



**Concrete Frame  
Design Manual**

**British Standard for  
Structural Use of Concrete**

**BS 8110-1997**

**For ETABS® 2016**

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Computers & Structures, Inc.

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# Chapter 1

## Introduction

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

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

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

In the design of columns, the program calculates the required longitudinal and shear reinforcement. However, the user may specify the longitudinal steel, in which case a column capacity ratio is reported. The column capacity ratio gives an indication of the stress condition with respect to the capacity of the column.

The biaxial column capacity check is based on the generation of consistent three-dimensional interaction surfaces. It does not use any empirical

formulations that extrapolate uniaxial interaction curves to approximate biaxial action.

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

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

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

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

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

## 1.1 Organization

This manual is designed to help the user quickly become productive with the concrete frame design options of the British Standard for Structural Use of Concrete BS 8110-1997, which is referred to as BS in this manual. Chapter 2 provides detailed descriptions of the Design Prerequisites used for BS 8110-1997. Chapter 3 provides detailed descriptions of the code specific process used for 8110-1997. Chapter 4 documents the design output produced by the programs. The appendices provide details on certain topics referenced in this manual.

## 1.2 Recommended Reading/Practice

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



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## Chapter 2

# Design Prerequisites

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

In writing this manual it has been assumed that the user has an engineering background in the general area of structural reinforced concrete design and familiarity with the BS 8110-1997 code.

### 2.1 Design Load Combinations

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

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

Separate combinations with negative factors for response spectrum cases are not required because the program automatically takes the minima to be the negative of the maxima for response spectrum cases and the permutations described in the preceding text 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. The combinations are based on the code recommendations and are documented for each code in the corresponding manuals.

For other loading conditions involving moving load, time history, pattern live loads, separate consideration of roof live load, snow load, 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 load cases declared as dead load to be additive. Similarly, all cases declared as live load are assumed additive. However, each load case declared as wind or earthquake, or response spectrum cases, is assumed to be non additive with each other and produces multiple lateral load combinations. Also wind and static earthquake cases produce separate loading combinations with the sense (positive or negative) reversed. If these conditions are not correct, the user must provide the appropriate design combinations.

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

## 2.2 Design and Check Stations

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

## 2.3 Identifying Beams and Columns

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

## 2.4 Design of Beams

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

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

In designing the flexural reinforcement for the major moment at a particular section of a particular beam, the steps involve the determination of the maximum factored moments and the determination of the reinforcing steel. The beam section is designed for the maximum positive and maximum negative factored moment envelopes obtained from all of the load combinations. Negative beam moments produce top steel. In such cases, the beam is always designed as a rectangular section. Positive beam moments produce bottom steel. In such cases, the beam may be designed as a rectangular 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.

## 2.5 Design of Columns

In the design of columns, the program calculates the required longitudinal steel, or if the longitudinal steel is specified, the column stress condition is reported in terms of a column capacity ratio, which is a factor that gives an indication of the stress condition of the column with respect to the capacity of the column. The design procedure for the reinforced concrete columns of the structure involves the following steps:

- Generate axial force-biaxial moment interaction surfaces for all of the different concrete section types of the model.
- Check the capacity of each column for the factored axial force and bending moments obtained from each loading combination at each end of the column. This step is also used to calculate the required reinforcement (if none was specified) that will produce a capacity ratio of 1.0.

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

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

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

## 2.6 P-Delta Effects

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

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

For lateral drift effects, the program assumes that the P-delta analysis is performed and that the amplification is already included in the results. The moments and forces obtained from P-delta analysis are further amplified for individual column stability effect if required by the governing code as in the SS CP65 codes.

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

## 2.7 Element Unsupported Lengths

To account for column slenderness effect, 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. The program, however, allows users to assign several elements to be treated as a single member for design. This can be done differently for major and minor bending as documented in Appendix B of this design manual.

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

## 2.8 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 subsequently correspond to that specific system of units, unless otherwise noted. For example, the BS 8110-1997 code is published in Newton-Millimeter-Second units. By default, all equations and descriptions presented in the chapter "Design for BS 8110-1997" correspond to Newton-Millimeter-Second units. However, any system of units can be used to define and design the structure in the program.

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## Chapter 3

### Design for BS 8110-1997

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This chapter describes in detail the various aspects of the concrete design procedure that is used by the program when the user selects the British Standard design code BS 8110-1997 incorporating Amendments Nos. 1, 2 and 3. For simplicity, all equations and descriptions presented in this chapter correspond to Newton-Millimeter-Second units unless otherwise noted.

