COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS® 2015

Integrated Building Design Software

Shear Wall Design Manual

IS 456:2000 & 13920:1993

Shear Wall Design Manual

IS 456:2000 and IS 13920:1993

For ETABS® 2015

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Chapter 1 Introduction

This chapter describes in detail the various aspects of the shear wall design procedure that is used by the program when the user selects the Indian IS 456-2000 code option. This covers the basic design code "IS 456:2000 Indian Standard Plain and Reinforced Concrete Code of Practice" (IS 2000), the seismic code "IS 13920:1993 (Reaffirmed 1998, Edition 1.2, 2002-2003), Indian Standard – Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice" (IS 2002), and a part of the draft seismic code (Jain and Murty 2008). Various notation used in this chapter are listed in Section 1.1. For referencing to the pertinent sections of the Indian codes in this manual, a prefix "IS" followed by the section number is used. The relevant prefixes are "IS," "IS 13920," and "IS 13920 Draft" for the basic code IS 456:2000, the seismic code IS 13920:1993, and the draft seismic code, respectively.

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

The design is based on loading combinations specified by the user. To facilitate the design process, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures (Section 1.3).

The program performs the following design, check, or analysis procedures in accordance with IS 456:2000 and IS 13920:1993 requirements:

- Design and check of concrete wall piers for flexural and axial loads (Chapter 2)
- Design of concrete wall piers for shear (Chapter 2)
- Design of concrete shear wall spandrels for flexure (Chapter 3)
- Design of concrete wall spandrels for shear (Chapter 3)
- Consideration of the boundary element requirements for concrete wall piers using an approach based on the requirements of Section 9.4 in IS 13920:1993 (Chapter 2)

The program provides detailed output data for Simplified pier section design, Section Designer pier section design, Section Designer pier section check, and spandrel design (Chapter 4).

1.1 Notation

This section provides the notations used in this manual.

A_{cv}	Net area of a wall pier bounded by the length of the wall pier, L_w, and the web thickness, t_p, mm^2
A_g	Gross area of a wall pier, mm^2
$A_{h\text{-}min}$	Minimum required area of distributed horizontal reinforcing steel required for shear in a wall spandrel, mm^2/mm
A_s	Area of reinforcing steel, mm^2
A'_s	Area of compression reinforcement, mm^2
A_{sc}	Area of reinforcing steel required for compression in a pier edge member, or the required area of tension steel required to balance the compression steel force in a wall spandrel, mm^2

$A_{sc\text{-max}}$	Maximum area of compression reinforcing steel in a wall pier edge member, mm^2
A_{sf}	The required area of tension reinforcing steel for balancing the concrete compression force in the extruding portion of the concrete flange of a T-beam, mm^2
A_{st}	Area of reinforcing steel required for tension in a pier edge member, mm^2
$A_{st\text{-}max}$	Maximum area of tension reinforcing steel in a wall pier edge member, mm^2
A_{sv}/S_v	Area of reinforcing steel required for shear pier unit length, mm^2/mm
A_{vd}	Area of diagonal shear reinforcement in a coupling beam, mm^2
$A_{v\text{-min}}$	Minimum required area of distributed vertical reinforcing steel required for shear in a wall spandrel, mm^2/mm
A_{sw}	The required area of tension reinforcing steel for balancing the concrete compression force in a rectangular concrete beam, or for balancing the concrete compression force in the concrete web of a T-beam, mm^2
A'_s	Area of compression reinforcing steel in a spandrel, mm^2
B_1, B_2,...	Length of a concrete edge member in a wall with uniform thickness, mm
C_c	Concrete compression force in a wall pier or spandrel, Newton
C_f	Concrete compression force in the extruding portion of a T-beam flange, Newton
C_s	Compression force in wall pier or spandrel reinforcing steel, Newton

C_w	Concrete compression force in the web of a T-beam, Newton
D	Overall depth of a spandrel, mm
D_f	Flange thickness in a T-beam spandrel, mm
D/C	Demand/capacity ratio as measured on an interaction curve for a wall pier, unitless
$DB1$	Length of a user-defined wall pier edge member, mm. This can be different on the left and right sides of the pier, and it also can be different at the top and the bottom of the pier.
$DB2$	Width of a user-defined wall pier edge member, mm. This can be different on the left and right sides of the pier, and it also can be different at the top and the bottom of the pier.
E_c	Modulus of elasticity of concrete, N/mm^2
E_s	Modulus of elasticity of reinforcement, assumed as 200,000 N/mm^2
H_w	Height of a wall spandrel, mm. This can be different on the left and right ends of the spandrel.
IP-max	The maximum ratio of reinforcing considered in the design of a pier with a Section Designer section, unitless
IP-min	The minimum ratio of reinforcing considered in the design of a pier with a Section Designer section, unitless
L_{BZ}	Horizontal length of the boundary zone at each end of a wall pier, mm
LL	Live load
L_s	Horizontal length of wall spandrel, mm
L_w	Horizontal length of wall pier, mm. This can be different at the top and the bottom of the pier

M_n	Nominal bending strength, N-mm
M_{single}	Design moment resistance of a section as a singly reinforced section, N-mm
M_u	Ultimate factored design moment at a section object, N-mm
M_{uc}	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression and the tension steel, N-mm
M_{uf}	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the extruding portion of the flange and the tension steel, N-mm
M_{us}	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the compression steel and the tension steel, N-mm
M_{uw}	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the web and the tension steel, N-mm
OC	On a wall pier interaction curve the “distance” from the origin to the capacity associated with the point considered
OL	On a wall pier interaction curve the “distance” from the origin to the point considered
P_b	The axial force in a wall pier at a balanced strain condition, Newton
PC_{max}	Maximum ratio of compression steel in an edge member of a wall pier, unitless

P_{left}	Equivalent axial force in the left edge member of a wall pier used for design, pounds. This may be different at the top and the bottom of the wall pier.
$P_{\max}$	Limit on the maximum compressive design strength specified by IS 456:2000, Newton
$P_{\max}$ Factor	Factor used to reduce the allowable maximum compressive design strength, unitless. IS 13920:1993 specifies this factor to be 0.80. This factor can be revised in the preferences.
P_n	Nominal axial strength, Newton
P_O	Nominal axial load strength of a wall pier, Newton
P_{oc}	The maximum compression force a wall pier can carry with strength reduction factors set equal to one, Newton
P_{ot}	The maximum tension force a wall pier can carry with strength reduction factors set equal to one, Newton
P_{right}	Equivalent axial force in the right edge member of a wall pier used for design, pounds. This may be different at the top and the bottom of the wall pier.
P_u	Factored axial force at a design section, Newton
$PT_{\max}$	Maximum ratio of tension steel in an edge member of a wall pier, unitless
RLL	Reduced live load
T_s	Tension force in wall pier reinforcing steel, Newton
V_c	The portion of the shear force carried by the concrete, Newton
V_n	Nominal shear strength, Newton
V_s	The portion of the shear force in a spandrel carried by the shear reinforcing steel, Newton

V_u	Shear force of ultimate design load, Newton
WL	Wind load
b_f	Width or effective width of flange in a flanged spandrel beam, mm
b_s	Width of the compression flange in a T-beam, mm. This can be different on the left and right end of the T-beam.
d	Effective depth of tension reinforcement in a spandrel, mm
d'	Effective depth of compression reinforcement, mm
$d_{\text{compression}}$	Depth of center of compression block from most compressed face, mm
$d_{\text{r-bot}}$	Distance from bottom of spandrel beam to centroid of the bottom reinforcing steel, inches. This can be different on the left and right ends of the beam.
$d_{\text{r-top}}$	Distance from top of spandrel beam to centroid of the top reinforcing steel, mm. This can be different on the left and right ends of the beam.
d_s	Depth of the compression flange in a T-beam, mm. This can be different on the left and right ends of the T-beam.
d_{spandrel}	Depth of spandrel beam minus cover to centroid of reinforcing, mm
f_{cd}	Design concrete strength = f_{ck}/γ_c, N/mm^2
f_{ck}	Characteristic compressive strength of concrete, N/mm^2
f_s'	Stress in compression steel of a wall spandrel, N/mm^2
f_y	Characteristic strength of reinforcement, N/mm^2
f_{yd}	Design yield strength of reinforcing steel = f_y/γ_s, N/mm^2
f_{ys}	Characteristic strength of shear reinforcement, N/mm^2

k	Enhancement factor of shear strength for depth of the beam
m	Normalized design moment, $M_u / \alpha f_{ck} d b^2$
$p_{\max}$	Maximum ratio of reinforcing steel in a wall pier with a Section Designer section that is designed (not checked), unitless.
$p_{\min}$	Minimum ratio of reinforcing steel in a wall pier with a Section Designer section that is designed (not checked), unitless.
s_v	Spacing of the shear reinforcement along the length of the spandrel beam, mm
t_p	Thickness of a wall pier, mm. This can be different at the top and bottom of the pier.
t_s	Thickness of a wall spandrel, mm. This can be different on the left and right ends of the spandrel.
x_u	Depth of neutral axis, mm
$x_{u,\max}$	Maximum permitted depth of neutral axis, mm
z	Lever arm, mm
α	Concrete strength reduction factor for sustained loading
β	Factor for the depth of compressive force resultant of the concrete stress block
δ	Enhancement factor of shear strength for compression
ε	Reinforcing steel strain, unitless
$\varepsilon_{c,\max}$	Maximum concrete strain in the beam and slab (= 0.0035)
ε_s	Strain in tension steel
ε_s'	Strain in compression steel

γ_c	Partial safety factor for concrete strength
γ_f	Partial safety factor for load
γ_m	Partial safety factor for material strength
γ_s	Partial safety factor for steel strength
σ_s	Reinforcing steel stress in a wall pier, N/mm^2
ρ	Tension reinforcement ratio, A_s/bd
τ_c	Basic design shear stress resisted by concrete, N/mm^2
$\tau_{c,\max}$	Maximum possible design shear stress permitted at a section, MPa
τ_{cd}	Design shear stress resisted by concrete, N/mm^2
τ_v	Average design shear stress resisted by the section, N/mm^2

1.2 Design Station Locations

The program designs wall piers at stations located at the top and bottom of the pier only. To design at the mid-height of a pier, break the pier into two separate “half-height” piers.

The program designs wall spandrels at stations located at the left and right ends of the spandrel only. To design at the mid-length of a spandrel, break the spandrel into two separate “half-length” piers. Note that if you break a spandrel into pieces, the program will calculate the seismic diagonal shear reinforcing separately for each piece. The angle used to calculate the seismic diagonal shear reinforcing of each piece is based on the length of the piece, not the length of the entire spandrel. This can cause the required area of diagonal reinforcing to be significantly underestimated. **Thus if you break a spandrel into pieces, calculate the seismic diagonal shear reinforcing separately by hand.**

1.3 Default Design Load Combinations

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

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

As an example, if a structure is subjected to dead load, D, and live load, L, only, the IS 456:2000 design check may need one design load combination only, namely, 1.5 D+1.5 L. However, if the structure is subjected to wind, earthquake or other loads, numerous additional design load combinations may be required.

For the IS 456:2000 code, if a structure is subjected to dead (D), live (L), pattern live (PL), wind (W), and earthquake (E) loads, and considering that wind and earthquake forces are reversible, the following load combinations may need to be defined (IS 36.4.1, Table 18):

1.5D	(IS 36.4.1)
1.5D + 1.5L	(IS 36.4.1)
1.5D + 1.5(0.75PL)	(IS 31.5.2.3)
1.5D ± 1.5W 0.9D ± 1.5W 1.2D ± 1.2L ± 1.2W	(IS 36.4.1)
1.5D ± 1.5E 0.9D ± 1.5E 1.2D + 1.2L ± 1.2E	(IS 36.4.1)

In the preceding equations,

D = the sum of all dead load (DL) cases defined for the model.

L = The sum of all live load (LL) and reducible live load (RLL) cases defined for the model. Note that this includes roof live loads as well as floor live loads.

W = Any single wind load (WL) case defined for the model.

E = Any single earthquake load (E) case defined for the model.

These are also the default load combinations in the program whenever the Indian IS 456-2000 code is used. The user should use other appropriate design load combinations if roof live load is separately treated, or if other types of loads are present. The pattern loading is approximately, but conservatively, performed in the program automatically. Here PL is the approximate pattern load that is the live load multiplied by the Pattern Live Load Factor. The Pattern Live Load Factor can be specified in the Preferences. While calculating forces for the specified pattern load combination, the program adds forces for the dead load, assuming that the member geometry and continuity are unchanged from the model, and the forces for the pattern live load, assuming the beam is simply supported at the two ends. The Pattern Live Load Factor normally should be taken as 0.75 (IS 31.5.2.3). If the Pattern Live Load Factor is specified to be zero, the program does not generate pattern loading.

Live load reduction factors can be applied to the member forces of the load case that has a live load on a member-by-member basis to reduce the contribution of the live load to the factored loading. However such a live load must be specified as type Reducible Live Load.

For slender compression members, the code recommends the use of a second order frame analysis, also called a P-Δ analysis, which includes the effect of sway deflections on the axial loads and moments in a frame. For an adequate and rational analysis, realistic moment curvature or moment rotation relationships should be used to provide accurate values of deflections and forces. The analysis also should include the effect of foundation rotation and sustained loads. Because of the complexity in the general second order analysis of frames, the code provides an approximate design method that takes into account the "additional moments" due to lateral deflections in columns (IS 39.7). See also Clause 38.7 of SP-24 1983 (IS 1993) for details.

