left\{ \frac{1.3 - \rho_t m A_g}{2.4} \frac{f'_c}{A_c f_{yt}} \frac{N^*}{\phi_c f'_c A_g} - 0.0084 \right\} \frac{h_{core}}{4} \quad (\text{Hoops}) \quad (\text{NZS 8.5.4.3})$$

In the above four equations for calculating minimum shear reinforcement, the following limits are imposed:

$$\frac{A_g}{A_c} \geq 1.2 \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

$$\rho_t m \leq 0.4 \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

$$f_{yt} \leq 800 \text{ MPa} \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

For the definition of the potential plastic hinge, it is assumed in the current version of the program that any beam and column segment near the joint is a potential plas-

tic hinge. The length of the plastic hinge, L_{hinge} , in a column depends on the level of axial compression in it and it is taken as follows:

$$L_{hinge} = \begin{cases} h & \text{if } N^*/(\varphi_c f'_c A_g) < 0.25 \\ 2h & \text{if } 0.25 \leq N^*/(\varphi_c f'_c A_g) < 0.50 \\ 3h & \text{if } N^*/(\varphi_c f'_c A_g) \geq 0.50 \end{cases} \quad (\text{NZS 8.5.4.1})$$

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

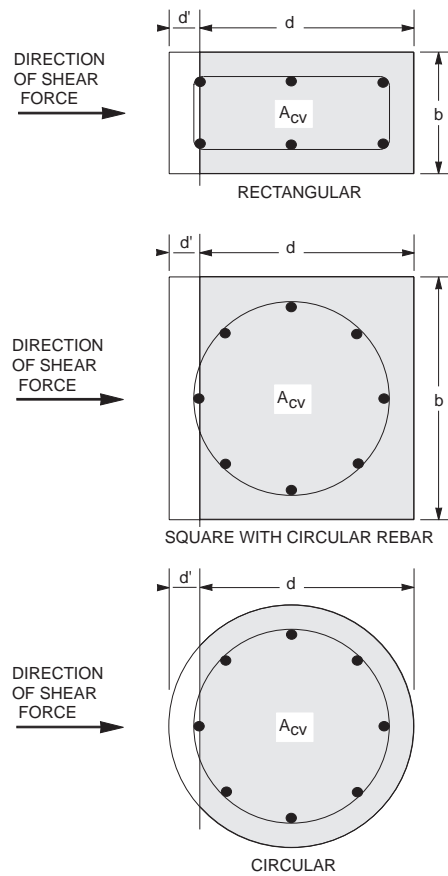


Figure VIII-2
Shear Stress Area, A_{cv}

Beam Design

In the design of concrete beams, SAP2000 calculates and reports the required areas of steel for flexure and shear based upon the beam moments, shears, load combination factors and other criteria described below. The reinforcement requirements are calculated at a user defined number of check stations along the beam span.

All the beams are only designed for major direction flexure and shear. 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

In designing the beam flexural and shear reinforcement, the following limits are imposed on the steel tensile strength and the concrete compressive strength:

$$f_y \leq 500 \text{ MPA} \quad (\text{NZS 3.8.2.1})$$

$$f'_c \leq 100 \text{ MPA (Ordinary and Elastic)} \quad (\text{NZS 3.8.1.1})$$

$$f'_c \leq 70 \text{ MPA (Ductile and Limited)} \quad (\text{NZS 3.8.4.4})$$

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 following steps are involved:

- Determine the maximum factored moments
- Determine the reinforcing steel

Determine Factored Moments

In the design of flexural reinforcement of Ordinary and Seismic (Ductile, Limited, and Elastic) moment resisting frame concrete 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 and maximum negative 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.

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 (NZS 8.3.1.6). Furthermore it is assumed that the compression carried by concrete is 0.75 times that which can be carried at the balanced condition (NZS 8.4.2). When the applied moment exceeds the moment capacity at the balanced condition, the area of compression reinforcement is calculated on the assumption that the additional moment will be carried by compression and additional tension reinforcement.