### 3.1 Notation

The various notations used in this chapter are described herein:

$A_{cv}$	Area of section for shear resistance, mm <sup>2</sup>
$A_g$	Gross area of cross-section, mm <sup>2</sup>
$A_s$	Area of tension reinforcement, mm <sup>2</sup>
$A'_s$	Area of compression reinforcement, mm <sup>2</sup>
$A_{sc}$	Total area of column longitudinal reinforcement, mm <sup>2</sup>
$A_{sv}$	Total cross-sectional area of links at the neutral axis, mm <sup>2</sup>
$A_{sv}/s_v$	Area of shear reinforcement per unit length of the member, mm <sup>2</sup> /mm

$a$	Depth of compression block, mm
$b$	Width or effective width of the section in the compression zone, mm
$b'$	Shorter section dimension, 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 200,000 MPa
$e_{\min}$	Minimum 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 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
$K$	Normalized design moment, $M_u/bd^2f_{cu}$
$K'$	Maximum $\frac{M_u}{bd^2f_{cu}}$ 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 for support compression
$k_2$	Concrete shear strength factor, $[f_{cu}/25]^{1/3}$
$l_e$	Effective height of a column, mm
$l_o$	Clear height between end restraints, mm



$M$	Design moment at a section, N-mm
$M_1, M_2$	Smaller and larger end moments in slender 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 the links along the length of the beam, mm
$T$	Tension force, N
$V$	Design shear force at ultimate design load, N
$v$	Design shear stress at a beam cross-section or at a punch critical section, 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
$\beta$	Effective length factor
$\beta_b$	Moment redistribution factor in a member
$\gamma_f$	Partial safety factor for load
$\gamma_m$	Partial safety factor for material strength
$\epsilon_c$	Maximum concrete strain, 0.0035
$\epsilon_s$	Strain in tension steel
$\epsilon'_s$	Strain in compression steel

## 3.2 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,  $\gamma_f$  (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 subjected to wind (WL), and considering that those loads are subject to reversals, the following load combinations for ultimate limit state should be considered (BS 2.4.3):

$$1.4D \quad (BS\ 2.4.3)$$

$$1.4D+1.6L$$

$$1.0D \pm 1.4W$$

$$1.4D \pm 1.4W \quad (BS\ 2.4.3)$$

$$1.2D+1.2L \pm 1.2W$$

$$1.0D \pm 1.4E$$

$$1.4D \pm 1.4E \quad (BS\ 2.4.3)$$

$$1.2D+1.2L \pm 1.2W$$

These are the default load combinations. In addition to these load combinations, the code requires that all buildings 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 to consider notional load in the program.

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.

### 3.3 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,  $\gamma_m$ . The values of  $\gamma_m$  used in the program are listed as follows (BS 2.4.4.1).

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

### 3.4 Column Design

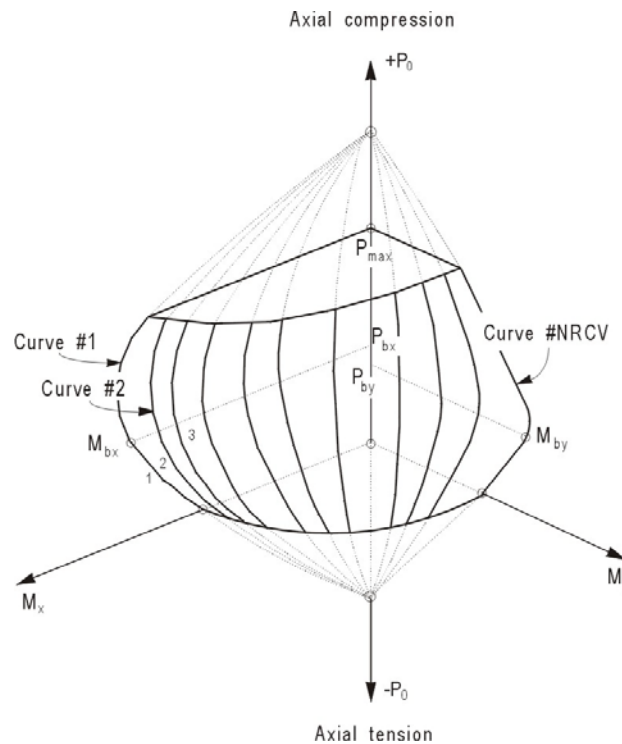
The user may define the geometry of the reinforcing bar configuration of each concrete column section. If the area of reinforcing is provided by the user, the program checks the column capacity. However, if the area of reinforcing is not provided by the user, the program calculates the amount of reinforcing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

- Generate axial force/biaxial moment interaction surfaces for all of the different concrete section types of the model. A typical biaxial interaction surface is shown in Figure 3-1. When the steel is undefined, the program generates the interaction surfaces for the range of allowable reinforcement from 0.4 to 6 percent (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 these steps.