Hence, when using the Indian IS 456-2000 code, it is recommended that the user include the P-delta analysis. With this option, the program can capture the lateral drift effect, i.e., the global effect or P-Δ effect, very nicely. But the program does not capture the local effect (P-δ effect) to its entirety because most often the column members are not meshed. To capture the local effects in columns, the program uses the approximate formula for additional moments as specified in the code (IS 39.7.1). Two major parameters in calculating the additional moments are the effective length factors for major and minor axis bending. The effective length factors for columns are computed using a code-specified procedure (IS 25.2, Annex E, Wood 1974). If P-Δ analysis is not included, the program calculates effective length factors, *k*, assuming the frame is a sway frame (sway unrestrained) (IS Annex E, Figure 27). However, if the P-Δ analysis is included, the program assumes the member is prevented from further sway and assumes that the frame can be considered non-sway where $k < 1$ (IS Annex E, Figure 26). In that case, the program takes *k* equal to 1 conservatively. For piers, *k* is taken as 1.

1.3.1 Dead Load Component

The dead load component of the default design load combinations consists of the sum of all dead loads multiplied by the specified factor. Individual dead load cases are not considered separately in the default design load combinations.

1.3.2 Live Load Component

The live load component of the default design load combinations consists of the sum of all live loads, both reducible and nonreducible, multiplied by the specified factor. Individual live load cases are not considered separately in the default design load combinations.

1.3.3 Wind Load Component

The wind load component of the default design load combinations consists of the contribution from a single wind load case. Thus, if multiple wind load cases are defined in the program model, each case will contribute multiple design load combinations, as given in section 1.3.

1.3.4 Earthquake Load Component

The earthquake load component of the default design load combinations consists of the contribution from a single earthquake load case. Thus, if multiple earthquake load cases are defined in the program model, each case will contribute multiple design load combinations, as given in Section 1.3.

The earthquake load cases considered when creating the default design load combinations include all static load cases that are defined as earthquake loads and all response spectrum cases. Default design load combinations are not created for time history cases or for static nonlinear cases.

1.3.5 Combinations That Include a Response Spectrum

In the program all response spectrum cases are assumed to be earthquake load cases. Default design load combinations are created that include the response spectrum cases.

The output from a response spectrum is all positive. Any shear wall design load combination that includes a response spectrum load case is checked for all possible combinations of signs on the response spectrum values. Thus, when checking shear in a wall pier or a wall spandrel, the response spectrum contribution of shear to the design load combination is considered once as a positive shear and then a second time as a negative shear. Similarly, when checking moment in a wall spandrel, the response spectrum contribution of moment to the design load combination is considered once as a positive moment and then a second time as a negative moment. When checking the flexural behavior of a two-dimensional wall pier or spandrel, four possible combinations are considered for the contribution of response spectrum load to the design load combination. They are:

- +P and +M
- +P and −M
- −P and +M
- −P and −M

where P is the axial load in the pier and M is the moment in the pier. Similarly, eight possible combinations of P, M2 and M3 are considered for three-dimensional wall piers.

Note that based on the preceding, equations with negative signs with earthquake load cases in Section 1.3 are redundant for a load combination with a response spectrum. For this reason, the program creates only default design load combinations based on the equations with positive factors with earthquake load cases. Default design load combinations using equations with negative factors with earthquake load cases are not created for response spectra.

1.3.6 Combinations that Include Time History Results

The default shear wall design load combinations do not include any time history results. To include time history forces in a design load combination, define the load combination yourself.

When a design load combination includes time history results, the design is performed for each step of the time history.

If a single design load combination has more than one time history case in it, that design load combination is designed for the envelopes of the time histories.

1.3.7 Combinations That Include Static Nonlinear Results

The default shear wall design load combinations do not include any static nonlinear results. To include static nonlinear results in a design load combination, define the load combination yourself.

If a design load combination includes a single static nonlinear case and nothing else, the design is performed for each step of the static nonlinear analysis. Otherwise, the design is only performed for the last step of the static nonlinear analysis.

1.4 Design Strength

The design strengths 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 as follows:

Partial safety factor for steel, $\gamma_s = 1.15$, and (IS 35.4.2.1)

Partial safety factor for concrete, $\gamma_c = 1.15$. (IS 35.4.2.1)

These factors are already incorporated in the design equations and tables in the code. Although not recommended, the program allows the defaults to be overwritten. If the defaults are overwritten, the program uses the revised values consistently by modifying the code mandated equations in every relevant place.

1.5 Shear Wall Design Preferences

The shear wall design preferences are basic properties that apply to all wall pier and spandrel elements. Appendix B identifies shear wall design preferences for "Indian IS 456-2000." For other codes, design preferences are set in a similar fashion. Default values are provided for all shear wall design preference items. Thus, it is not required that preferences be specified. However, at least review the default values for the preference items to make sure they are acceptable. Refer to Appendix A for details.

1.6 Shear Wall Design Overwrites

The shear wall design overwrites are basic assignments that apply only to those piers or spandrels to which they are assigned. The overwrites for piers and spandrels are separate. Appendix C identifies the shear wall overwrites for "Indian IS 456-2000." For other codes design preferences are set in a similar fashion. Note that the available overwrites change depending on the pier section type (Uniform Reinforcing, General Reinforcing, or Simplified T and C). Default values are provided for all pier and spandrel overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure 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; that is, to the elements that are selected when the overwrites are changed. Refer to Appendix B for details.

1.7 Choice of Units

For shear wall design in this program, any set of consistent units can be used for input. Also, the system of units being used can be changed at any time. Typically, design codes are based on one specific set of units.

The IS 456:2000 and IS 13920:1993 codes are based on Newton-mm-Second units. For simplicity, all equations and descriptions presented in this manual correspond to **Newton-mm-Second** units unless otherwise noted.

The shear wall design preferences allow the user to specify special units for concentrated and distributed areas of reinforcement. These units are then used for reinforcement in the model, regardless of the current model units displayed in the drop-down list on the status bar (or within a specific form). The special units specified for concentrated and distributed areas of reinforcing can only be changed in the shear wall design preferences.

The choices available in the shear wall design preferences for the units associated with an area of concentrated reinforcing are in^2, cm^2, mm^2, and current units. The choices available for the units associated with an area per unit length of distributed reinforcing are in^2/ft, cm^2/m, mm^2/m, and current units.

The current units option uses whatever units are currently displayed in the drop-down list on the status bar (or within a specific form). If the current length units are cm, this option means concentrated areas of reinforcing are in cm^2 and distributed areas of reinforcing are in cm^2/cm. Note that when using the "current" option, areas of distributed reinforcing are specified in $Length^2/Length$ units, where Length is the currently active length unit. For example, if you are working in KN and cm units, the area of distributed reinforcing is specified in cm^2/cm. If you are in KN and mm, the area of distributed reinforcing is specified in mm^2/mm.

For details on general use of the program, refer to the CSi Analysis Reference Manual (CSI 2009).

Chapter 2
Pier Design

This chapter describes how the program designs each leg of concrete wall piers for shear using the “Indian IS 456-2000” code. It should be noted that in the program you cannot specify shear reinforcing and then have the program check it. The program only designs the pier for shear and reports how much shear reinforcing is required. The shear design is performed at stations at the top and bottom of the pier.

This chapter also describes how the program designs and checks concrete wall piers for flexural and axial loads using the Indian IS 456-2000 code. First we describe how the program *designs* piers that are specified by a Simplified section. Next we describe how the program *checks* piers that are specified by a Section Designer section. Then we describe how the program *designs* piers that are specified by a Section Designer section.

The program designs/checks only seismic or non-seismic reinforced concrete wall pier sections. Currently, other concrete wall pier design is not in the scope of the program.

2.1 Wall Pier Flexural Design

ETABS can design the pier sections that are defined as "General Reinforcing Pier" sections or "Uniform Reinforcing Pier" sections when reinforcing layout is given and "Design Reinforcement" is requested. The program can check those two types of sections when reinforcements are given and "Check Reinforcement" is requested. The program can design the pier sections that are defined as "Simplified T and C Pier" sections. In all cases, P-M-M interaction is used for the overall section. For Simplified T and C sections, approximations are used to perform simple design. For the design problems, design reinforcement is reported. For the check problems, the P-M-M interaction ratio is reported.

For flexural design/check of piers, the factored forces axial force P_u, and the bending moments M_{u2} and M_{u3} for a particular design load combination at a particular design section (top or bottom) are obtained by factoring the associated forces and moments with the corresponding design load combination factors.

The piers are designed for minimum eccentricity moment, $P_u e_{\min}$, where

$$e_{\min} = \quad e_{\min} = \begin{cases} L_{33}/500 + L_w/30 \\ L_{22}/500 + t_w/30 \end{cases} \qquad \text{(IS 25.4)}$$

where

L_w = Overall horizontal length of the wall,

t_w = The thickness of the wall,

L_{33} = Unbraced length of the wall in the major direction,

= Story height,

L_{22} = Unbraced length of the wall in the minor direction,

= Story height.

The minimum eccentricity is applied in only one direction at a time (IS 25.4).

The program computes the slenderness ratios as L_{33}/L_w and L_{22}/t_w where L_{33} and L_{22} are effective lengths of column about local axes 3 and 2. If the slenderness ratio is greater than 12, the pier is considered as slender in that plane (IS 25.1.2). Effectively, the pier may be slender in one or both planes.

If a column is slender in a plane, additional slenderness moments M_{a2} and M_{a3} are computed using the following formula:

$$M_{a3} = k\frac{P_u D}{2000}\left\{\frac{L_{33}}{L_w}\right\}^2 \qquad \text{(IS 39.7.1, 39.7.1.1)}$$

$$M_{a2} = k\frac{P_u b}{2000}\left\{\frac{L_{22}}{t_w}\right\}^2 \qquad \text{(IS 39.7.1, 39.7.1.1)}$$

where

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1 \qquad \text{(IS 39.7.1.1)}$$

P_{uz} = $0.45\, f_{ck}A_c + 0.75\, f_y A_{sc}$ (IS 39.6)

M_{a2}, M_{a3} = Additional moment t accounts for the column slenderness about the column local 2 and 3 axes, respectively

P_u = Factored axial force in the column for a particular load combination

P_{uz} = Theoretical axial capacity of the column

P_b = Axial load corresponding to the condition of maximum compressive strain of 0.0035 in concrete and tensile strain of 0.002 in the outermost layer of tensile steel

L_{33}, L_{22} = Effective length of the column about the local 3 and 2 axes, respectively

$L_{22} = k_{22}L_{22}$

$L_{33} = k_{33}L_{33}$

D, b = Lateral column dimension perpendicular to the local 3 and 2 axes, respectively

k = Reduction factor for reducing additional moments

Other variables are described in Section 1.1 *Notation* in Chapter 1.

When designing the pier, A_{sc} is not known in advance, and thus P_{uz} and P_b are not known. In such cases, k is conservatively taken as 1.

$$k = 1$$

The program assumes the effective length factors $k_{33} = 1$ and $k_{22} = 1$.

The use of code-specified additional moment (IS 39.7) is an approximate procedure (IS SP-24, 1993, Clause 38.7). It is recommended that the user include P-Delta analysis. With this option, the program can capture the lateral drift effect (i.e., global effect or P-Δ effect), but the program does not capture local effect (i.e., P-δ effect) to its entirety through analysis. To capture the local effect correctly, the program uses the approximate design formula for additional moments with the assumption that $k = 1$.

It should be noted that the minimum eccentricity is enforced and additional moment is employed for a planar wall, as stated previously. However for multi-legged 3D piers, the minimum eccentricity is enforced based on equivalent thickness. Similar slenderness effects are calculated based on equivalent thickness.

2.1.1 Checking a General or Uniform Reinforcing Pier Section

When you specify that a General Reinforcing or Uniform Reinforcing pier section is to be checked, the program creates an interaction surface for that pier and uses that interaction surface to determine the critical flexural demand/capacity ratio for the pier. This section describes how the program generates the interaction surface for the pier and how it determines the demand/capacity ratio for a given design load combination.

Note: In this program, the interaction surface is defined by a series of P-M-M interaction curves that are equally spaced around a 360-degree circle.

2.1.1.1 Interaction Surface

In this program, a three-dimensional interaction surface is defined with reference to the P, M_2 and M_3 axes. The surface is developed using a series of interaction curves that are created by rotating the direction of the pier neutral axis in equally spaced increments around a 360-degree circle. For example, if 24 P-M-M curves are specified (the default), there is one curve every 15 degrees (360°/24 curves = 15°). Figure 2-1 illustrates the assumed orientation of the pier neutral axis and the associated sides of the neutral axis where the section is in tension (designated T in the figure) or compression (designated C in the figure) for various angles.

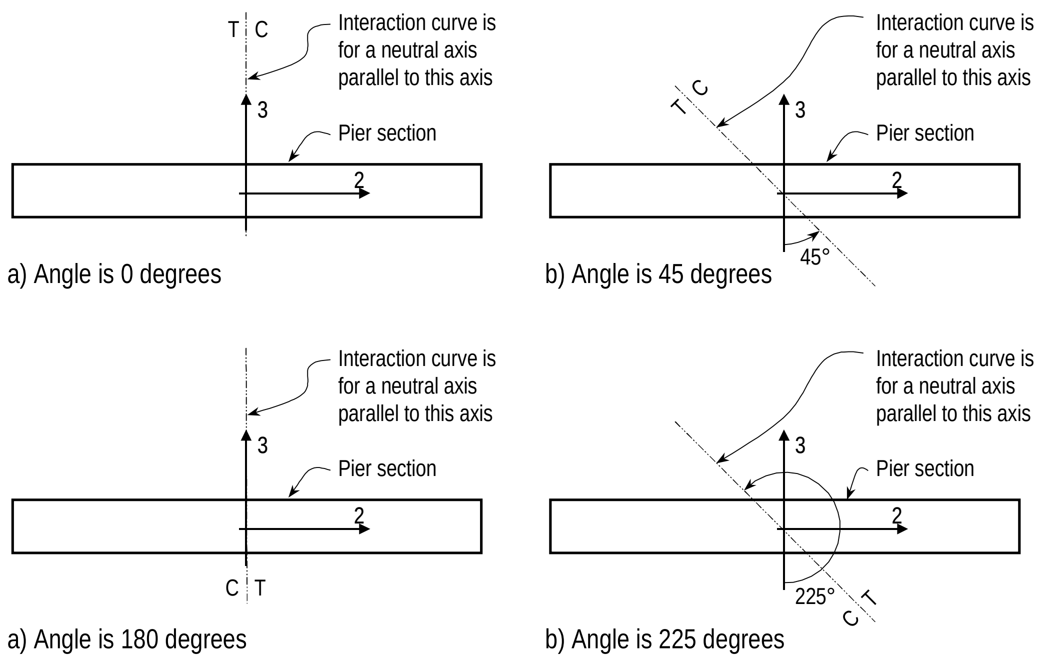

Figure 2-1: Orientation of the Pier Neutral Axis for Various Angles

Note that the orientation of the neutral axis is the same for an angle of θ and θ +180°. Only the side of the neutral axis where the section is in tension or compression changes. We recommend that you use 24 interaction curves (or more) to define a three-dimensional interaction surface.