The design procedure used by SAP2000, for both rectangular and flanged sections (L- and T-beams) is summarized below. All the beams are designed only for major direction flexure and shear.

Design for Flexure of a Rectangular Beam

In designing for a factored negative or positive moment, M^* , (i.e. designing top or bottom steel) the depth of the compression block, a (See Figure VIII-3), is computed as,

$$a = d - \sqrt{d^2 - \frac{2|M^*|}{\alpha_1 f'_c \phi_b b}}, \quad (\text{NZS 8.3.1})$$

where the default value of ϕ_b is 0.85 (NZS 3.4.2.2) in the above and the following equations. Also α_1 is calculated as follows:

$$\alpha_1 = 0.85 - 0.004(f'_c - 55), \quad 0.75 \leq \alpha_1 \leq 0.85. \quad (\text{NZS 8.3.1.7})$$

Also β_1 and c_b are calculated as follows:

$$\beta_1 = 0.85 - 0.008(f'_c - 30), \quad 0.65 \leq \beta_1 \leq 0.85, \text{ and} \quad (\text{NZS 8.3.1.7})$$

$$c_b = \frac{600}{600 + f_y} d . \quad (\text{NZS 8.4.1.2})$$

The maximum allowed depth of the compression block is given by

$$a_{max} = 0.75\beta_1 c_b . \quad (\text{NZS 8.4.2 and NZS 8.3.1.7})$$

- If $a \leq a_{max}$ (NZS 8.4.2), the area of tensile steel reinforcement is then given by

$$A_s = \frac{M^*}{\phi_b f_y \left(d - \frac{a}{2} \right)} .$$

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

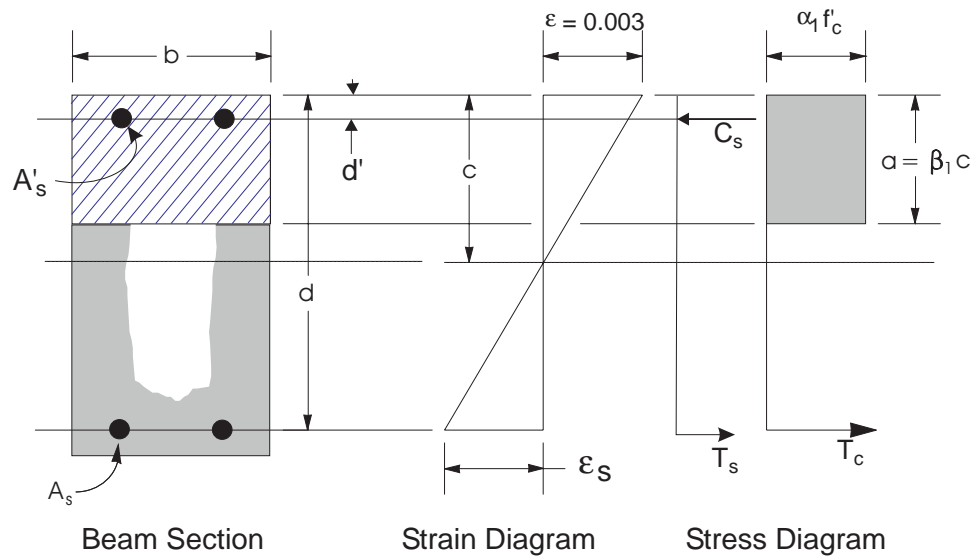


Figure VIII-3
Design of a Rectangular Beam Section

- If $a > a_{max}$ (NZS 8.4.2), compression reinforcement is required (NZS 8.4.1.3) and is calculated as follows:

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

$$C = \alpha_1 f'_c b a_{max}, \text{ and} \quad (\text{NZS 8.3.1.7})$$

the moment resisted by the concrete and bottom steel is

$$M_c^* = C \left(d - \frac{a_{max}}{2} \right) \phi_b.$$

- The moment resisted by compression steel and tensile steel is

$$M_s^* = M^* - M_c^*.$$

- So the required compression steel is given by

$$A'_s = \frac{M_s^*}{(f'_s - \alpha_1 f'_c)(d - d')} \phi_b, \text{ where}$$

$$f'_s = 0.003 E_s \left[\frac{c - d'}{c} \right] \leq f_y. \quad (\text{NZS 8.3.1.2 and NZS 8.3.1.3})$$

- The required tensile steel for balancing the compression in concrete is

$$A_{s1} = \frac{M_c^*}{f_y \left(d - \frac{a_{max}}{2} \right) \phi_b}, \text{ and}$$

the tensile steel for balancing the compression in steel is

$$A_{s2} = \frac{M_s^*}{f_y (d - d') \phi_b}.$$

- 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^* is positive, and vice versa.

Design for Flexure of a T-Beam**(i) Flanged Beam Under Negative Moment**

In designing for a factored negative moment, $M^* \leq 0$, (i.e. designing top steel), the calculation of the steel area is exactly the same as above, i.e., no T-Beam data is to be used.

(ii) Flanged Beam Under Positive Moment

If $M^* > 0$, the depth of the compression block is given by (see Figure VIII-4).

$$a = d - \sqrt{d^2 - \frac{2|M^*|}{\alpha_1 f'_c \phi_b b_f}}, \quad (\text{NZS 8.3.1})$$

The maximum allowed depth of the compression block is given by

$$a_{max} = 0.75 \beta_1 c_b. \quad (\text{NZS 8.4.2 and NZS 8.3.1.7})$$

- If $a \leq d_s$ (NZS 8.4.2), the subsequent calculations for A_s are exactly the same as previously done 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 whether $a > a_{max}$.
- If $a > d_s$ (NZS 8.4.2), calculation for A_s is done 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 ,

$$C_f = \alpha_1 f'_c (b_f - b_w) d_s. \quad (\text{NZS 8.3.1.7})$$

Therefore, $A_{s1} = \frac{C_f}{f_y}$ and the portion of M^* that is resisted by the flange is

given by

$$M_f^* = C_f \left(d - \frac{d_s}{2} \right) \phi_b.$$

Therefore, the balance of the moment, M^* to be carried by the web is given by

$$M_w^* = M^* - M_f^*.$$

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

$$a_1 = d - \sqrt{d^2 - \frac{2M_w^*}{\alpha_1 f'_c \phi_b b_w}} \quad (\text{NZS 8.3.1})$$

- If $a_1 \leq a_{max}$ (NZS 8.4.2), the area of tensile steel reinforcement is then given by

$$A_{s2} = \frac{M_w^*}{\phi_b f_y \left(d - \frac{a_1}{2} \right)}, \text{ and}$$