### 3.4.1 Generation of Biaxial Interaction Surfaces

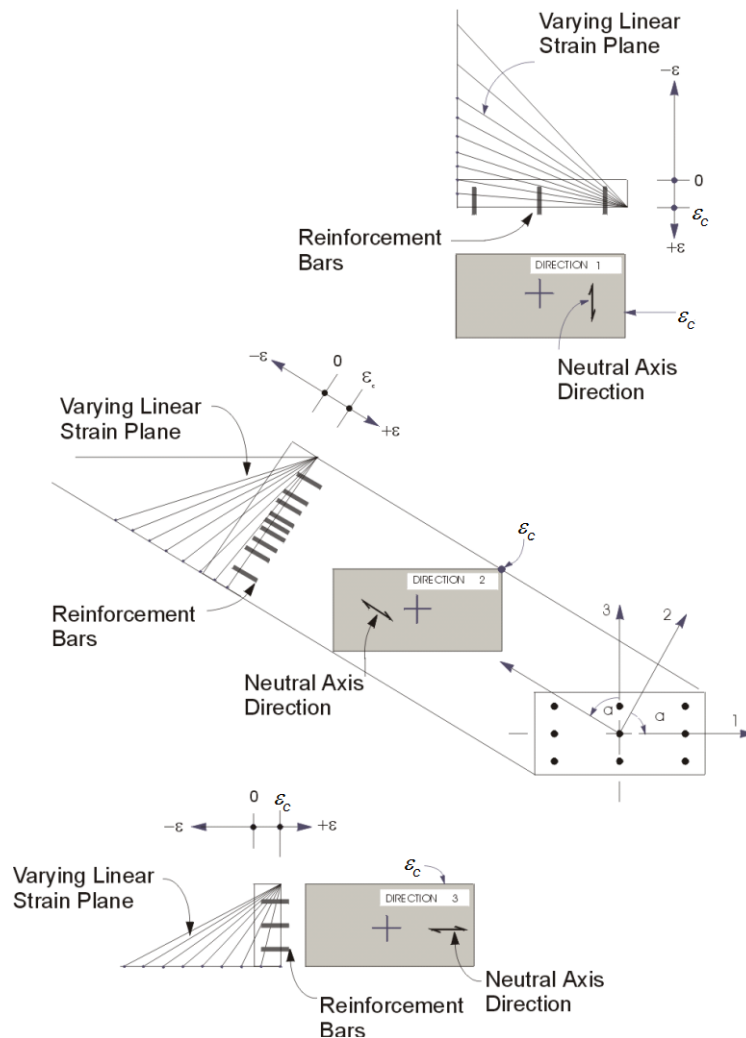
The column capacity interaction volume is numerically described by a series of discrete points that are generated on the three-dimensional interaction failure surface. In addition to axial compression and biaxial bending, the formulation allows for axial tension and biaxial bending considerations (BS 3.8.4.1). A typical interaction diagram is shown in Figure 3-1.



*Figure 3-1 A typical column interaction surface*

The coordinates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column (BS 3.4.4.4). See Figure 3-2. The linear strain diagram limits the maximum concrete strain,  $\epsilon_c$ , at the extremity of the section, to 0.0035 (BS 3.4.4.1).

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



**Figure 3-2** Idealized strain distribution for generation of interaction surface

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity,  $\epsilon_s E_s$ , and is limited to the design strength of the steel,  $(f_y/1.15)$ . 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 3-3.

The concrete compression stress block is assumed to be rectangular, with a stress value of  $0.67 f_{cu} / \gamma_m = 0.45 f_u$  (BS 3.4.4.1). See Figure 3-3. The interaction algorithm provides corrections to account for the concrete area that is displaced by the reinforcement in the compression zone.

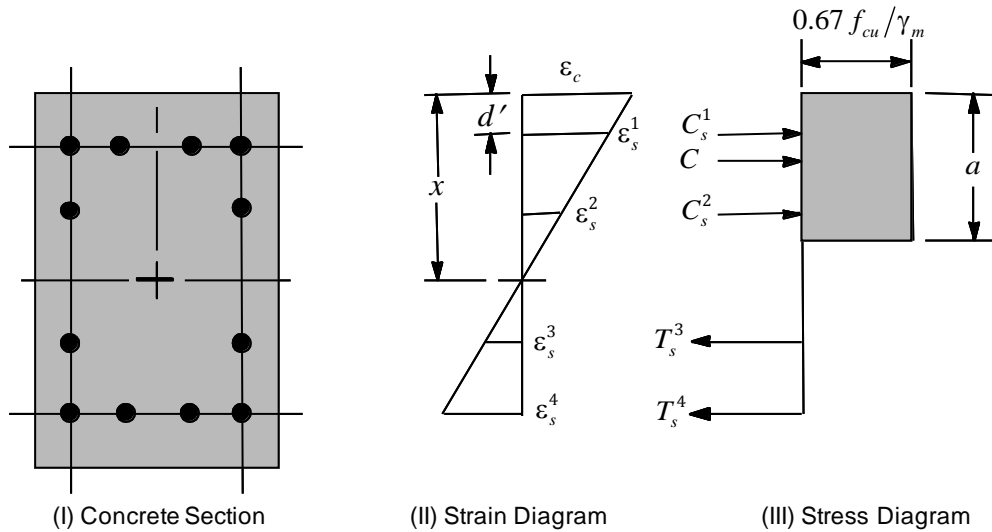


Figure 3-3 Idealization of stress and strain distribution in a column section

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,

$$\frac{x}{d} \leq 0.5 \quad (\text{BS 3.4.4.4})$$

to safeguard against non-ductile failures (BS 3.4.4.4). In addition, the area of compression reinforcement is calculated assuming that the neutral axis depth remains at the maximum permitted value.

The depth of the compression block is given by:

$$a = 0.9x \quad (\text{BS 3.4.4.4})$$

## 3.4.2 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 program uses the following steps:

- Determine the factored moments and forces from the load cases and the specified load combination factors to give  $N$ ,  $V_2$ ,  $V_3$ ,  $M_{33}$ , and  $M_{22}$ .
- 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 these steps.

### 3.4.2.1 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  $N$ ,  $V_2$ ,  $V_3$ ,  $M_{33}$ , and  $M_{22}$ .

### 3.4.2.2 Determine Additional Moments

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

### 3.4.2.2.1 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),

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

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

$$a_u = \beta_a Kh \quad \text{and} \quad (\text{BS 3.8.3.1})$$

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

In the preceding 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  $\beta$  is the effective length factor, and  $l_0$  is the unsupported length corresponding to instability in the major or minor direction of the element,  $l_{33}$  or  $l_{22}$ . In calculating the value of the effective length, the  $\beta$  factor is conservatively taken as 1. However, the program allows the user to overwrite 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.