Each P-M-M interaction curve that makes up the interaction surface is numerically described by a series of discrete points connected by straight lines. The coordinates of these points are determined by rotating a plane of linear strain about the neutral axis on the section of the pier. Details of this process are described in the next section, 2.1.1.2 *Generation of the Biaxial Interaction Surface*.

By default, 11 points are used to define a P-M-M interaction curve. This number can be changed in the preferences; any odd number of points greater than or equal to 11 can be specified, to be used in creating the interaction curve. If an even number is specified for this item in the preferences, the program will increment up to the next higher odd number.

Note that when creating an interaction surface for a two-dimensional wall pier, the program considers only two interaction curves—the 0° curve and the 180° curve—regardless of the number of curves specified in the preferences. Furthermore, only moments about the M3 axis are considered for two-dimensional walls.

2.1.1.2 Generation of the Biaxial Interaction Surface

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 2-2. The coordinates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column, as shown in Figure 2-3.

The linear strain diagram limits the maximum concrete strain, ε_c, at the extremity of the section, as given by the following equations:

(a) When there is any tensile strain in the section, the maximum strain in concrete at the outermost compression fiber is taken as 0.0035 (IS 38.1(b)).

$$\varepsilon_c = 0.0035, \quad \text{when tensile strain is present} \qquad \text{(IS 38.1(b))}$$

(b) When the section is uniformly compressed, the maximum compression strain in concrete is taken as 0.002 (IS 39.1(a)).

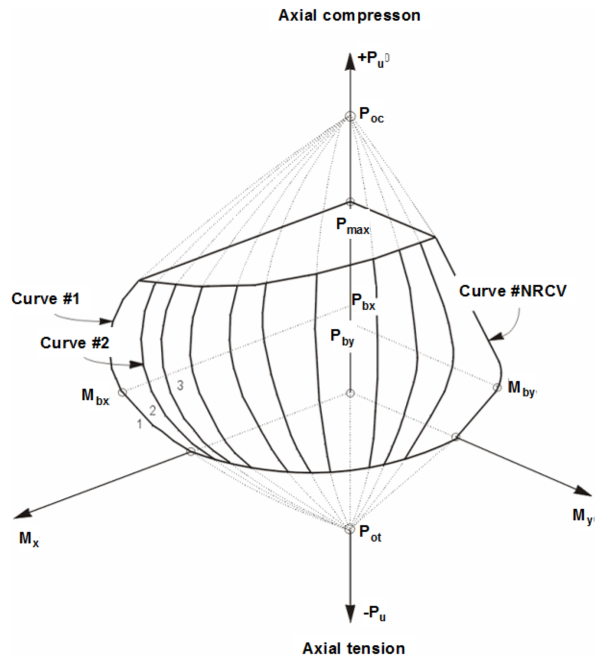

Figure 2-2 A typical column interaction surface

$\varepsilon_c = 0.002$, when the section in uniformly compressed (IS 39.1(a))

(c) When the entire section is under non-uniform compression, the maximum compressive strain at the highly compressed extreme fiber is taken as 0.0035 minus 0.75 times the strain at the least compressed extreme fiber (IS 39.1(b)).

$\varepsilon_{c,\max} = 0.0035 - 0.75\ \varepsilon_{c,\min}$, when the section is non-uniformly compressed (IS 39.1(b))

The formulation is based consistently on the basic principles of limit state of collapse under compression and bending (IS 38, 39).

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, $\varepsilon_s E_s$, and is limited to the design strength of the steel, f_y / γ_s (IS 38.1(e)). 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 with respect to distributing the area of steel over the cross-section of the column, as shown in Figure 2-3.

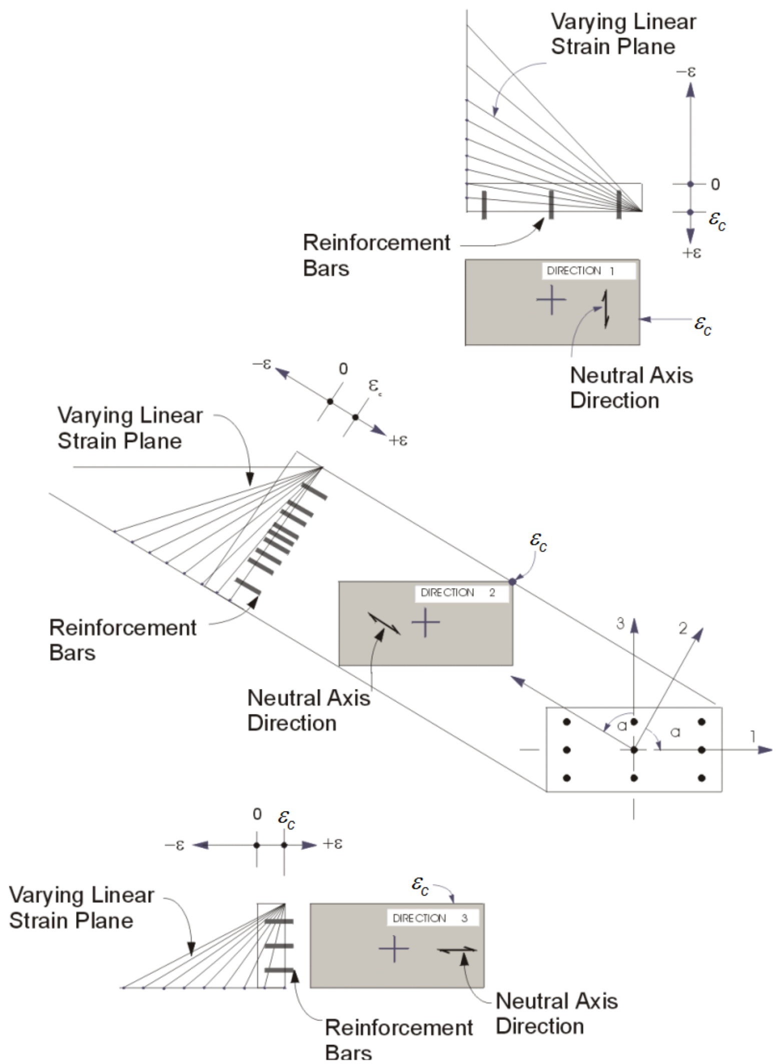

Figure 2-3 Idealized strain distribution for generation of interaction surface

The concrete compression stress block is assumed to be parabolic, with a stress value of $0.67\, f_{ck}/\gamma_m$ (IS 38.1.c). See Figure 2-4. The interaction algorithm provides corrections to account for the concrete area that is displaced by the reinforcement in the compression zone.

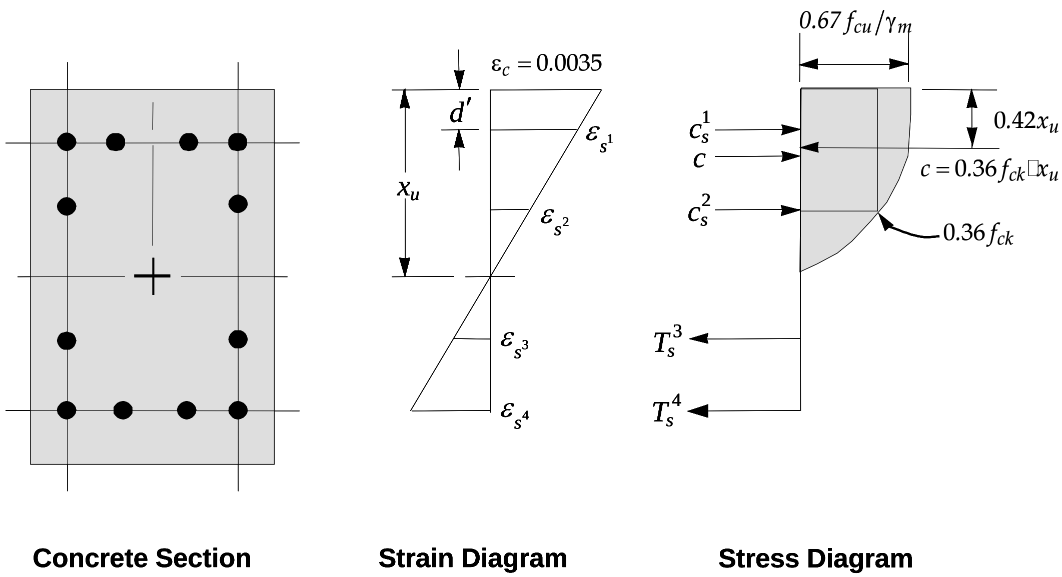

Figure 2-4 Idealization of stress and strain distribution in a column section

The equivalent concrete compression stress block is assumed to be rectangular, with a stress value of 0.36 f_{ck} [IS 38.1(c), Figure 22], as shown in Figure 2-4. The depth of the equivalent rectangular block, *a*, is taken as:

$$a = \beta_1 \cdot x_u \qquad \text{(IS 38.1(c), Figure 22)}$$

where *c* is the depth of the stress block in compression strain, and

$$\beta_1 = 2 \times 0.42 = 0.84. \qquad \text{(IS 38.1(c), Figure 22)}$$

The maximum compressive axial strength is limited to P_u, where

$$P_u = [0.4 f_{ck} A_c + 0.67\, f_y A_{sc}]. \qquad \text{(IS 39.3)}$$

However, the preceding limit is not normally reached unless the section is heavily reinforced.

Note: The number of points to be used in creating interaction diagrams can be specified in the shear wall preferences and overwrites.

As previously mentioned, by default, 11 points are used to define a single interaction curve. When creating a single interaction curve, the program includes the points at P_b, P_{oc} and P_{ot} on the interaction curve. Half of the remaining number of specified points on the interaction curve occur between P_b and P_{oc} at approximately equal spacing along the P_u axis. The other half of the remaining number of specified points on the interaction curve occur between P_b and P_{ot} at approximately equal spacing along the P_u axis.

2.1.1.3 Wall Pier Demand/Capacity Ratio

Refer to Figure 2-5, which shows a typical two-dimensional wall pier interaction diagram. The forces obtained from a given design load combination are P_u and M_{u3}. Point *L*, defined by (P_u, M_{u3}), is placed on the interaction diagram, as shown in the figure. If the point lies within the interaction curve, the wall pier capacity is adequate. If the point lies outside of the interaction curve, the wall pier is overstressed.

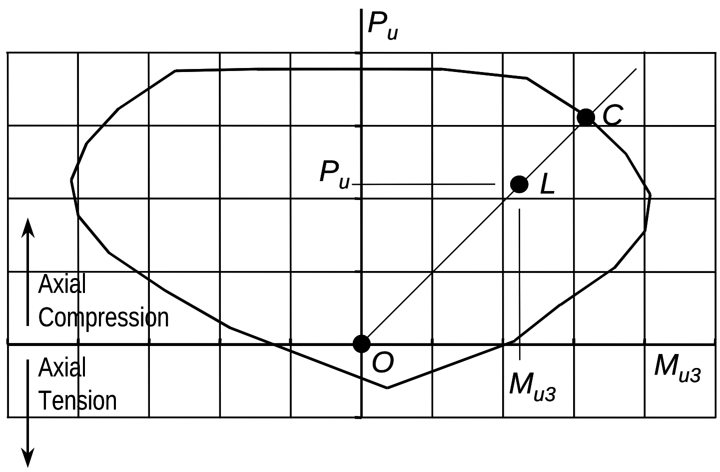

Figure 2-5: Two-Dimensional Wall Pier Demand/Capacity Ratio

As a measure of the stress condition in the wall pier, the program calculates a stress ratio. The 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* (extended outward if needed) intersects the interaction curve. The demand/capacity ratio, D/C, is given by D/C = *OL* / *OC* where *OL* is the "distance" from point *O* (the origin) to point *L* and *OC* is the "distance" from point *O* to point *C*. Note the following about the D/C ratio:

- If *OL* = *OC* (or D/C = 1), the point (P_u, M_{u3}) lies on the interaction curve and the wall pier is stressed to capacity.

- If $OL < OC$ (or D/C < 1), the point (P_u, M_{u3}) lies within the interaction curve and the wall pier capacity is adequate.

- If $OL > OC$ (or D/C > 1), the point (P_u, $M3_u$) lies outside of the interaction curve and the wall pier is overstressed.

The wall pier D/C ratio is a factor that gives an indication of the stress condition of the wall with respect to the capacity of the wall.

The D/C ratio for a three-dimensional wall pier is determined in a similar manner to that described here for two-dimensional piers.

The maximum of all the D/C ratios calculated for each design load combination is reported for each check station of the pier along with the controlling (P_u, M_{u2}, M_{u3}) set and the associated load combination name.

2.1.1.4 Designing a General or Uniform Reinforcing Pier Section

When a General Reinforcing pier section is specified to be designed, the program creates a series of interaction surfaces for the pier based on the following items:

- The size of the pier as specified in Section Designer.