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

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

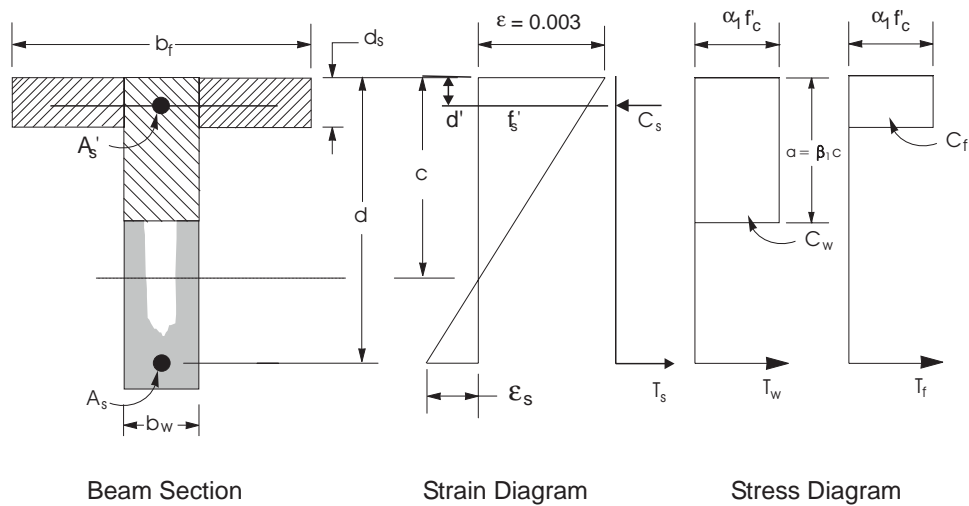


Figure VIII-4
Design of a T-Beam Section

- If $a_1 > a_{max}$ (NZS 8.4.2), compression reinforcement is required and is calculated as follows:
 - The compressive force in the concrete web alone is given by

$$C_w = \alpha_1 f'_c b a_{max} , \text{ and} \quad (\text{NZS 8.3.1.7})$$

the moment resisted by the concrete web and tensile steel is

$$M_c^* = C_w \left(d - \frac{a_{max}}{2} \right) \phi_b .$$

– The moment resisted by compression steel and tensile steel is

$$M_s^* = M_w^* - M_c^* .$$

– Therefore, the compression steel is computed as

$$A_s' = \frac{M_s^*}{(f_s' - \alpha_1 f_c')(d - d')} \phi_b , \text{ where}$$

$$f_s' = 0.003 E_s \left[\frac{c - d'}{c} \right] \leq f_y . \quad (\text{NZS 8.3.1.2 and NZS 8.3.1.3})$$

– The tensile steel for balancing compression in web concrete is

$$A_{s2} = \frac{M_c^*}{f_y \left(d - \frac{a_{max}}{2} \right) \phi_b} , \text{ and}$$

the tensile steel for balancing compression in steel is

$$A_{s3} = \frac{M_s^*}{f_y (d - d') \phi_b} .$$

– 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.

Minimum and Maximum Tensile Reinforcement

The minimum flexural tensile steel provided in a rectangular section in an Ordinary and Elastic moment resisting frame is given by the minimum of the two limits:

$$A_s \geq \frac{\sqrt{f_c'}}{4f_y} b_w d, \text{ or} \quad (\text{NZS 8.4.3.1})$$

$$A_s \geq \frac{4}{3} A_{s \text{ (required)}} . \quad (\text{NZS 8.4.3.3})$$

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

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

$$A'_s \leq \begin{cases} 0.04 b d & \text{Rectangular beam} \\ 0.04 b_w d & \text{T-beam} \end{cases}$$

Additional Consideration for Seismic Design

For Seismic moment resisting concrete Ductile frames and frames with Limited ductility (not Elastically responding structures), the following additional conditions are enforced for beam design (NZS 8.5.3.2 and NZS 17.3.5.4). See Table VIII-2 for comprehensive listing.

- The minimum tensile longitudinal reinforcement shall be provided at both at the top and bottom for both Ductile frames and frames with Limited ductility. Any of the top and bottom reinforcement shall not be less than $A_{s(min)}$ if tensile reinforcement is required.

$$A_{s(min)} \geq \frac{\sqrt{f'_c}}{4f_y} b_w d. \quad (\text{NZS 8.5.3.2})$$

- The beam flexural tensile steel is limited to a maximum given by

$$A_s \leq 0.025 b_w d \quad \text{and} \quad (\text{NZS 8.5.3.2, NZS 17.3.5.4})$$

$$A_s \leq \frac{f'_c + 10}{6f_y} b_w d. \quad (\text{NZS 8.5.3.2, NZS 17.3.5.4})$$

- At any section of a beam within a potential plastic hinge region for Ductile moment resisting frames, as defined later on page 171, the compression reinforcement area, A'_s , shall not be less than 1/2 of the tension reinforcement area, A_s , at the same section (NZS 8.5.3.2). At any section of a beam within a potential plastic hinge region for moment resisting frames with Limited ductility, the compression reinforcement area shall not be less than 3/8 of the tension reinforcement area at the same section (NZS 17.3.5.4).
- At least 1/4 of the larger of the top flexural reinforcement required at either end of the beam in a Ductile frame or in a frame with Limited ductility shall be continued throughout its length (NZS 8.5.3.2, 17.3.5.4).