$K$  is the correction factor to the deflection to take care of the influence of the axial force, and  $K$  is conservatively taken as 1.

The program 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, the maximum design moment for the column is taken as the greatest of the following:



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

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

$$M_1 + \frac{M_{\text{add}}}{2} \quad (\text{BS 3.8.3.2})$$

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

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

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

$M_1$  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.

$e_{\text{min}}$  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 3.8.3.4).

$$e_{\text{min}} = \frac{h}{20} \leq 20 \text{ mm}. \quad (\text{BS 3.8.3.4})$$

#### 3.4.2.2.2 Unbraced Column

In the case of the unbraced column, it is assumed that the program analysis includes P-delta 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 be at least

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

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.

### 3.4.2.3 Determine Capacity Ratio

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

Before entering the interaction diagram to check the column capacity, the design forces  $N$ ,  $M_{33}$ , and  $M_{22}$  are obtained according to the previous subsections. The point  $N$ ,  $M_{33}$ , and  $M_{22}$  is then placed in the interaction space shown as point L in Figure 3-4. If the point lies within the interaction volume, the column capacity is adequate; however, if the point lies outside the interaction volume, the column is overstressed.

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 3-4. The capacity ratio,  $CR$ , is given by the ratio  $\frac{OL}{OC}$ .

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

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

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

The maximum of all of the values of  $CR$  calculated from each load combination is reported for each check station of the column along with the controlling  $N$ ,  $M_{33}$ , and  $M_{22}$  set and associated load combination number.

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

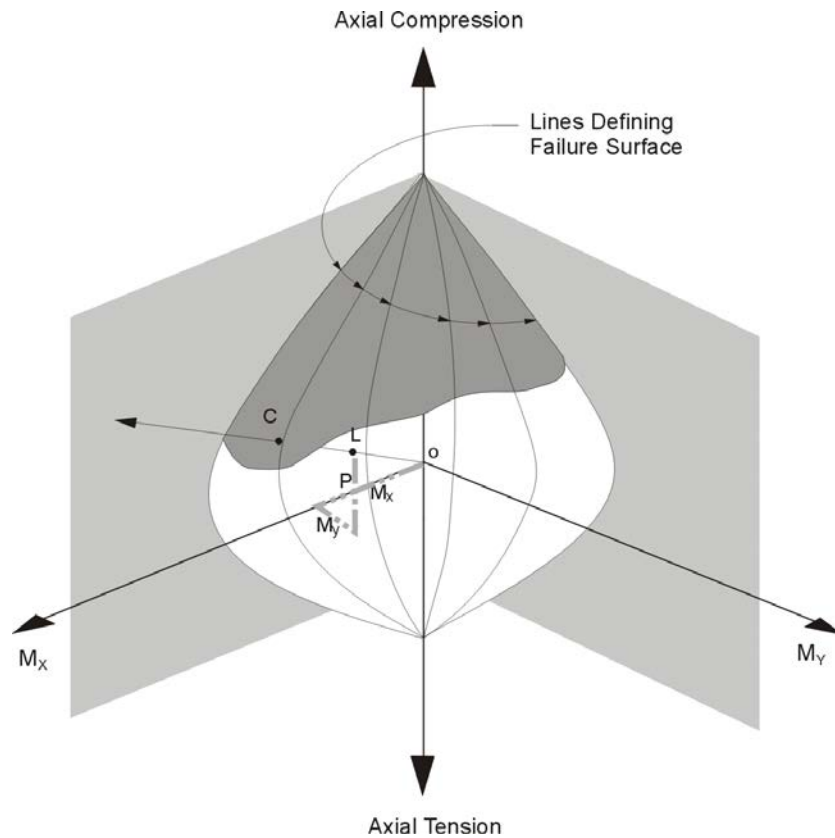


Figure 3-4 Geometric representation of column capacity ratio

### 3.4.3 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

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

The maximum allowable shear stress,  $v_{\max}$  is defined as:

$$v_{\max} = \min(0.8\sqrt{f_{cu}}, 5 \text{ MPa}) \quad (\text{BS 3.4.5.12})$$

– If  $v_{\max}$  exceeds the limit, the section area should be increased.

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

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

$$v_c = \frac{0.79k_1k_2}{\gamma_m} \left( \frac{100A_s}{bd} \right)^{1/3} \left( \frac{400}{d} \right)^{1/4} \quad (\text{BS 3.4.5.4})$$

where,

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

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

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

However the following limitations also apply:

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

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

$$\left( \frac{400}{d} \right)^4 \geq 1.00 \quad (\text{BS 3.4.5.4})$$

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

$A_s$  is the area of tensile steel, which is assumed to be half of total rebar.