- The location of the reinforcing specified in Section Designer.

- The size of each reinforcing bar specified in Section Designer *relative* to the size of the other bars.

The interaction surfaces are developed for eight different ratios of reinforcing-steel-area-to-pier-area. The pier area is held constant and the rebar area is modified to obtain these different ratios; however, the *relative* size (area) of each rebar compared to the other bars is always kept constant.

The smallest of the eight reinforcing ratios used is that specified in the shear wall design preferences as Section Design IP-Min. Similarly, the largest of the eight reinforcing ratios used is that specified in the shear wall design preferences as Section Design IP-Max.

The eight reinforcing ratios used are the maximum and the minimum ratios plus six more ratios. The spacing between the reinforcing ratios is calculated as an increasing arithmetic series in which the space between the first two ratios is equal to one-third of the space between the last two ratios. Table 1 illustrates the spacing, both in general terms and for a specific example, when the minimum reinforcing ratio, IP-Min, is 0.0025 and the maximum, IP-Max, is 0.02.

After the eight reinforcing ratios have been determined, the program develops interaction surfaces for all eight of the ratios using the process described earlier in this section.

Next, for a given design load combination, the program generates a D/C ratio associated with each of the eight interaction surfaces. The program then uses linear interpolation between the eight interaction surfaces to determine the reinforcing ratio that gives a D/C ratio of 1 (actually the program uses the Utilization Factor Limits instead of 1; the Utilization Factor Limit is 0.95 by default, but it can be overwritten by the user in the preferences). This process is repeated for all design load combinations and the largest required reinforcing ratio is reported.

Design of a Uniform Reinforcing pier section is similar to that described herein for the General Reinforcing section.

Table 2-1 The Eight Reinforcing Ratios Used by the Program

Curve	Ratio, *e*	Example, *e*
1	IPmin	0.0025
2	$\text{IPmin} + \frac{\text{IPmax} - \text{IPmin}}{14}$	0.0038
3	$\text{IPmin} + \frac{7}{3}\left(\frac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0054
4	$\text{IPmin} + 4\left(\frac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0075
5	$\text{IPmin} + 6\left(\frac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0100
6	$\text{IPmin} + \frac{25}{3}\left(\frac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0129
7	$\text{IPmin} + 11\left(\frac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0163
8	IPmax	0.0200

2.1.2 Designing a Simplified T and C Pier Section

This section describes how the program designs a pier that is assigned a simplified section. The geometry associated with the simplified section is illustrated in Figure 2-6. The pier geometry is defined by a length, thickness and size of the edge members at each end of the pier (if any).

The central idea behind simplified T and C pier section design is that the axial forces and moments are carried by only the wall pier end zones known as "Boundary Elements," located at wall edges, which provide the moment of resistance by developing a tension-compression couple, under combined action of axial force and moment. The central part of the wall is ignored in strength computation and is provided with minimum throwaway reinforcement. This method results in a conservative design of the wall pier for axial force and flexure.

A simplified T and C pier section is always planar (not three-dimensional). The dimensions shown in the figure include the following:

- The length of the wall pier is designated L_w. This is the horizontal length of the wall pier in plan.
- The thickness of the wall pier is designated t_p. The thickness specified for left and right edge members ($DB2_{left}$ and $DB2_{right}$) may be different from this wall thickness.
- DB1 represents the horizontal length of the pier edge member. DB1 can be different at the left and right sides of the pier. However in symmetrical walls it is preferred to have them of same length.
- DB2 represents the horizontal width (or thickness) of the pier edge member. DB2 can be different at the left and right sides of the pier.

The dimensions illustrated are specified in the shear wall overwrites (Appendix C), and can be specified differently at the top and bottom of the wall pier.

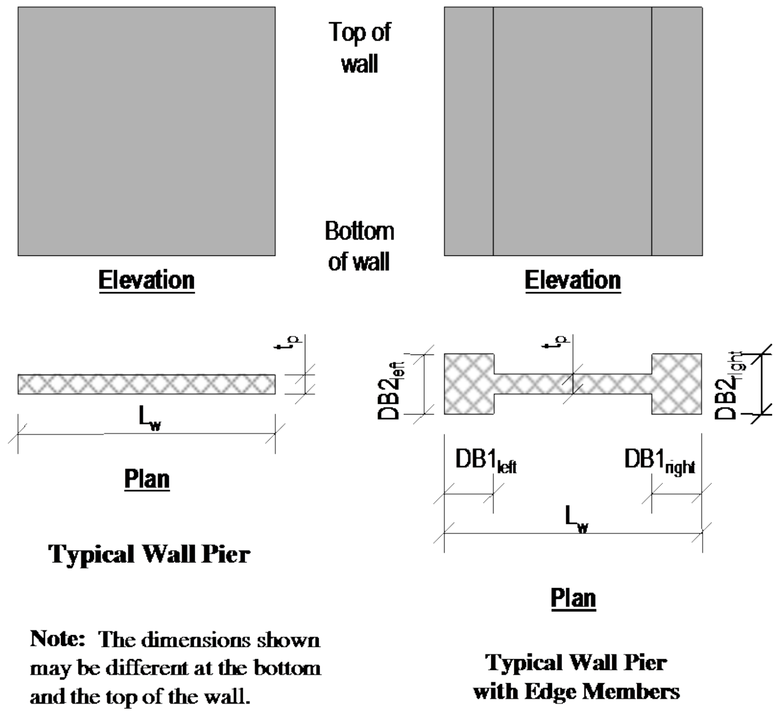

Figure 2-6: Typical Wall Pier Dimensions Used for Simplified Design

If no specific edge member dimensions have been specified by the user, the program assumes that the edge member is the same width as the wall, and the program determines the required length of the edge member. In all cases, whether the edge member size is user-specified or program-determined, the program reports the required area of reinforcing steel at the center of the edge member. This section describes how the program-determined length of the edge member is determined and how the program calculates the required reinforcing at the center of the edge member.

Three design conditions are possible for a simplified T and C wall pier. These conditions, illustrated in Figure 2-7, are as follows:

- The wall pier has program-determined (variable length and fixed width) edge members on each end.
- The wall pier has user-defined (fixed length and width) edge members on each end.

- The wall pier has a program-determined (variable length and fixed width) edge member on one end and a user-defined (fixed length and width) edge member on the other end.

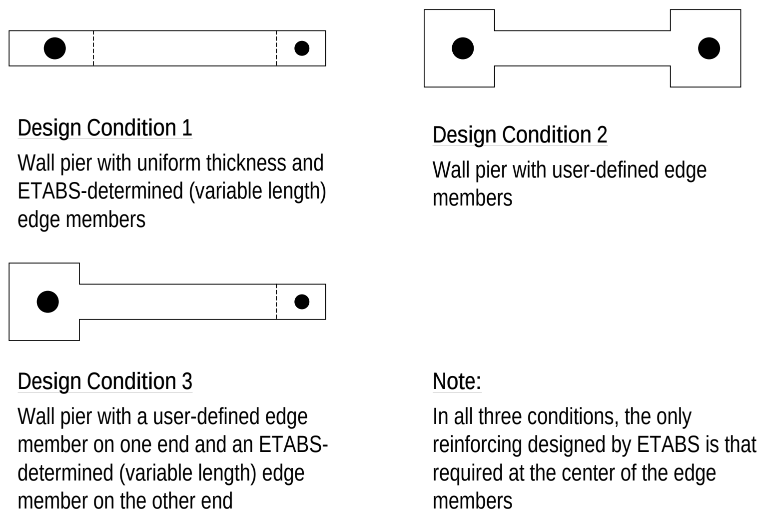

Figure 2-7: Design Conditions for Simplified Wall Piers

2.1.2.1 Design Condition 1

Design condition 1 applies to a wall pier with uniform design thickness and program-determined edge member length. For this design condition, the design algorithm focuses on determining the required size (length) of the edge members, while limiting the compression and tension reinforcing located at the center of the edge members to user-specified maximum ratios. The maximum ratios are specified in the shear wall design preferences and the pier design overwrites as Edge Design PC-Max and Edge Design PT-Max.

Consider the wall pier shown in Figure 2-8. For a given design section, for example the top of the wall pier, the wall pier for a given design load combination is designed for a factored axial force $P_{u\text{-top}}$ and a factored moment $M_{u\text{-top}}$.

The program initiates the design procedure by assuming an edge member at the left end of the wall of thickness t_p and width $B_{1\text{-left}}$, and an edge member at the right end of the wall of thickness t_p and width $B_{1\text{-right}}$. Initially $B_{1\text{-left}} = B_{1\text{-right}} = t_p$.

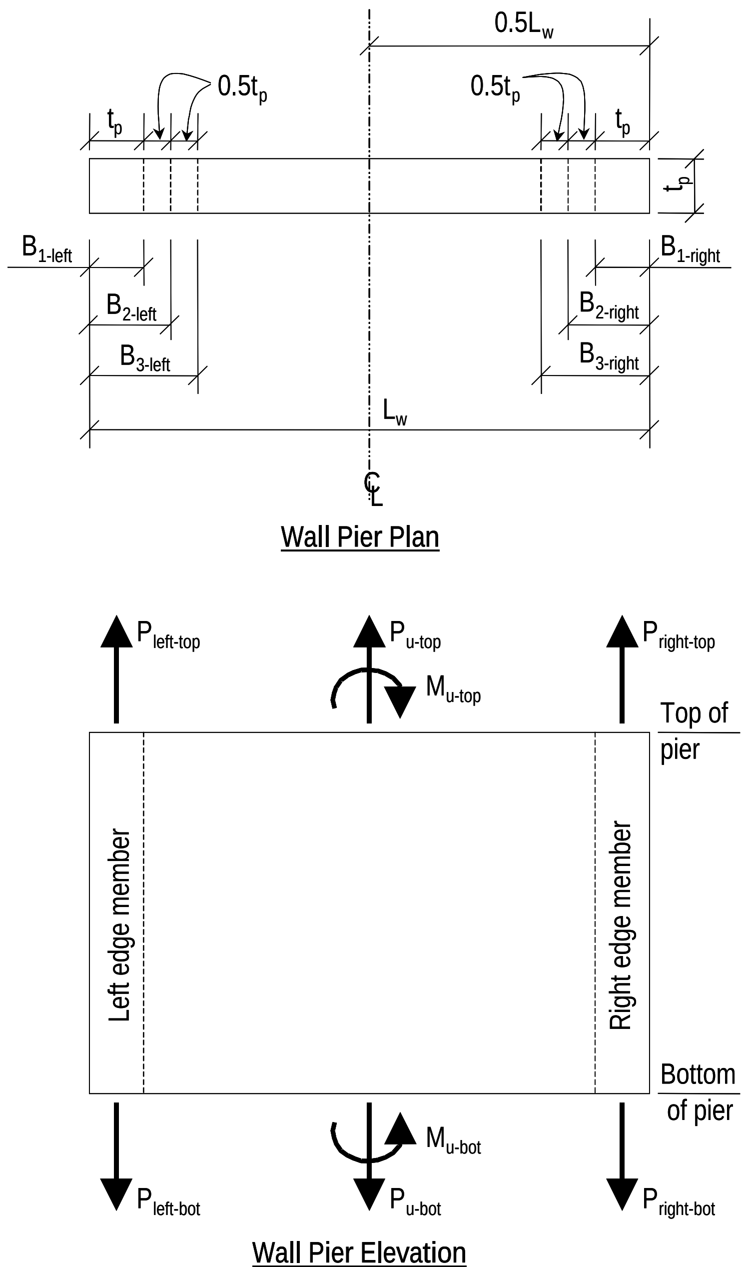

Figure 2-8: Wall Pier for Design Condition 1

The moment and axial force are converted to an equivalent force set $P_{\text{left-top}}$ and $P_{\text{right-top}}$ using the equations that follow. (Similar equations apply at the bottom of the pier.)

$$P_{\text{left-top}} = \frac{P_{u\text{-top}}}{2} + \frac{M_{u\text{-top}}}{L_w - 0.5B_{1\text{-left}} - 0.5B_{1\text{-right}}}$$

$$P_{\text{right-top}} = \frac{P_{u\text{-top}}}{2} - \frac{M_{u\text{-top}}}{L_w - 0.5B_{1\text{-left}} - 0.5B_{1\text{-right}}}$$

For any given loading combination, the net values for $P_{\text{left-top}}$ and $P_{\text{right-top}}$ could be tension or compression.

Note that for dynamic loads, $P_{\text{left-top}}$ and $P_{\text{right-top}}$ are obtained at the modal level and the modal combinations are made, before combining with other loads. Also for design loading combinations involving SRSS, the $P_{\text{left-top}}$ and $P_{\text{right-top}}$ forces are obtained first for each load case before the combinations are made.

If any value of $P_{\text{left-top}}$ or $P_{\text{right-top}}$ is tension, the area of steel required for tension, A_{st}, is calculated as:

$$A_{st} = \frac{P}{f_y / \gamma_s}. \qquad \text{(IS 38)}$$

If any value of $P_{\text{left-top}}$ or $P_{\text{right-top}}$ is compression, for section adequacy, the area of steel required for compression, A_{sc}, must satisfy the following relationship.

$$P = P_{\text{max,factor}}[0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}] \qquad \text{(IS 39.3)}$$

where P is either $P_{\text{left-top}}$ or $P_{\text{right-top}}$, $A_g = t_p B_1$ and the $P_{\text{max,factor}}$ is defined in the shear wall design preferences (the default is 0.80). In general, we recommend that you use the default value.

Area of compression rebar is calculated as follows:

$$A_{sc} = \frac{P / \left(P_{\text{max,factor}}\right) - 0.4 f_{ck} A_g}{0.67 f_y - 0.4 f_{ck}}$$

If A_{sc} calculates as negative, no compression reinforcing is needed.