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 following steps are involved:

- Determine the factored shear force, V^* .
- Determine the shear stress, v_c , that can be resisted by the concrete.
- Determine the reinforcing steel required to carry the balance.

For Seismic moment resisting frames, the shear design of the beams is also based on the overstrength moment capacities of the members.

The following three sections describe in detail the algorithms associated with the above-mentioned steps.

Determine Shear Force and Moment

- In the design of the beam shear reinforcement of an **Ordinary moment resisting 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 shear design of **Seismic moment resisting Ductile frames and frames of Limited ductility** (NOT Elastically responding frames), however, the shear force, V^* , is calculated from the overstrength moment capacities of each end of the beam, and the gravity shear forces (NZS 4.4.1.1). The procedure for calculating the design shear force in a beam from overstrength moment capacity is the same as that described for a column in section “Column Design” on page 154 (NZS 8.5.1.3). See also for more details.

The design shear force, V^* , is then given by (NZS 4.4, NZS 17.3.7.1)

$$V^* = V_p + V_{D+L} \quad (\text{NZS 4.4, NZS 17.3.7.1})$$

where, V_p is the shear force obtained by applying the calculated overstrength moment capacities at the two ends of the beams acting in two opposite directions. Therefore, V_p is the maximum of V_{P_1} and V_{P_2} , where

$$V_{P_1} = \frac{M_I^- + M_J^+}{L}, \text{ and}$$

$$V_{P_2} = \frac{M_I^+ + M_J^-}{L},$$

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 ($\phi = 1.0$),

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 ($\phi = 1.0$),

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 ($\phi = 1.0$),

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 ($\phi = 1.0$), and

L = Clear span of beam.

For Seismic moment resisting frames, α , is taken as 1.25 for both Ductile moment resisting frames and frames with Limited ductility (NZS A4.1.6). ϕ is taken as 1 (NZS 4.4.3.3). V_{D+L} is the contribution of shear force from the load in-span distribution of gravity loads.

Determine Concrete Shear Capacity

The shear capacity provided by the concrete alone, v_c , is calculated as follows:

- The basic shear strength for a rectangular section is computed as,

$$v_b = \left[0.07 + 10 \frac{A_s}{b_w d} \right] \sqrt{f'_c} \quad (\text{NZS 9.3.2.1})$$

where,

$$\sqrt{f'_c} \leq \sqrt{70}, \text{ and} \quad (\text{NZS 9.3.2.1})$$

$$0.08\sqrt{f'_c} \leq v_b \leq 0.2\sqrt{f'_c}. \quad (\text{NZS 9.3.2.1})$$

- For Ordinary and Elastically responding moment resisting frames, the concrete shear capacity for beams is taken as follows:

$$v_c = v_b. \quad (\text{NZS 9.3.2.1})$$

- For Seismic moment resisting Ductile frames, the shear strength of a beam section is taken as zero since the axial force is considered to be very small or tensile. This is given as

$$v_c = 0 . \quad (\text{NZS } 9.4.2.1)$$

- For Seismic moment resisting frames with Limited ductility, the concrete shear capacity for the beam is taken as follows:

$$v_c = 0.5 v_b . \quad (\text{NZS } 17.3.7.3)$$

Determine Required Shear Reinforcement

- The average shear stress is computed for a rectangular beam section as,

$$v^* = \frac{V^*}{b_w d} . \quad (\text{NZS } 9.3.1.1)$$

For other types of sections $b_w d$ is replaced by A_{cv} , the effective shear area which is shown in Figure VIII-2.