- Calculate the design average shear stress that can be carried by minimum transverse rebar,  $v_r$ , as follows:

$$v_r = 0.4 \frac{N}{\text{mm}^2} \quad (\text{BS 3.4.5.3})$$

– If  $v \leq v'_c + v_r$ , provide minimum links defined by

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

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

$$\frac{A_{sv}}{s_v} = \frac{(v - v'_c) b}{0.87 f_{yv}}. \quad (\text{BS 3.4.5.3, Table 3.8})$$

- If  $v > v_{\text{max}}$ , a failure condition is declared. (BS 3.4.5.2)

In the preceding expressions, a limit is imposed on the  $f_{yv}$  as

$$f_{yv} \leq 500 \text{ MPa}. \quad (\text{BS 3.4.5.1})$$

## 3.5 Beam Design

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

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

The beam design procedure involves the following steps:

- Design beam flexural reinforcement
- Design beam shear reinforcement

## 3.5.1 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

### 3.5.1.1 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.

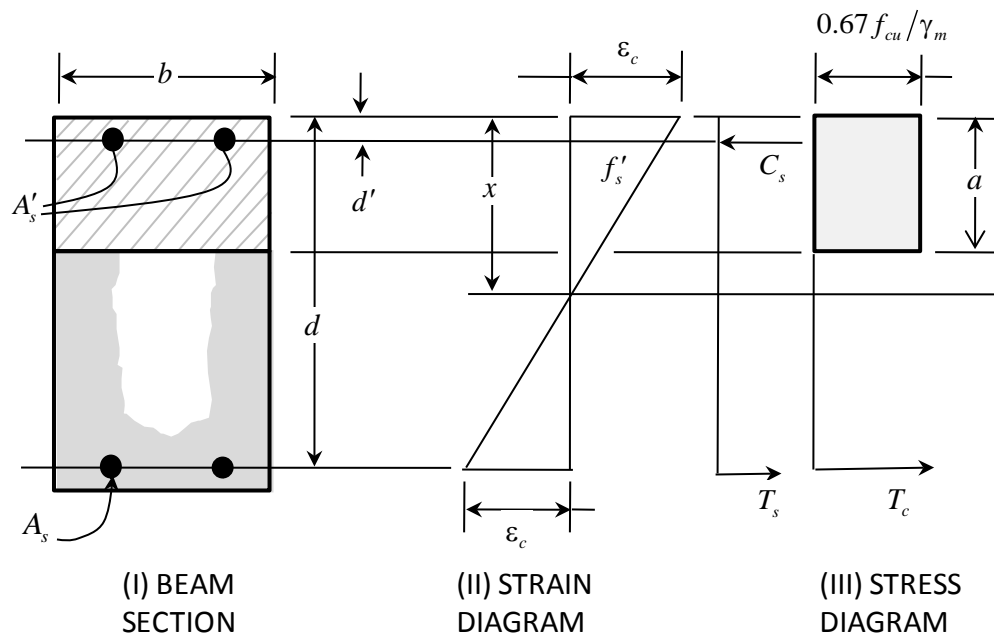
Negative beam moments produce top steel. In such cases, the beam is always designed as a Rectangular section. Positive beam moments produce bottom steel. In such cases, the beam may be designed as a Rectangular section, or T beam effects may be included.

### 3.5.1.2 Determine Required Flexural Reinforcement

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

The design procedure is based on the simplified rectangular stress block shown in Figure 3-5 (BS 3.4.4.4), where  $\epsilon_c$  is defined as:

$$\epsilon_c = 0.0035 \quad (\text{BS 2.5.3})$$



*Figure 3-5 Rectangular beam design*

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,

$$\frac{x}{d} \leq 0.5 \quad (\text{BS 3.4.4.4})$$

to safeguard against non-ductile failures (BS 3.4.4.4). In addition, the area of compression reinforcement is calculated assuming that the neutral axis depth remains at the maximum permitted value.

The depth of the compression block is given by:

$$a = 0.9x \quad (\text{BS 3.4.4.4})$$

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

### 3.5.1.2.1 Design of a Rectangular Beam

For rectangular beams, the moment capacity as a singly reinforced beam,  $M_{\text{single}}$ , is obtained first for a section. The reinforcing steel area is determined based on whether  $M$  is greater than, less than, or equal to  $M_{\text{single}}$ . Please see Figure 3-5.

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

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

$$K' = 0.156$$

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

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

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

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

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

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

$$A'_s = \frac{M - M_{\text{single}}}{\left( f'_s - \frac{0.67 f_{cu}}{\gamma_m} \right) (d - d')} \quad (\text{BS 3.4.4.4})$$

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

$$f'_s = 0.87 f_y \quad \text{if } d'/d \leq \frac{1}{2} \left[ 1 - \frac{f_y}{800} \right] \quad (\text{BS 3.4.4.1})$$



$$f'_s = E_s \varepsilon_c \left[ 1 - \frac{2d'}{d} \right] \text{ if } d'/d > \frac{1}{2} \left[ 1 - \frac{f_y}{800} \right] \quad (\text{BS 3.4.4.4})$$

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K'}{0.9}} \right\}. \quad (\text{BS 3.4.4.4})$$