The maximum tensile reinforcing to be packed within the t_p times B_1 concrete edge member is limited by:

$$A_{st\text{-max}} = PT_{\max} t_p B_1.$$

Similarly, the compression reinforcing is limited by:

$$A_{sc\text{-max}} = PC_{\max} t_p B_1.$$

If A_{st} is less than or equal to $A_{st\text{-max}}$ and A_{sc} is less than or equal to $A_{sc\text{-max}}$, the program will proceed to check the next loading combination; otherwise the program will increment the appropriate B_1 dimension (left, right or both, depending on which edge member is inadequate) by one-half of the wall thickness to B_2 (i.e., $0.5t_p$) and calculate new values for $P_{\text{left-top}}$ and $P_{\text{right-top}}$ resulting in new values of A_{st} and A_{sc}. This iterative procedure continues until A_{st} and A_{sc} are within the allowed steel ratios for all design load combinations.

If the value of the width of the edge member B increments to where it reaches a value larger than or equal to $L_p/2$, the iteration is terminated and a failure condition is reported.

This design algorithm is an approximate, but convenient, algorithm. Wall piers that are declared overstressed using this algorithm could be found to be adequate if the reinforcing steel is user-specified and the wall pier is accurately evaluated using interaction diagrams.

2.1.2.2 Design Condition 2

Design condition 2 applies to a wall pier with user-specified edge members at each end of the pier. The size of the edge members is assumed to be fixed; that is, the program does not modify them. For this design condition, the design algorithm determines the area of steel required in the center edge members and checks if that area gives reinforcing ratios less than the user-specified maximum ratios. The design algorithm used is the same as described for condition 1; however, no iteration is required.

2.1.2.3 Design Condition 3

Design condition 3 applies to a wall pier with a user-specified (fixed dimension) edge member at one end of the pier and a variable length (program-determined) edge member at the other end. The width of the variable length edge member is equal to the width of the wall.

The design is similar to that which has previously been described for design conditions 1 and 2. The size of the user-specified edge member is not changed. Iteration occurs only on the size of the variable length edge member.

2.2 Wall Pier Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. The wall pier boundary element requirement is checked when the design load combination involves only seismic load. Each planar leg of multi-legged wall piers is checked. Both edges of each planar leg are checked separately.

The basis for boundary element calculation is Section 9.4 of IS 13920. The boundary element calculation is similar to the design of a pier leg assuming a Simplified C-T wall pier, as described earlier in this chapter.

The program reports the horizontal extents of the boundary, suggested longitudinal rebar area, and the required transverse reinforcement.

2.2.1 Details of Check for Boundary Element Requirements

The following procedure is used for checking the required boundary elements, and if needed, for designing the boundary elements:

- Calculate the factored forces P_u, V_u, and M_u for the pier planar leg section.
- Retrieve the geometric properties of the leg: the height of the wall segment (story height) h_w; length of the wall pier planar leg, L_w; the thickness of the pier leg t_p. Refer to Figure 2-6 earlier in this chapter for an illustration of the dimensions.
- Retrieve the material properties of the pier leg, f_{ck} and f_y.
- Calculate the stress at the two extreme ends. Assume linearized elastic stress distribution and gross section properties. Ignore rebar area A_s and M_{uz}.

$$\sigma = \frac{P_u}{A} \pm \frac{M_u \cdot \left(L_w/2\right)}{t_w \cdot L_w^{\,3}/12} \qquad \text{(IS 13920 9.4.1)}$$

- If any of the stresses at the two ends are compressive and exceed the following limit

  $\sigma > 0.2f_{ck}$, (IS 13920 9.4.1)

  boundary elements are required (IS 13920 9.4.1)

- If boundary elements are needed at any edge of the pier leg, calculate the length of the required boundary element, the required longitudinal rebar, and the required transverse rebar from the following procedure.

  If boundary elements are required for any edge of a pier leg, as determined by the previous steps, the determination of the horizontal length of the boundary and the required longitudinal rebar is similar to the procedure that has been described in *Section 2.1.2 Designing a Simplified T and C Pier Section* (IS 13920 9.4.2). Refer to that section for the details. However the following points should be noted.

  - The boundary width determination is based on the maximum rebar density as described by PT_{max} and PT_{min}.

  - The boundary width determination is an iterative procedure.

  - The boundary element axial compressive capacity is based on the "short column," for which the capacity is given in IS Section 39.3.

  - The P_{max} factor is taken as 1 for boundary element calculation.

  - The moment of resistance provided by the distributed vertical reinforcement across the wall section is ignored.

2.2.2 Transverse Reinforcement for Boundary Elements

Where special boundary elements are required by IS:13920 Section 9.4, the program computes and reports the total cross-sectional area of rectangular hoop reinforcement as follows:

$$\frac{A_{sh}}{s} = 0.18h\frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]. \qquad \text{(IS13920 7.4.8)}$$

2.3 Wall Pier Shear Design

The wall pier shear reinforcing is designed for each of the design load combinations. The required area of reinforcing for in-plane horizontal shear is calculated for each planar leg of the pier and only at the ends of the pier (i.e., at the story level).

In this program, wall pier legs are designed for major (in-plane) direction shear force only. Effects caused by minor direction shear (out-of-plane) force that may exist in the pier planar legs must be investigated by the user independent of the program.

The following steps are involved in designing the shear reinforcing for a particular wall pier leg section for a particular design loading combination.

- Determine the factored forces P_u and V_u that are acting on the wall pier section. Note that P_u is required for the calculation of τ_{cd}.

- Determine the shear stress, τ_{cd}, that can be carried by the concrete alone.

- Determine the required shear reinforcing, A_{sv}/s_v, to carry the balance of the shear force.

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

The assumptions in designing the shear reinforcement are as follows:

- The pier planar leg section can be considered to be prismatic. The program does not adjust the shear force for non-prismatic pier legs (IS 40.5.1).

- The effect on the concrete shear capacity of any concentrated or distributed load in the span of the pier leg between two diaphragms is ignored. Also, the effect of the direct support on the piers provided by the diaphragms is ignored.

- All shear reinforcement is assumed to be perpendicular to the longitudinal reinforcement.
- The effect of axial force is considered (IS 40.2.2).

2.3.1 Determine Factored Shear Force

In the design of the shear reinforcement for pier planar legs, the shear forces and axial forces for a particular design load combination at a particular beam section are obtained by factoring the associated shear force and axial force with the corresponding design load combination factors.

The section is essentially considered to be prismatic. If the section is non-prismatic and the depth varies with length, the program does not adjust the design shear force as recommended by the code (IS 40.1.1). In that case, the user is expected to check the shear independently of the program.

2.3.2 Determine the Concrete Shear Capacity

Given the design force V_u and P_u, the design shear stress that can be carried by concrete alone, τ_{cd}, and the maximum limit of nominal shear stress in the wall, $\tau_{c,\max}$, are calculated differently when the design load combination does not involve any seismic loading and when it does.

When the design load combination does not involve any seismic load, the shear force carried by the concrete, V_c, by planar legs of wall piers is calculated as follows (IS 32.4.2, 32.4.3):

$$V_c = \tau_{cd} A_{cv}, \text{ where} \qquad \text{(IS 13920 9.2.2; IS 40.2)}$$

$$\tau_{cd} = \begin{cases} K_1\sqrt{f_{ck}}\,(3.0 - H_w / L_w) & \text{for } H_w/L_w \le 1 \\ K_2\sqrt{f_{ck}}\,\dfrac{(H_w/L_w + 1)}{(H_w/L_w - 1)} \ge K_3\sqrt{f_{ck}} & \text{for } H_w/L_w > 1 \end{cases} \qquad \text{(IS 32.4.3)}$$

τ_{cd} = Design shear strength of concrete in walls

A_{cv} = Effective area of wall pier section under shear, taken as $b_w d_w$ (IS 32.4.2)

b_w = Thickness of wall pier planar leg,

d_w = Effective depth of wall pier taken as $0.8L_w$, (IS 32.4.2)

L_w = Overall horizontal length of the wall, (IS 32.4.2)

$K_1 = 0.2$ (IS 32.4.3)

$K_2 = 0.045$ (IS 32.4.3)

$K_3 = 0.15$ (IS 32.4.3)

For non-seismic load combinations, the absolute maximum limit on the nominal shear stress, $\tau_{c,\max}$, is calculated as follows (IS 34.4.2.1):

$$\tau_{c,\max} = 0.17\, f_{ck}. \qquad \text{(IS 34.4.2.1)}$$

When the design load combination involves any seismic load, the shear force carried by the concrete, V_c, by planar legs of the wall piers is calculated as follows (IS 13920 9.2.2; IS 40.2):

$$V_c = \tau_{cd} A_{cv} \qquad \text{(IS 13920 9.2.2; IS 40.2)}$$

where :

A_{cv} = Effective area of wall pier section under shear, taken as $t_w d_w$

b_w = Thickness of wall pier planar leg, (IS 13920 9.2.1)

d_w = Effective depth of wall pier taken as $0.8L_w$, (IS 13920 9.2.1; IS 32.4.2)

L_w = Overall horizontal length of the wall (IS 32.4.2)

$\tau_{cd} = k\delta\tau_c$, (IS 40.2 and IS 13920 9.2.2)

$$\delta = \begin{cases} 1 + 3\dfrac{P_u}{A_g f_{ck}} \le 1.5 & \text{if} \quad Pu > 0, \quad \text{Under Compression} \\ 1 & \text{if} \quad Pu \le 0, \quad \text{Under Tension} \end{cases} \qquad \text{(IS 40.2.2)}$$

k is always taken as 1, and (IS 40.2.1.1)

τ_c is the basic design shear strength for concrete, which is given by IS Table 19. It should be mentioned that the value of γ_c has already been incorporated in IS Table 19 (see Note in IS 36.4.2.1). The following limitations are enforced in the determination of the design shear strength as is done in the Table.

$$0.15 \le \frac{100 A_s}{bd} \le 3$$

$f_{ck} \le 40$ N/mm^2 (for calculation purposes only)

The determination of τ_c from Table 19 of the code requires two parameters: $100\frac{A_s}{bd}$ and f_{ck}. If $100\frac{A_s}{bd}$ become more than 3.0, τ_c is calculated based on $100\frac{A_s}{bd}$ = 3.0. If $100\frac{A_s}{bd}$ becomes less than 0.15, τ_c is calculated based on $100\frac{A_s}{bd}$ = 0.15. Similarly, if f_{ck} is larger than 40 N/mm^2, τ_c is calculated based on f_{ck} = 40 N/mm^2. However, if f_{ck} is less than 15 N/mm^2, τ_c is reduced by a factor of $\left(\frac{f_{ck}}{0.15}\right)^{1/4}$. If γ_c is chosen to be different from 1.5, τ_c is adjusted with a factor of $(1.5/\gamma_c)$. For seismic load combinations, the absolute maximum limit on nominal shear stress, $\tau_{c,\max}$ is calculated in accordance with IS Table 20, which is reproduced in the table that follows (IS 40.2.3, Table 20):

Maximum Shear Stress, $\tau_{c,\max}$ (N/mm^2)						
Concrete Grade	M15	M20	M25	M30	M35	M40
$\tau_{c,\max}$ (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4.0

If f_{ck} is between the limits, linear interpolation is used.

$$\tau_{c,\max} = \begin{cases} 2.5 & \text{if} \quad f_{ck} < 15 \\ 2.5 + 0.3\dfrac{f_{ck} - 15}{5} & \text{if} \quad 15 \leq f_{ck} < 20 \\ 2.8 + 0.3\dfrac{f_{ck} - 20}{5} & \text{if} \quad 20 \leq f_{ck} \geq 25 \\ 3.1 + 0.4\dfrac{f_{ck} - 25}{5} & \text{if} \quad 25 \leq f_{ck} < 30 \\ 3.5 + 0.2\dfrac{f_{ck} - 30}{5} & \text{if} \quad 30 \leq f_{ck} < 35 \\ 3.7 + 0.3\dfrac{f_{ck} - 35}{5} & \text{if} \quad 35 \leq f_{ck} < 40 \\ 4.0 & \text{if} \quad f_{ck} \geq 40 \end{cases} \qquad \text{(IS 40.2.3)}$$

2.3.3 Determine the Required Shear Reinforcing

Given V_u, the required shear reinforcing in the form of area within a spacing, s_v, is given by the following:

- Calculate the design nominal shear stress as

$$\tau_v = \frac{V_u}{t_w d_w} \qquad \text{(IS 32.4.3; IS 13920 9.2.1)}$$

where,

t_w = Thickness of the wall pier planar leg,

d_w = Effective depth of the wall pier planar leg,

= $8\ L_w$ (IS 32.4.2; IS 13920 9.2.1)

L_w = Overall horizontal length of the wall pier planar leg.

- Calculate the design permissible nominal shear stress, τ_{cd}, following the procedure described in the previous section (IS 40.2.1, Table 19, 40.2; IS 13920 9.2.2: IS 32.4.2).

- Calculate the absolute maximum permissible nominal shear stress, $\tau_{c,\max}$, following the procedure described in the previous section (IS 40.2.3, Table 20, IS 34.4.2.1).