- The average shear stress, v^* , is limited to a maximum limit, v_{max} , which is given for Ordinary frames and Elastically responding seismic frames as,

$$v_{max} = \min \left\{ 1.1\sqrt{f'_c}, 0.2f'_c, 9 \text{ MPa} \right\} \phi_s \quad (\text{NZS } 9.3.1.8)$$

and for Ductile frames and frames with Limited ductility as,

$$v_{max} = \min \left\{ 0.85\sqrt{f'_c}, 0.16f'_c, 9 \text{ MPa} \right\} \phi_s . \quad (\text{NZS } 9.4.4.2)$$

- The shear reinforcement per unit spacing is computed as follows:

$$\text{If } v^* \leq \phi_s (v_c/2),$$

$$\frac{A_v}{s} = 0, \quad (\text{NZS } 9.3.4.1)$$

$$\text{else if } \phi_s (v_c/2) < v^* \leq \phi_s (v_c + 0.35),$$

$$\frac{A_v}{s} = \frac{0.35b_w}{f_{yt}}, \quad f_{yt} \leq 500 \text{ MPa} \quad (\text{NZS } 9.3.4.3)$$

else if $\varphi_s(v_c + 0.35) < v^* \leq \varphi_s v_{max}$,

$$\frac{A_v}{s} = \frac{(v^* - \varphi_s v_c) b_w}{\varphi_s f_{yt}}, \quad f_{yt} \leq 500 \text{ MPa} \quad (\text{NZS 9.3.6.3})$$

else if $v^* > v_{max}$,

a failure condition is declared. (NZS 9.3.1.8)

In calculating the design shear reinforcement, a limit is imposed on the f_{yt} as

$$f_{yt} \leq 500 \text{ Mpa.} \quad (\text{NZS 3.8.2.1 and NZS 9.3.6.1})$$

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

For all beams and at any station, the minimum area of transverse stirrup reinforcement is imposed as follows:

$$\frac{A_v}{s} \geq \left\{ \frac{1 - \rho_t m A_g f_c'}{3.3 A_c f_{yt} \varphi_c f_c' A_g} \frac{N^*}{\varphi_c f_c' A_g} - 0.0065 \right\} h_{core} \quad (\text{NZS 8.4.7.2})$$

In potential plastic hinge locations, as described later, of Seismic moment resisting Ductile frames and frames with Limited ductility, the minimum area of transverse stirrup is imposed as follows:

$$\frac{A_v}{s} \geq \left\{ \frac{1.3 - \rho_t m A_g f_c'}{3.3 A_c f_{yt} \varphi_c f_c' A_g} \frac{N^*}{\varphi_c f_c' A_g} - 0.0060 \right\} h_{core} \quad (\text{NZS 8.5.4.3})$$

In the above two equations for calculating the minimum shear reinforcement, the following limits are imposed:

$$\frac{A_g}{A_c} \geq 1.2 \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

$$\rho_t m \leq 0.4 \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

$$f_{yt} \leq 800 \text{ MPa} \quad (\text{NZS 8.4.7 and NZS 8.5.4.3})$$

For the definition of the potential plastic hinge, it is assumed in the current version of the program that any beam and column segment near the joint is a potential plastic hinge. The length of the plastic hinge, L_{hinge} , in a beam is taken as follows:

$$L_{hinge} = 2h \quad \text{(NZS 8.5.3.1)}$$

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

Type of Check/ Design	Ordinary (non-Seismic) or Elastically Responding (Seismic)	Moment Resisting Frames with Limited ductility or Elastically responding (Seismic)	Ductile Moment Resisting Frames (Seismic)
Column Check (interaction)	NLD ^a Combinations	NLD ^a Combinations $R_m = 1$	NLD ^a Combinations R_m is from NZS Table A3
Column Design (Interaction)	NLD ^a Combinations $0.008 < \rho < 0.08$ $0.008 < \rho < 18/f_y$ (Elastic only)	NLD ^a Combinations $\alpha = 1.0$ $0.008 < \rho < 18/f_y$ $R_m = 1$	NLD ^a Combinations $\alpha = 1.0$ $0.008 < \rho < 18/f_y$ R_m is from NZS Table A3
Column Shears	NLD ^a Combinations	Modified NLD ^a Combs Column Capacity Shear (V_p) $\phi = 1.0$ and $\alpha = 1.25$	NLD ^a Combinations and Column Capacity Shear (V_p) Extra Combo with $1.7 V_E$ $j = 1.0$ and $\alpha = 1.25$
Beam Design Flexure	NLD ^a Combinations	NLD ^a Combinations $\rho \leq 0.025$ $\rho \leq \frac{f'_c + 10}{6 f_y}$ $\rho \geq \frac{\sqrt{f'_c}}{4 f_y}$	NLD ^a Combinations $\rho \leq 0.025$ $\rho \leq \frac{f'_c + 10}{6 f_y}$ $\rho \geq \frac{\sqrt{f'_c}}{4 f_y}$
Beam Min. Moment Override Check	No Requirement	$A_{sEND}^+ \geq \frac{3}{8} A_{sEND}^-$ $A_{sSPAN}^+ \geq \frac{1}{4} \max\{A_{sEND}^-\}$	$A_{sEND}^+ \geq \frac{1}{2} A_{sEND}^-$ $A_{sSPAN}^+ \geq \frac{1}{4} \max\{A_{sEND}^-\}$
Beam Design Shear	NLD ^a Combinations $v_c = v_b$	NLD ^a Combinations Beam Capacity Shear (V_p) with $\alpha = 1.25$ and $\phi = 1.0$ plus V_{D+L} $v_c = v_b / 2$	NLD ^a Combinations Beam Capacity Shear (V_p) with $\alpha = 1.25$ and $\phi = 1.0$ plus V_{D+L} $v_c = 0$

^a NLD = Number of specified loading

Table VIII-2
Comparison of Ordinary and Seismic Moment Resisting Frame Design

Chapter IX

Design Output

Overview

SAP2000 creates design output in three major 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 which 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 the load combinations at all the design sections of a specific frame member. For a column member, it can also 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, and member-specific detailed design information are described. Some of the design information is specific to the chosen concrete design codes which are available in the program and are only described where required. The ACI 318-99 design code is described in the latter part of this chapter. For all other codes, the design outputs are similar.

Graphical Display of Design Output

The graphical output can be produced either as color screen display or in gray-scaled printed form. Moreover, the active screen display can be sent directly to the printer. The graphical display of design output includes input and output design information.

Input design information, for the ACI 318-99 code, includes

- Design section labels,
- K -factors for major and minor direction of buckling,
- Unbraced Length Ratios,
- C_m -factors,
- Live Load Reduction Factors,
- δ_s -factors, and
- δ_b -factors.

The output design information which can be displayed is

- Longitudinal Reinforcing,
- Shear Reinforcing, and
- Column Capacity Ratios.

The graphical displays can be accessed from the **Design** menu. For example, the longitudinal reinforcement can be displayed by selecting **Display Design Info...** from the **Design** menu. This will pop up a dialog box called **Display Design Results**. Then the user should switch ON the **Design Output** option button (default) and select **Longitudinal Reinforcing** in the drop-down box. Then clicking the **OK** button will show the longitudinal reinforcing in the active window.

The graphics can be displayed in either 3D or 2D mode. The SAP2000 standard view transformations are available for all concrete design output displays. For switching between 3D or 2D view of graphical displays, there are several buttons

on the main toolbar. Alternatively, the view can be set by choosing **Set 3D View...** from the **View** menu.

The graphical display in an active window can be printed in gray scaled black and white from the SAP2000 program. To send the graphical output directly to the printer, click on the **Print Graphics** button in the **File** menu. A screen capture of the active window can also be made by following the standard procedure provided by the Windows operating system.