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

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

### 3.5.1.2.2 Design as a T Beam

#### 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 3-6.

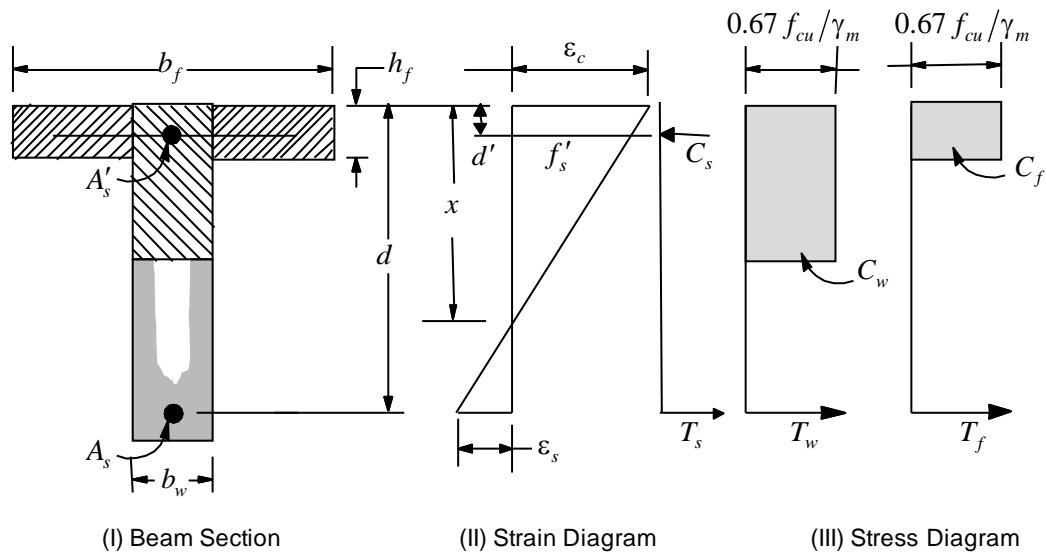


Figure 3-6 T beam design

**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, the contribution of the web to the flexural strength of the beam is taken into account. Please see Figure 3-6.

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

$$K = \frac{M}{f_{cu} b_f d^2}. \quad (\text{BS 3.4.4.4})$$

Then the moment arm is computed as

$$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\} \leq 0.95d, \quad (\text{BS 3.4.4.4})$$

the depth of neutral axis is computed as

$$x = \frac{d - z}{0.45} \quad (\text{BS 3.4.4.4})$$

the depth of compression block is given by

$$a = 0.9x. \quad (\text{BS 3.4.4.4})$$

- 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 that case, the width of the compression flange,  $b_f$ , is taken as the width of the beam,  $b$ , for analysis. Compression reinforcement is required if  $K > K'$ .
- If  $a > h_f$ , calculation for  $A_s$  is performed in two parts. The first part is for balancing the compressive force from the flange,  $C_f$ , and the second part is for balancing the compressive force from the web,  $C_w$ , as shown in Figure 3-6.

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

$$M_f = \frac{0.67}{\gamma_m} f_{cu} (b_f - b_w) h_f (d - 0.5h_f),$$

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

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

- If  $K_w \leq K_f$ , 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.

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

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

- If  $K_w \leq K'$ , 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

$$A'_s = \frac{M_w - M_{iw}}{\left(f'_s - \frac{0.67f_{cu}}{\gamma_m}\right)(d - d')}$$

where,

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

$$f'_s = 0.87f_y \quad \text{if } d'/d \leq \frac{1}{2} \left[1 - \frac{f_y}{800}\right] \quad (\text{BS 3.4.4.4})$$

$$f'_s = E_s \varepsilon_c \left[1 - \frac{2d'}{d}\right] \quad \text{if } d'/d > \frac{1}{2} \left[1 - \frac{f_y}{800}\right] \quad (\text{BS 3.4.4.4})$$

The area of tension reinforcement is obtained from equilibrium

$$A_s = \frac{1}{0.87f_y} \left[ \frac{M_f}{d - 0.5h_f} + \frac{M_{iw}}{z} + \frac{M_w - M_{iw}}{d - d'} \right], \text{ where}$$

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

### 3.5.1.3 Minimum and Maximum Tensile Reinforcement

The minimum flexural tensile steel required for a beam section is given by the following table, which is taken from BS Table 3.25 (BS 3.12.5.3), with interpolation for reinforcement of intermediate strength:

Section	Situation	Definition of percentage	Minimum percentage	
			$f_y = 250 \text{ MPa}$	$f_y = 460 \text{ MPa}$
Rectangular	—	$100 \frac{A_s}{bh}$	0.24	0.13

Section	Situation	Definition of percentage	Minimum percentage	
			$f_y = 250 \text{ MPa}$	$f_y = 460 \text{ MPa}$
T or L beam with web in tension	$\frac{b_w}{b_f} < 0.4$	$100 \frac{A_s}{b_w h}$	0.32	0.18
	$\frac{b_w}{b_f} \geq 0.4$	$100 \frac{A_s}{b_w h}$	0.24	0.13
T beam with web in compression	—	$100 \frac{A_s}{b_w h}$	0.48	0.26
L beam with web in compression	—	$100 \frac{A_s}{b_w h}$	0.36	0.20

The minimum flexural compression steel, if it is required at all, provided in a rectangular beam or T beam section is given by the following table, which is taken from BS Table 3.25 (BS 3.12.5.3), with interpolation for reinforcement of intermediate strength:

Section	Situation	Definition of percentage	Minimum percentage
Rectangular	—	$100 \frac{A_s'}{bh}$	0.20
T beam	Web in tension	$100 \frac{A_s'}{b_f h_f}$	0.40
	Web in compression	$100 \frac{A_s'}{b_w h}$	0.20

In addition, an upper limit on both the tension reinforcement and compression reinforcement has been imposed to be 0.04 times the gross cross-sectional area (BS 3.12.6.1).