- Compute the horizontal shear reinforcement as follows:

  – If $\tau_v < \tau_{cd}$, provide minimum links defined by the minimum horizontal shear rebar in accordance with code (IS 32.5; IS 13920 9.2.4, 9.14),

  else if $\tau_{cd} < \tau_v < \tau_{c,\max}$,

$$\frac{A_{sh}}{s_v} = \frac{\left(\tau_v - \tau_{cd}\right) t_w}{0.87 f_{ys}} \qquad \text{(IS 40.4, 32.4.4; IS 13920 9.2.5)}$$

  else if $\tau_v > \tau_{c,\max}$,

  a failure condition is declared. (IS 32.4.2.1; IS 13920 9.2.3)

  In calculating the horizontal shear reinforcement, a limit is imposed on the f_{ys} as

$$f_{ys} \leq 415 \text{ N/mm}^2. \qquad \text{(IS 13920 5.3; IS 40.4)}$$

The minimum ratio of horizontal shear reinforcement to gross concrete area is taken as follows:

- If there is any load combination involving seismic load present (seismic design), the minimum horizontal shear reinforcing in the wall pier planar leg is taken as

$$\frac{A_{sh}}{s_v} = 0.0025 t_w. \qquad \text{(IS 13920 9.1.4)}$$

- If there is no load combination involving seismic load present (non-seismic design), the minimum horizontal shear reinforcing in the wall pier planar leg is taken as:

$$\frac{A_{sh}}{s_v} = \begin{cases} 0.0025 t_w & \text{if } f_{ys} \leq 415 \text{N/mm}^2, \\ 0.0020 t_w & \text{if } f_{ys} > 415 \text{N/mm}^2. \end{cases} \qquad \text{(IS 32.5.c)}$$

The maximum of all the calculated A_{sh}/s values, obtained from each design load combination, are reported for the major direction of the wall pier, along with the controlling combination name.

The wall pier planar leg shear reinforcement requirements reported by the program are based purely on shear strength consideration. Any other requirements to satisfy spacing considerations or transverse reinforcement volumetric considerations must be investigated independently of the program by the user.

Chapter 3
Spandrel Design

This chapter describes how the program designs concrete shear wall spandrels for flexure and shear when the "Indian IS 456-2000" code is selected. The program allows consideration of Rectangular sections and T-beam sections for shear wall spandrels. Note that the program designs spandrels at stations located at the ends of the spandrel. No design is performed at center (mid-length) of the spandrel. To compute steel reinforcement at stations located between the two ends, either specify auto mesh options or do a manual meshing of the spandrel and then assign different spandrel labels. The program designs the spandrel for flexure and shear only and reports how much flexural and shear reinforcing is required. The program does not allow reinforcing to be specified or checked.

3.1 Spandrel Flexural Design

In this program, wall spandrels are designed for major direction flexure and shear only. Effects caused by any axial forces, minor direction bending, torsion or minor direction shear that may exist in the spandrel must be investigated by the user independent of the program. Spandrel flexural reinforcing is designed for each of the design load combinations. The required area of reinforcing for flexure is calculated and reported at the ends of the spandrel beam.

The following steps are involved in designing the flexural reinforcing for a particular wall spandrel section for a particular design loading combination at a particular station.

- Determine the maximum factored moment M_u.
- Determine the required flexural reinforcing, A_s and A_s'.

These steps are described in the following sections.

3.1.1 Determine the Maximum Factored Moments

In the design of flexural reinforcing for spandrels, the factored moments for each design load combination at a particular beam station are obtained first. Then the beam section is designed for the maximum positive and the maximum negative factored moments obtained from all of the design load combinations.

3.1.2 Determine the Required Flexural Reinforcing

In this program, negative beam moments produce top steel. In such cases, the beam is always designed as a rectangular section.

In this program, positive beam moments produce bottom steel. In such cases, the beam may be designed as a rectangular section, or as a T-beam section. Indicate that a spandrel is to be designed as a T-beam by specifying the appropriate slab width and depth dimensions, which are provided in the spandrel design overwrites (Appendix B).

The flexural design procedure is based on the simplified parabolic stress block, as shown in Figure 3-1 (IS 38.1, Figures 21 and 22). The area of the stress block, C, and the depth of the center of the compressive force from the most compressive fiber, $d_{\text{compression}}$, are taken as

$$C = a f_{ck} x_u \text{ and} \qquad \text{(IS 28.1.b)}$$

$$d_{\text{compression}} = \beta x_u \qquad \text{(IS 38.1.c)}$$

where x_u is the depth of the compression block, and a and β are taken respectively as

$a = 0.36$, and (IS 38.1.c)

$\beta = 0.42$. (IS 38.1.c)

a is the reduction factor to account for sustained compression and the partial safety factor for concrete. a is taken as 0.36 for the assumed parabolic stress block (IS 38.1). The β factor establishes the location of the resultant compressive force in concrete in terms of the neutral axis depth.

Furthermore, it is assumed that moment redistribution in the member does not exceed the code specified limiting value. The code also places a limit on the neutral axis depth as given below, to safeguard against non-ductile failures (IS 38.1.f).

f_y	$x_{u,\max}/d$
250	0.53
415	0.48
500	0.46

The program uses interpolation between the three discrete points given in the code.

$$\frac{x_{u,\max}}{d} = \begin{cases} 0.53 & \text{if} \quad f_y \le 250 \\ 0.53 - 0.05\dfrac{f_y - 250}{165} & \text{if} \quad 250 < f_y \le 415 \\ 0.48 - 0.02\dfrac{f_y - 415}{85} & \text{if} \quad 415 < f_y \le 500 \\ 0.46 & \text{if} \quad f_y > 500 \end{cases} \qquad \text{(IS 38.1.f)}$$

The preceding table and the interpolating equation are a manifestation of Clause 38.1(f), which states that the maximum strain in the tension reinforcement in the section at failure should not be less than $0.002 + f_y/(1.15E_s)$ (IS 38.1.f). This leads to the following limit:

$$\frac{x_{u,\max}}{d} = \frac{\varepsilon_{c,\min}}{\varepsilon_{c,\max} + \varepsilon_{s,\min}} \qquad \text{(IS 38.1.f)}$$

where

$$\varepsilon_{c,\max} = 0.0035 \qquad \text{(IS 38.1.b)}$$

$$\varepsilon_{s,\min} = 0.002 + \frac{f_y}{1.15E_s} \qquad \text{(IS 38.1.f)}$$

If the spandrel satisfies the deep beam criterion, a limit on the depth of the lever arm, z, is enforced based on the assumption that the spandrel is a continuous beam.

$$z_{\max} = \begin{cases} 0.5l_s & \text{for} \quad 0 < l_s/D < 1 \\ 0.2\left(l_s + 1.5D\right) & \text{for} \quad 1 \le l_s/D \le 2.5 \end{cases} \qquad \text{(IS 29.2.6)}$$

The spandrel is considered to be a deep beam if the following condition is satisfied:

$$l_s/D \le 2.5 \qquad \text{(IS 29.1.a)}$$

where D is the total depth of the spandrel and l_s is the effective span, which is taken as 1.15 times the clear span (IS 29.2).

When the applied moment exceeds the capacity of the beam as a singly reinforced beam, the area of compression reinforcement is calculated on the assumption that the neutral axis depth remains at the maximum permitted value. The maximum fiber compression is taken as

$$\varepsilon_{c,\max} = 0.0035 \qquad \text{(IS 38.1.b)}$$

and the modulus of elasticity of steel is taken to be

$$E_s = 200{,}000 \text{ N/mm}^2. \qquad \text{(IS 5.63, 38.1.e, Figure 23)}$$

It is assumed that the design ultimate axial force can be neglected. The effect of torsion is neglected.

The design procedure used by the program for both rectangular and flanged sections (L-beams and T-beams) is summarized in the sections that follow.

3.1.2.1 Design for Rectangular Spandrel Beam

For rectangular spandrel beams, the limiting depth of neutral axis, $x_{u,\max}$, and the moment capacity as a singly reinforced beam, M_{single}, are obtained first for the section. The reinforcing steel area is determined based on whether M_u is greater than, less than, or equal to M_{single}. (See Figure 3-1.)

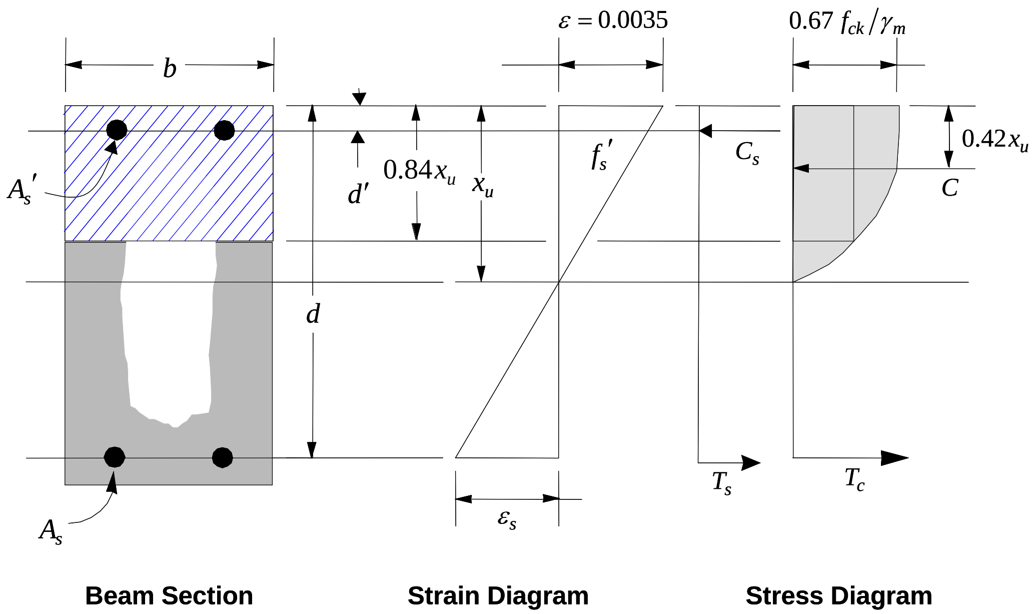

Figure 3-1 Rectangular Spandrel beam design, Positive Moment

- Calculate the limiting depth of the neutral axis.

$$\frac{x_{u,\max}}{d} = \begin{cases} 0.53 & \text{if} \quad f_y \le 250 \\ 0.53 - 0.05\dfrac{f_y - 250}{165} & \text{if} \quad 250 < f_y \le 415 \\ 0.48 - 0.02\dfrac{f_y - 415}{85} & \text{if} \quad 415 < f_y \le 500 \\ 0.46 & \text{if} \quad f_y \ge 500 \end{cases} \qquad \text{(IS 38.1.f)}$$

- Calculate the limiting ultimate moment of resistance as a singly reinforced beam.

$$M_{\text{single}} = \alpha f_{ck} b d^2 \, \frac{x_{u,\max}}{d} \left[1 - \beta \frac{x_{u,\max}}{d} \right], \text{ where} \qquad \text{(IS G-1.1.c)}$$

$$\alpha = 0.36, \text{ and} \qquad \text{(IS G-1.1.c, 38.1.c)}$$

$$\beta = 0.42. \qquad \text{(IS G-1.1.c, 38.1.c)}$$

- Calculate the depth of neutral axis x_u as

$$\frac{x_u}{d} = \frac{1 - \sqrt{1 - 4\beta m}}{2\beta},$$

where the normalized design moment, m, is given by

$$m = \frac{M_u}{\alpha f_{ck} b d^2}.$$

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

$$A = \frac{M_u}{\left(f_y / \gamma_s \right) z}, \quad \text{where} \qquad \text{(IS G-1.1)}$$

$$z = d \left\{ 1 - \beta \frac{x_u}{d} \right\}. \qquad \text{(IS 38.1)}$$

If the spandrel is a deep beam, z is limited by the following limit:

$$z \le z_{\max}. \qquad \text{(IS 29.2.5)}$$

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

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

$$A_s' = \frac{M_u - M_{\text{single}}}{f_s' \left(d - d' \right)}, \qquad \text{(IS G-1.2)}$$

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

$$f_s' = \varepsilon_{c,\max} E_s \left[1 - \frac{d'}{x_{u,\max}}\right] \le \frac{f_y}{\gamma_s}. \qquad \text{(IS G-1.2)}$$

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

$$A_s = \frac{M_{\text{single}}}{\left(f_y/\gamma_s\right)z} + \frac{M_u - M_{\text{single}}}{\left(f_y/\gamma_s\right)\left(d - d'\right)}, \text{ where} \qquad \text{(IS G-1.2)}$$

$$z = d\left\{1 - \beta \frac{x_{u,\max}}{d}\right\}. \qquad \text{(IS 38.1)}$$

If the spandrel is a deep beam, z is limited by the following limit:

$$z \le z_{\max}. \qquad \text{(IS 29.2.5)}$$

A_s is to be placed at the bottom and A_s' is to be placed at the top if M_u is positive, and A_s' is to be placed at the bottom and A_s is to be placed at the top if M_u is negative.

3.1.2.2 Design of Spandrel with T-Beam Section

In designing a T-beam spandrel, a simplified stress block, as shown in Figure 3-2, is assumed if the flange is under compression, i.e., if the moment is positive. If the moment is negative, the flange comes under tension, and the flange is ignored. In that case, a simplified stress block similar to that shown in Figure 3-1 is assumed in the compression side.

3.1.2.2.1 Flanged Spandrel 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 described for a rectangular beam, i.e., no T-beam data is used. The width of the web, b_w, is used as the width of the beam.