Tabular Display of Design Output

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

The tabular design output includes input and output design information which depends on the design code of choice. For the ACI 318-99 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, and
 - Bar area.
- Concrete Beam Property Data
 - Material label,
 - Beam dimensions,
 - Top and bottom concrete cover, and
 - Top and bottom reinforcement areas.
- Load Combination Multipliers
 - Combination name,
 - Load types, and
 - Load factors.

- Concrete Design Element Information (code dependent)
 - Design Section ID,
 - K -factors for major and minor direction of buckling,
 - Unbraced Length Ratios,
 - C_m -factors,
 - Live Load Reduction Factors.
- Concrete Moment Magnification Factors (code dependent)
 - Section ID,
 - Element Type,
 - Framing Type,
 - δ_s -factors, and
 - δ_b -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, and
 - 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, and
 - Major shear reinforcement and the governing load combination.

The tabular output can be accessed by selecting **Print Design Tables...** from the **File** menu. This will pop up a dialog box. Then the user can specify the design quantities for which the results are to be tabulated. By default, the output will be sent to the printer. If the user wants the output stream to be redirected to a file, he/she can check the **Print to File** box. This will provide a default filename. The default filename can be edited. Alternatively, a file list can be obtained by clicking

the **File Name** button to choose a file from. Then clicking the **OK** button will direct the tabular output to the requested stream — the file or the printer.

Member Specific Information

The member specific design information shows the details of the calculation from the designer's point of view. It provides an access to the geometry and material data, other input data, design forces, design section dimensions, reinforcement details, and some of the intermediate results for a member. The design detail information can be displayed for a specific load combination and for a specific station of a frame member. For a column member, it can also show the position of the current state of design forces on the column interaction diagram.

The detailed design information can be accessed by **right clicking** on the desired frame member. This will pop up a dialog box called **Concrete Design Information** which includes the following tabulated information for the specific member. If the selected member is a column, the dialog box includes

- Load combination ID,
- Station location,
- Longitudinal reinforcement area,
- Major shear reinforcement area, and
- Minor shear reinforcement area.

If the selected member is a beam, the dialog box includes

- Load combination ID,
- Station location,
- Top reinforcement area,
- Bottom reinforcement area, and
- Shear reinforcement area.

Additional information can be accessed for column members by clicking on the **ReDesign**, **Details**, and **Interaction** buttons in the dialog box. For beams additional information can be accessed by clicking on the **ReDesign** and **Details** buttons in the dialog box.

Additional information that is available by clicking on the **ReDesign** button is as follows:

- Design Factors (code dependent)
 - Effective length factors, K , for major and minor direction of buckling,
 - Unbraced Length Ratios,
 - C_m -factors,
 - Live Load Reduction Factors,
 - δ_s -factors, and
 - δ_b -factors.
- Element Section ID
- Element Framing Type

Additional information that is available by clicking on the **Details** button is given below. The details of this information depends on whether the selected member is a beam or a column. If the member is a column, the information includes:

- Frame, Section, Station, and Load Combination IDs,
- Section geometric information and graphical representation,
- Material properties of steel and concrete,
- Design axial force and biaxial moments,
- Minimum design moments,
- Moment factors,
- Longitudinal reinforcing areas,
- Design shear forces,
- Shear reinforcing areas, and
- Shear capacities of concrete and steel.

If the member is a beam, the information includes:

- Frame, Section, Station, and Load Combination IDs,
- Section geometric information and graphical representation,
- Material properties of steel and concrete,
- Design moments and shear forces,
- Minimum design moments,
- Top and bottom reinforcing areas,
- Shear capacities of concrete and steel, and

- Shear reinforcing area.

Clicking on the **Interaction** button displays the interaction diagram in a three dimensional space for the column section. The design axial force and the biaxial moments are plotted on the interaction diagram to show the state of stress in the column. The interaction diagram can be viewed in any orientation and the view can be manipulated from the interaction dialog box. The interaction diagram can be printed for hard-copy output.

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