### 3.5.2 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

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

the maximum allowable shear stress,  $v_{\max}$  is defined as:

$$v_{\max} = \min(0.8\sqrt{f_{cu}}, 5 \text{ MPa}) \quad (\text{BS 3.4.5.2})$$

– If  $v$  exceeds the limits, the section area should be increased.

- Calculate the design concrete shear stress from

$$v_c = \frac{0.79k_1k_2\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}}{\gamma_m}, \quad (\text{BS 3.4.5.2})$$

where,

$k_1$  is the enhancement factor for support compression,

and  $k_1$  is conservatively taken as 1, (BS 3.4.5.8)

$$k_2 = \left(\frac{f_{cu}}{30}\right)^{1/3}, \text{ and } 1 \leq k_2 \leq \left(\frac{80}{30}\right)^{1/3} \quad (\text{BS 3.4.5.4})$$

$\gamma_m = 1.25.$  (BS 2.4.4.1)

However, the following limitations also apply:

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

$$\left(\frac{400}{d}\right)^{1/4} \geq 1 \quad (\text{BS 3.4.5.4})$$

$$f_{cu} \leq 40 \text{ MPa (for calculation purpose only)} \quad (\text{BS 3.4.5.4})$$

$A_s$  is the area of tension reinforcement.

- Calculate the design average shear stress that can be carried by minimum transverse rebar,  $v_r$ , as follows:

$$v_r = 0.4 + v_c \quad (\text{BS 3.4.5.3})$$

$A_s$  is the area of tensile steel.

– If  $v \leq v_c + v_r$ , provide minimum links defined by

$$\frac{A_s}{s_v} = \frac{v_r b}{0.87 f_{yv}}, \quad (\text{BS 3.4.5.3})$$

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

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

In the preceding expressions, a limit is imposed on the  $f_{yv}$  as

$$f_{yv} \leq 500 \text{ MPa} \quad (\text{BS 3.4.5.1})$$

The maximum of all of the calculated  $A_{sv}/s_v$  values, obtained from each load combination, is reported along with the controlling shear force and associated load combination.

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

## **APPENDICES**



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## Appendix A

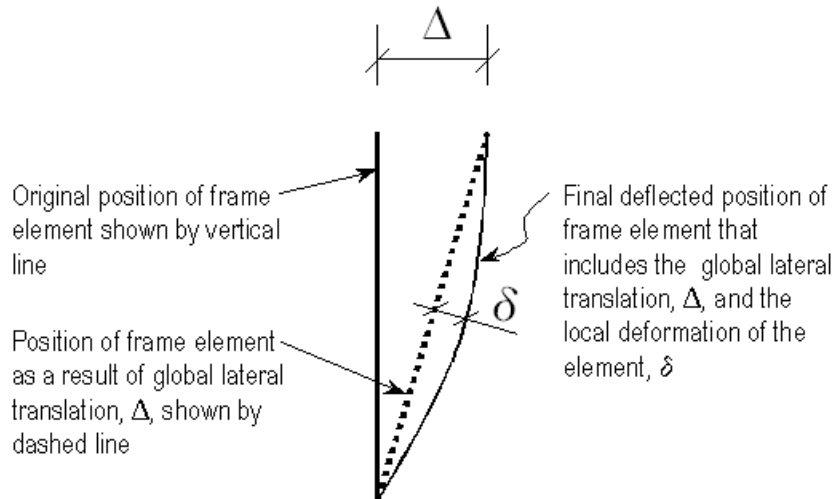
### Second Order P-Delta Effects

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

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

The program has an option to consider P-delta effects in the analysis. When P-delta effects are considered in the analysis, the program does a good job of capturing the effect due to the  $\Delta$  deformation shown in Figure A-1, but it does not typically capture the effect of the  $\delta$  deformation (unless, in the model, the frame object is broken into multiple elements over its length).



**Figure A-1 The Total Second Order P-Delta Effects on a Frame Element Caused by Both  $\Delta$  and  $\delta$**

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

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

$M_{CAP}$  = Flexural design capacity required

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

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

$a$  = Unitless factor multiplying  $M_{nt}$

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

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

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## Appendix B

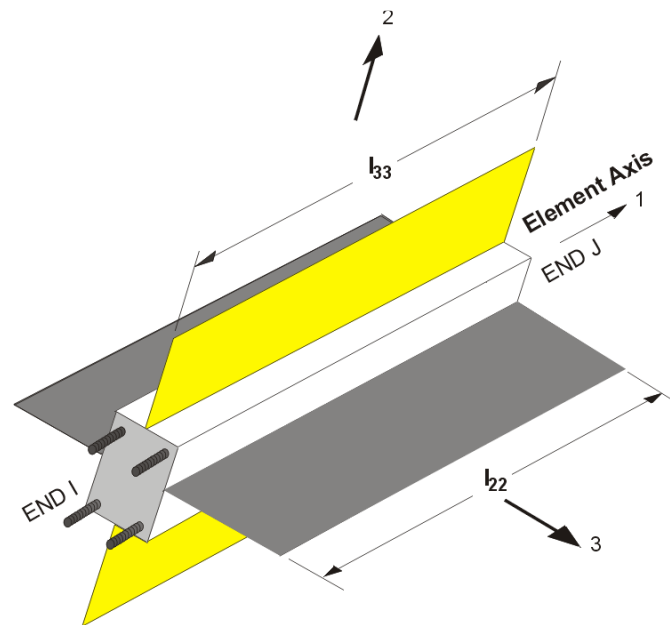
### Member Unsupported Lengths and Computation of $K$ -Factors