3.1.2.2.2 Flanged Spandrel 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 3-2.

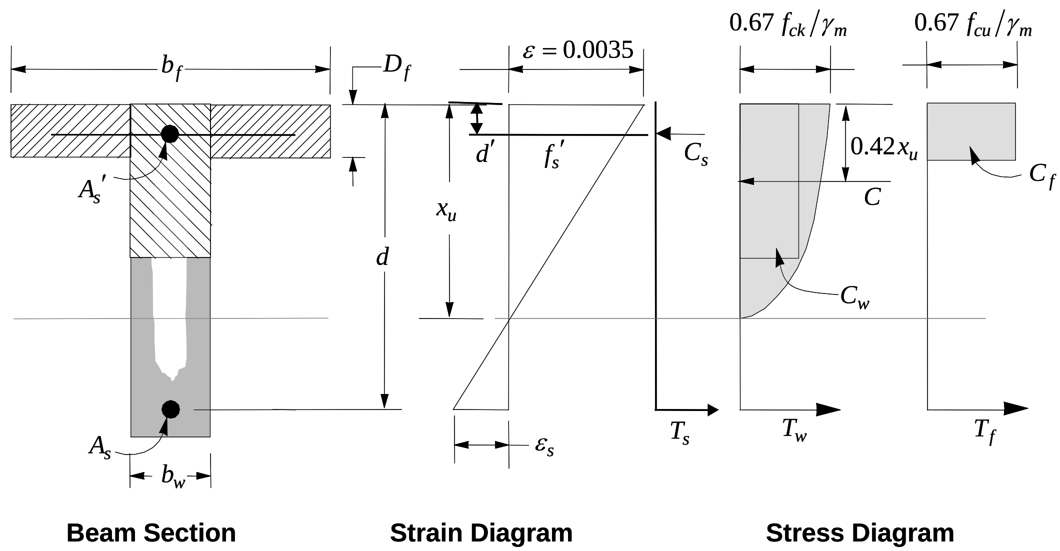

Figure 3-2 Design of Wall Spandrel with a T-Beam Section, Positive Moment

- Assuming the neutral axis to lie in the flange, calculate the depth of neutral axis, x_u, as

$$\frac{x_u}{d} = \frac{1 - \sqrt{1 - 4\beta m}}{2\beta},$$

where the normalized design moment, m, is given by

$$m = \frac{M_u}{\alpha f_{ck} b_f d^2}.$$

- If $\left(\frac{x_u}{d}\right) \le \left(\frac{D_f}{d}\right)$, the neutral axis lies within the flange. The subsequent calculations for A_s are exactly the same as previously defined for the Rectangular section design (IS G-2.1). However, in this case, the width of the compression flange, b_f, is taken as the width of the beam, b. Compression reinforcement is required when $M_u > M_{\text{single}}$.

- If $\left(\frac{x_u}{d}\right) > \left(\frac{D_f}{d}\right)$, the neutral axis lies below the flange. Then calculation for A_s has 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 3-2.

  – Calculate the ultimate resistance moment of the flange as

  $$M_f = 0.45\, f_{ck}\,(b_f - b_w) y_f\,(d - 0.5\, y_f)\,, \qquad \text{(IS G-2.2)}$$

  where y_f is taken as follows:

  $$y_f = \begin{cases} D_f & \text{if} \quad D_f \le 0.2d, \\ 0.15x_u + 0.65D_f & \text{if} \quad D_f > 0.2d. \end{cases} \qquad \text{(IS G-2.2)}$$

  – Calculate the moment taken by the web as

  $$M_w = M_u - M_f.$$

  – Calculate the limiting ultimate moment resistance of the web for tension reinforcement only.

  $$M_{w,\text{single}} = \alpha f_{ck} b_w d^2\, \frac{x_{u,\max}}{d}\left[1 - \beta \frac{x_{u,\max}}{d}\right] \text{ where} \qquad \text{(IS G-1.1)}$$

$$\frac{x_{u,\max}}{d} = \begin{cases} 0.53 & \text{if} \quad f_y \leq 250 \\ 0.53 - 0.05\dfrac{f_y - 250}{165} & \text{if} \quad 250 < f_y \leq 415 \\ 0.48 - 0.02\dfrac{f_y - 415}{85} & \text{if} \quad 415 < f_y \leq 500 \\ 0.46 & \text{if} \quad f_y > 500 \end{cases} \qquad \text{(IS 38.1.f)}$$

$\alpha = 0.36$, and (IS G-1.1.c, 38.1.c)

$\beta = 0.42$. (IS G-1.1.c, 38.1.c)

– If $M_w \leq M_{w,\text{single}}$, the spandrel 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.

$$A_s = \frac{M_f}{\left(f_y/\gamma_s\right)\left(d - 0.5y_f\right)} + \frac{M_w}{\left(f_y/\gamma_s\right)z}, \text{ where}$$

$$z = d\left\{1 - \beta\frac{x_u}{d}\right\}.$$

$$\frac{x_u}{d} = \frac{1 - \sqrt{1 - 4\beta m}}{2\beta}, \text{ and}$$

$$m = \frac{M_w}{\alpha f_{ck} b_w d^2}.$$

If the spandrel is a deep beam, z is limited by the following limit:

$z \leq z_{\max}$ (IS 29.2.b)

– If $M_w > M_{w,\text{single}}$, the area of compression reinforcement, A_s', is given by

$$A_s' = \frac{M_w - M_{w,\text{single}}}{f_s'\left(d - d'\right)},$$

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

$$f_s' = \varepsilon_{c,\max} E_s \left[1 - \frac{d'}{x_{u,\max}}\right] \leq \frac{f_y}{\gamma_s}. \qquad \text{(IS G-1.2)}$$

From equilibrium, the area of tension reinforcement is calculated as

$$A_s = \frac{M_f}{\left(f_y/\gamma_s\right)\left(d - 0.5y_f\right)} + \frac{M_{w,\text{single}}}{\left(f_y/\gamma_s\right)z} + \frac{M_w - M_{w,\text{single}}}{\left(f_y/\gamma_s\right)(d - d')},$$

where

$$z = d\left\{1 - \beta\frac{x_{u,\max}}{d}\right\}.$$

If the spandrel is a deep beam, z is limited by the following limit:

$$z \leq z_{\max}. \qquad \text{(IS 29.2.b)}$$

A_s is to be placed at the bottom and A_s' is to be placed at the top for positive moment.

3.2 Spandrel Shear Design

The program allows consideration of Rectangular sections and T-Beam sections for wall spandrels. The shear design for both of these types of spandrel sections is identical.

The wall spandrel shear reinforcing is designed for each of the design load combinations. The required area of reinforcing for vertical shear is calculated only at the ends of the spandrel beam.

In this program, wall spandrels are designed for major direction flexure and shear forces only. Effects caused by any minor direction bending, torsion or minor direction shear that may exist in the spandrels must be investigated by the user independent of the program.

The assumptions in designing the shear reinforcement are as follows:

- The spandrel beam sections are considered to be prismatic. The program does not adjust the shear force for non-prismatic spandrel beam (IS 40.1.1).
- The effect on the concrete shear capacity of any concentrated or distributed load in the span of the spandrel beam between two walls or columns is ignored. Also, the effect of the direction support on the beams provided by the columns or walls is ignored.
- All shear reinforcement is assumed to be perpendicular to the longitudinal reinforcement.
- The effect of axial force is considered (IS 40.2.2.).
- The effect of torsion is ignored (IS 41.3.1).

The following steps are involved in designing the shear reinforcement for a particular wall spandrel section for a particular design loading combination at a particular station.

- Determine the factored shear force V_u.
- Determine the shear stress, τ_{cd}, that can be carried by the concrete.
- Determine the required shear reinforcing, A_{sv}/s_v, to carry the balance of the shear force.

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

3.2.1 Determine Factored Shear Force

In the design of spandrel beam shear reinforcement, the shear forces and axial force for a particular design load combination at a particular beam section are obtained by factoring the associated shear forces and axial force with the corresponding design load combination factors.

The section is essentially considered to be prismatic. If the section is non-prismatic and the depth varies with length, the program does not adjust the design shear force as recommended by code (IS 40.1.1). In that case, the user is expected to check the shear independently of the program.

The program ignores any torsion that might be present. If the spandrel encounters any torsion, the program does not adjust the design shear force as recommended by the code (IS 41.3.1). In that case, the user is expected to check the shear independently of the program.

3.2.2 Determine Concrete Shear Capacity

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

$$V_c = \tau_{cd} A_{cv}$$

where,

$$A_{cv} = b_w d \qquad \text{(IS 40.1)}$$

$$d = \min\left\{h - d_{r,\text{top}}, h - d_{r,\text{bot}}\right\}. \qquad \text{(IS 40.2)}$$

$$\tau_{cd} = k\delta\tau_c \qquad \text{(IS 40.2)}$$

$$\delta = \begin{cases} 1 + 3\dfrac{P_u}{A_g f_{ck}} \leq 1.5 & \text{if } P_u > 0, \text{ under compression} \\ 1 & \text{if } P_u \leq 0, \text{ under tension} \end{cases} \qquad \text{(IS 40.2.2)}$$

k is always taken as 1, and (IS 40.2.1.1)

τ_c is the basic design shear strength for concrete, which is given by IS Table 19. It should be mentioned that the value of γ_c has already been incorporated in IS Table 19 (see Note in IS 36.4.2.1). The following limitations are enforced in the determination of the design shear strength as is done in Table 19.

$$0.15 \leq \frac{100 A_s}{bd} \leq 3$$

$f_{ck} \leq 40$ N/mm^2 (for calculation purposes only)

The determination of τ_c from Table 19 of the code requires two parameters: $100\dfrac{A_s}{bd}$ and f_{ck}. If $100\dfrac{A_s}{bd}$ becomes more than 3.0, τ_c is calculated based on $100\dfrac{A_s}{bd}$ = 3.0. If $100\dfrac{A_s}{bd}$ becomes less than 0.15, τ_c is calculated based on $100\dfrac{A_s}{bd}$ = 0.15. Similarly, if f_{ck} is larger than 40 N/mm^2, τ_c is calculated based on f_{ck} = 40 N/mm^2. However, if f_{ck} is less than 15 N/mm^2, τ_c is reduced by a factor of $\left(\dfrac{f_{ck}}{0.15}\right)^{1/4}$. If γ_c is chosen to be different from 1.5, τ_c is adjusted with a factor of $\left(1.5/\gamma_c\right)$. The absolute maximum limit on nominal shear stress, $\tau_{c,\max}$ is calculated in accordance with IS Table 20, which is reproduced in the table that follows (IS 40.2.3, Table 20):

Maximum Shear Stress, $\tau_{c,\max}$ (N/mm^2)						
Concrete Grade	M15	M20	M25	M30	M35	M40
$\tau_{c,\max}$ (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4.0

If f_{ck} is between the limits, linear interpolation is used.

$$\tau_{c,\max} = \begin{cases} 2.5 & \text{if} \quad f_{ck} < 15 \\ 2.5 + 0.3\dfrac{f_{ck} - 15}{5} & \text{if} \quad 15 \le f_{ck} < 20 \\ 2.8 + 0.3\dfrac{f_{ck} - 20}{5} & \text{if} \quad 20 \le f_{ck} < 25 \\ 3.1 + 0.4\dfrac{f_{ck} - 25}{5} & \text{if} \quad 25 \le f_{ck} < 30 \\ 3.5 + 0.2\dfrac{f_{ck} - 30}{5} & \text{if} \quad 30 \le f_{ck} < 35 \\ 3.7 + 0.3\dfrac{f_{ck} - 35}{5} & \text{if} \quad 35 \le f_{ck} < 40 \\ 4.0 & \text{if} \quad f_{ck} \ge 40 \end{cases} \qquad \text{(IS 40.2.3)}$$

3.2.3 Determine Required Shear Reinforcement

Given V_u, the required shear reinforcement in the form of stirrups within a spacing, s_v, is given for Rectangular or T-beams by the following:

- Calculate the design nominal shear stress as

$$\tau_v = \frac{V_u}{bd} \qquad \text{(IS 40.1)}$$

where b is the width of the Rectangular beam or the width of the T-beam web, i.e., ($b = b_w$).

- Calculate the basic permissible nominal shear stress, τ_c, and the design permissible nominal shear stress, τ_{cd}, following the procedure described in the previous section (IS 40.2.1, Table 19, 40.2).

- Calculate the absolute maximum permissible nominal shear stress, $\tau_{c,\max}$, following the procedure described in the previous section (IS 40.2.3, Table 20).

- Compute the shear reinforcement as follows:

If $\tau_v \le \tau_{cd}$, provide minimum links defined by

$$\frac{A_{sv}}{s_v} = \frac{0.4 b_w}{0.87 f_{ys}} \qquad \text{(IS 40.3, 26.5.1.6)}$$

else if $\tau_{cd} < \tau_v \le \tau_{c,\max}$, provide links given by

$$\frac{A_{sv}}{s_v} = \frac{(\tau_v - \tau_{cd}) b_w}{0.87 f_{ys}} \ge \frac{0.46 b_w}{0.87 f_{ys}}, \qquad \text{(IS 40.4)}$$

else if $\tau_v > \tau_{c,\max}$,

a failure condition is declared. (IS 40.2.3)

The following additional checks are performed for spandrels involving seismic load or spandrels satisfying the deep beam criterion.

- If any load combination involving seismic load is present (seismic design), the minimum areas of vertical and horizontal shear reinforcing in the spandrel are as follows (IS 13920 9.1.4):

$$A_{sv}/s_v = 0.0025b_w \qquad \text{(IS 13920 9.1.4)}$$

$$A_{sh}/s_h = 0.0025b_w \qquad \text{(IS 13920 9.1.4)}$$

- If the spandrel beam satisfies the deep beam criterion, the minimum areas of vertical and horizontal shear reinforcing in the spandrel are as follows (IS 32.5a):

$$\frac{A_{sv}}{s_v} = 0.0025b_w \qquad \text{(IS 32.5.c)}$$

$$\frac{A_{sh}}{s_h} = 0.0015b_w\,. \qquad \text{(IS 32.5.a)}$$

A spandrel beam is considered to be a deep beam if the following condition is satisfied.

$$L/h \le 2.5. \qquad \text{(IS 29.1)}$$

The length L is taken as 1.15 times the clear span (IS 29.2).

In calculating the shear reinforcement, a limit is imposed on the f_{ys} as

$$f_{ys} \le 415\ \text{N/mm}^2. \qquad \text{(IS 39.4)}$$

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

The shear reinforcement requirements displayed 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.

3.3 Spandrel Diagonal Reinforcement

If seismic load is not present, diagonal reinforcement is not required. If seismic load is present, diagonal reinforcement may or may not be required.

If

$$l_s / D \leq 3, \qquad \text{(IS 13920 Draft 9.5.1)}$$

or if

$$\tau_v > 0.1\sqrt{f_{ck}}\,(l_s / D), \qquad \text{(IS 13920 9.5.1)},$$

diagonal reinforcement is required. The area of reinforcement to be provided along each diagonal in a diagonally reinforced spandrel beam is computed as follows:

$$A_{sd} = \frac{V_u}{2\left(f_{ys}/\lambda_s\right)\sin\alpha} \qquad \text{(IS 13920 9.5.2)}$$

where

$$\sin\alpha = \frac{0.8D}{\sqrt{l_s^{\,2} + (0.8D)^2}}$$

where D is the total depth of the spandrel and l_s is the length of the spandrel.

In the output, the program reports the diagonal shear reinforcing as required or not required (i.e., optional). The diagonal shear reinforcing is reported as required if $l_s / D \leq 3$ or if earthquake-induced shear stress exceeds 0.1 $l_s \Big/ \sqrt{f_{ck}/D}$.

Appendix A
Shear Wall Design Preferences

The shear wall design preferences are basic properties that apply to all wall pier and spandrel elements. Table B1 identifies shear wall design preferences for IS 456-2000. Default values are provided for all shear wall design preference items. Thus, it is not required that preferences be specified. However, at least review the default values for the preference items to make sure they are acceptable. Refer to the program Help for an explanation of how to change a preference.

Table A1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	Indian IS 456-2000	Design code used for design of concrete shear wall elements (wall piers and spandrels)
Rebar units	in^2, cm^2, mm^2, current	in^2 or mm^2	Units used for concentrated areas of reinforcing steel..
Rebar/Length Units	in^2/ft, cm^2/m, mm^2/m, current	in^2/ft or mm^2/m	Units used for distributed areas of reinforcing steel.
Gamma (Steel)	> 0	1.15	The potential safety factor for steel.

Table A1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
Gamma (Concrete)	> 0	1.15	The potential safety factor for concrete.
Pmax Factor	> 0	0.8	A factor used to reduce the allowable maximum compressive design strength.
Number of Curves	≥ 4	24	Number of equally spaced interaction curves used to create a full 360-degree interaction surface (this item should be a multiple of four). We recommend that you use 24 for this item.
Number of Points	≥ 11	11	Number of points used for defining a single curve in a wall pier interaction surface (this item should be odd).
Edge Design PT-max	> 0	0.06	Maximum ratio of tension reinforcing allowed in edge members, PT_{max}.
Edge Design PC-max	> 0	0.04	Maximum ratio of compression reinforcing allowed in edge members, PC_{max}.
Section Design IP-Max	≥ Section Design IP-Min	0.02	The maximum ratio of reinforcing considered in the design of a pier with a Section Designer section.
Section Design IP-Min	> 0	0.0025	The minimum ratio of reinforcing considered in the design of a pier with a Section Designer section.
Utilization Factor Limit	> 0	0.95	The utilization factor limit. this is the target P-M-M interaction ratio for designing rebar in wall pier.

Appendix B
Design Procedure Overwrites

The shear wall design overwrites are basic assignments that apply only to those piers or spandrels to which they are assigned. The overwrites for piers and spandrels are separate. Tables B1 and B2 identify the shear wall overwrites for piers and spandrels, respectively, for Indian IS 456-2000. Note that the available overwrites change depending on the pier section type (Uniform Reinforcing, General Reinforcing, or Simplified T and C).

Default values are provided for all pier and spandrel overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values for the overwrite items to make sure 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; that is, to the elements that are selected when the overwrites are changed. Refer to the program Help for an explanation of how to change the overwrites.

B1 Pier Design Overwrites

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
Design this Pier	Yes or No	Yes	Toggle for design of the pier when you click the **Design menu > Shear Wall Design > Start Design/Check** command.

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
LL Reduction Factor	Program calculated, > 0	Program calculated	A reducible live load is multiplied by this factor to obtain the reduced live load. Entering 0 for this item means that it is program calculated. Refer to the *LL Reduction Factor* subsection later in this appendix for more information.
Pier Section Type	Uniform Reinforcing, General Reinforcing, Simplified T and C	Uniform Reinforcing	This item indicates the type of pier. The General Reinforcing option is not available unless General pier sections have previously been defined in Section Designer.
Overwrites Applicable to Uniform Reinforcing Pier Sections			
Edge Bar Name	Any defined bar size	Varies	The size of the uniformly spaced edge bars.
Edge Bar Spacing	>0	250 mm	The spacing of the uniformly spaced edge bars.
End/Corner Bar Name	Any defined bar size	Varies	The size of end and corner bars.
Clear Cover	>0	30 mm	The clear cover for the edge, end and corners bars.
Material	Any defined concrete material property	Varies	The material property associated with the pier.
Check/Design Reinforcing	Check or Design	Design	This item indicate whether the pier section is to be designed or checked.
Overwrites Applicable to General Reinforcing Pier Sections			
Section Bottom	Any general pier section defined in Section Designer	The first pier in the list of Section Designer piers	Name of a pier section, defined in Section Designer that is assigned to the bottom of the pier.
Section Top	Any general pier section defined in Section Designer	The first pier in the list of Section Designer piers	Name of a pier section, defined in Section Designer, that is assigned to the top of the pier.
Check/Design Reinforcing	Check or Design	Design	This item indicates whether the pier section is to be designed or checked.

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
Overwrites Applicable to Simplified T and C Pier Sections			
ThickBot	Program calculated, or > 0	Program calculated	Wall pier thickness at bottom of pier, t_p. Inputting 0 means the item is to be program calculated.
LengthBot	Program calculated, or > 0	Program calculated	Wall pier length at bottom of pier, L_p. Inputting 0 means the item is to be program calculated.
DB1LeftBot	≥ 0	0	Length of the bottom of a user-defined edge member on the left side of a wall pier, $DB1_{left}$.
DB2LeftBot	≥ 0	0	Width of the bottom of a user-defined edge member on the left side of a wall pier, $DB2_{left}$. Refer to the *User-Defined Edge Members* subsection that follows this table for more information.
DB1RightBot	≥ 0	Same as DB1-left-bot	Length of the bottom of a user-defined edge member on the right side of a wall pier, $DB1_{right}$.
DB2RightBot	≥ 0	Same as DB2-left-bot	Width of the bottom of a user-defined edge member on the right side of a wall pier, $DB2_{right}$.
ThickTop	Program calculated, or > 0	Program calculated	Wall pier thickness at the top of a pier, t_p. Inputting 0 means the item is to be program calculated.
LengthTop	Program calculated, or > 0	Program calculated	Wall pier length at the top of a pier, L_p. Inputting 0 means the item is to be program calculated.
DB1LeftTop	≥ 0	0	Length of the top of a user-defined edge member on the left side of a wall pier, $DB1_{left}$.
DB2LeftTop	≥ 0	0	Width of the top of a user-defined edge member on the left side of a wall pier, $DB2_{left}$.
DB1RightTop	≥ 0	Same as DB1-left-bot	Length of the top of a user-defined edge member on the right side of a wall pier, $DB1_{right}$.

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
DB2RightTop	≥ 0	Same as DB2-left-bot	Width of the top of a user-defined edge member on the right side of a wall pier, $DB2_{right}$.
Material	Any defined concrete material property	Material Property Used in Pier Section	Material property associated with the pier.
Edge Design PC-max	> 0	Specified in Preferences	Maximum ratio of compression reinforcing allowed in edge members, PC_{max}.
Edge Design PT-max	> 0	Specified in Preferences	Maximum ratio of tension reinforcing allowed in edge members, PT_{max}.

B2 Spandrel Design Overwrites

Table B-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
Design this Spandrel	Yes or No	Yes	Toggle for design of the spandrel when you click the **Design menu > Shear Wall Design > Start Design/Check** command.
LL Reduction Factor	Program calculated, > 0	Program calculated	A reducible live load is multiplied by this factor to obtain the reduced live load. Entering 0 for this item means that it is program calculated. See the subsection entitled "LL Reduction Factor" later in this appendix for more information.
Length	Program calculated, or > 0	Program calculated	Wall spandrel length, L_s. Inputting 0 means the item is to be program calculated.
ThickLeft	Program calculated, or > 0	Program calculated	Wall spandrel thickness at the left side of a spandrel, t_s. Inputting 0 means the item is to be program calculated.
DepthLeft	Program calculated, or > 0	Program calculated	Wall spandrel depth at lthe eft side of a spandrel, h_s. Inputting 0 means the item is to be program calculated.

Table B-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
CoverBotLeft	Program calculated, or > 0	Program calculated	Distance from the bottom of the spandrel to the centroid of the bottom reinforcing, $d_{r\text{-bot left}}$ on the left side of the beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
CoverTopLeft	Program calculated, or > 0	Program calculated	Distance from top of spandrel to centroid of top reinforcing, $d_{r\text{-top left}}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
SlabWidthLeft	≥ 0	0	Slab width for T-beams at the left end of the spandrel, b_s.
SlabDepthLeft	≥ 0	0	Slab depth for T-beams at the left end of the spandrel, d_s.
ThickRight	Program calculated, or > 0	Program calculated	Wall spandrel thickness at the right side of the spandrel, t_s. Inputting 0 means the item is to be program calculated.
DepthRight	Program calculated, or > 0	Program calculated	Wall spandrel depth at the right side of the spandrel, h_s. Inputting 0 means the item is to be program calculated.
CoverBotRight	Program calculated, or > 0	Program calculated	Distance from the bottom of the spandrel to the centroid of the bottom reinforcing, $d_{r\text{-bot right}}$ on the right side of the beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
Cover-TopRight	Program calculated, or > 0	Program calculated	Distance from the top of the spandrel to the centroid of the top reinforcing, $d_{r\text{-top right}}$ on the right side of the beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
SlabWidthRight	≥ 0	0	Slab width for T-beams at the right end of the spandrel, b_s.
SlabDepthRight	≥ 0	0	Slab depth for T-beams at the right end of the spandrel, d_s.
Material	Any defined concrete material property	Material Property Used in Spandrel Section	Material property associated with the spandrel.

B1.1 LL Reduction Factor

If the LL Reduction Factor is program calculated, it is based on the live load reduction method chosen in the live load reduction preferences. If you specify your own LL Reduction Factor, the program ignores any reduction method specified in the live load reduction preferences and simply calculates the reduced live load for a pier or spandrel by multiplying the specified LL Reduction Factor times the reducible live load.

Important Note: The LL reduction factor is ***not*** applied to any load combination that is included in a design load combination. For example, assume you have two static load cases labeled DL and RLL. DL is a dead load and RLL is a reducible live load. Now assume that you create a design load combination named DESCOMB1 that includes DL and RLL. Then for design load combination DESCOMB1, the RLL load is multiplied by the LL reduction factor. Next assume that you create a load combination called COMB2 that includes RLL. Now assume that you create a design load combination called DESCOMB3 that included DL and COMB2. For design load combination DESCOMB3, the RLL load that is part of COMB2 is ***not*** multiplied by the LL reduction factor.

B1.2 User-Defined Edge Members

When defining a user-defined edge member, you must specify both a nonzero value for DB1 and a nonzero value for DB2. If either DB1 or DB2 is specified as zero, the edge member width is taken as the same as the pier thickness and the edge member length is determined by the program.

Appendix C
Analysis Sections and Design Sections

It is important to understand the difference between analysis sections and design sections when performing shear wall design. Analysis sections are simply the objects defined in your model that make up the pier or spandrel section. The analysis section for wall piers is the assemblage of wall and column sections that make up the pier. Similarly, the analysis section for spandrels is the assemblage of wall and beam sections that make up the spandrel. The analysis is based on these section properties, and thus, the design forces are based on these analysis section properties.

The design section is completely separate from the analysis section. Three types of pier design sections are available. They are:

- **Uniform Reinforcing Section:** For flexural designs and/or checks, the program automatically (and internally) creates a Section Designer pier section of the same shape as the analysis section pier. Uniform reinforcing is placed in this pier. The reinforcing can be modified in the pier overwrites. The Uniform Reinforcing Section pier may be planar or it may be three-dimensional.

  For shear design and boundary zone checks, the program automatically (and internally) breaks up the analysis section pier into planar legs and then performs the design on each leg separately and reports the results separately for each leg. Note that the planar legs are derived from the area objects defined in the model, not from the pier section defined in Section Designer. The pier section defined in Section Designer is used for the flexural design/check only.

- **General Reinforcing Section:** For flexural designs and/or checks, the pier geometry and the reinforcing are defined by the user in the Section Designer utility. The pier defined in Section Designer may be planar or it may be three-dimensional.

  For shear design and boundary zone checks, the program automatically (and internally) breaks the analysis section pier into planar legs and then performs the design on each leg separately and reports the results separately for each leg. Note that the planar legs are derived from the area objects defined in the model, not from the pier section defined in Section Designer. The pier section defined in Section Designer is used for the flexural design/check only.

- **Simplified Pier Section:** This pier section is defined in the pier design overwrites. The simplified section is defined by a length and a thickness. The length is in the pier 2-axis direction and the thickness is in the pier 3-axis direction.

In addition, you can, if desired, specify thickened edge members at one or both ends of the simplified pier section. You cannot specify reinforcing in a simplified section. Thus, the simplified section can only be used for design, not for checking user-specified sections. Simplified sections are always planar.

Only one type of spandrel design section is available. It is defined in the spandrel design overwrites. A typical spandrel is defined by a depth, thickness and length. The depth is in the spandrel 2-axis direction; the thickness is in the spandrel 3-axis direction; and the length is in the spandrel 1-axis direction. Spandrel sections are always planar.

In addition, you can, if desired, specify a slab thickness and depth, making the spandrel design section into a T-beam. You cannot specify reinforcing in a spandrel section. Thus, you can only design spandrel sections, not check them.

The pier and spandrel design sections are designed for the forces obtained from the program's analysis, which is based on the analysis sections. In other words, the design sections are designed based on the forces obtained for the analysis sections.

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