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

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

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



*Figure B-1 Axis of bending and unsupported length*

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

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## Appendix C

### Concrete Frame Design Preferences

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The Concrete Frame Design Preferences are basic assignments that apply to all of the concrete frame members. Table C-1 lists the Concrete Frame Design Preferences for the BS 8110-97 code. Default values are provided for all preference items. Thus, it is not necessary to specify or change any of the preferences. However, at least review the default values to ensure they are acceptable. Some of the preference items also are available as member specific overwrite items. The Overwrites are described in Appendix D. Overwritten values take precedence over the preferences.

**Table C-1 Design Criteria**

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

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Item	Possible Values	Default Value	Description
Number Interaction Curves	Multiple of 4 $\geq 4$	24	Number of equally spaced interaction curves used to create a full 360 <i>deg</i> interaction surface (this item should be a multiple of four). We recommend 24 for this item.
Number	Any odd value $\geq 5$	11	Number of points used to define a single curve in a concrete frame; should be odd.
Consider Minimum Eccentricity	No, Yes	Yes	Toggle to consider if minimum eccentricity should be considered in design.
Gamma (Steel)	$> 0$	1.15	The strength reduction factor for shear and torsion.
Gamma (Concrete)	$> 0$	1.50	The strength reduction factor for shear in structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects.
Gamma (Concrete Shear)	$> 0$	1.25	The strength reduction factor for shear and torsion.
Phi (Pattern Live Load Factor)	$\geq 0$	0.75	The strength reduction factor for shear in structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects.
Utilization Factor Limit	$> 0$	0.95	Stress ratios that are less than or equal to this value are considered acceptable.

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## Appendix D Concrete Frame Overwrites

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The concrete frame design overwrites are basic assignments that apply only to those elements to which they are assigned. Table D-1 lists concrete frame design overwrites for BS 8110-1997. Default values are provided for all overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values to ensure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned.

**Table D-1 Design Criteria**

<b>Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Description</b>
Current Design Section	Any defined concrete section	Analysis section	The design section for the selected frame objects. When this overwrite is applied, any previous auto select section assigned to the frame object is removed.

Item	Possible Values	Default Value	Description
Element Type	Sway NonSway	From Reference	Frame type per moment frame definition given in BS 8110-1997.
Live Load Reduction Factor	$\geq 0$	Calculated	The reduced live load factor. A reducible live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying 0 means the value is program determined.
Unbraced Length Ratio (Major)	$\geq 0$	Calculated	Unbraced length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined.
Unbraced Length Ratio (Minor)	$\geq 0$	Calculated	Unbraced length factor for buckling about the frame object minor axis. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined. This factor is also used in determining the length for lateral-torsional buckling.
Effective Length Factor (Major)	$\geq 0$	Calculated	Effective length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. Specifying 0 means the value is program determined.
Effective Length Factor (Minor)	$\geq 0$	Calculated	Effective length factor for buckling about the frame object minor axis. This item is specified as a fraction of the frame object length. Specifying 0 means the value is program determined.



Item	Possible Values	Default Value	Description
Moment Coefficient (Cm Major)	$\geq 0$	Calculated	Unitless factor for major axis bending, used in determining the interaction ratio. This item applies only to frame objects with column-type current design sections. Program determined value means it is calculated for each element for each load combination uniquely. Specifying 0 means the value is program determined.
Moment Coefficient (Cm Minor)	$\geq 0$	Calculated	Unitless factor for minor axis bending, used in determining the interaction ratio. This item applies only to frame objects with column-type current design sections. Program determined value means it is calculated for each element for each load combination uniquely. Specifying 0 means the value is program determined.
Nonsway Moment Factor (Dns Major)	$\geq 0$	Calculated	Unitless moment magnification factor for non-sway major-axis bending moment. This item applies only to frame objects with column-type current design sections. Program determined value means it is calculated for each element for each load combination uniquely. Specifying 0 means the value is program determined.
Nonsway Moment Factor (Dns Minor)	$\geq 0$	Calculated	Unitless moment magnification factor for non-sway minor-axis bending moment. This item applies only to frame objects with column-type current design sections. Program determined value means it is calculated for each element for each load combination uniquely. Specifying 0 means the value is program determined.

Item	Possible Values	Default Value	Description
Nonsway Moment Factor (Ds Major)	$\geq 0$	Calculated	Unitless moment magnification factor for sway major-axis bending moment. Specifying 0 means the value is program determined. The program determined value is taken as 1 because it is assumed that P-Delta effects were specified to be included in the analysis, and thus no further magnification is required. This item applies only to frame objects with column-type current design sections.
Nonsway Moment Factor (Ds Minor)	$\geq 0$	Calculated	Unitless moment magnification factor for sway minor-axis bending moment. Specifying 0 means the value is program determined. The program determined value is taken as 1 because it is assumed that P-Delta effects were specified to be included in the analysis, and thus no further magnification is required. This item applies only to frame objects with column-type current design sections.

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