

COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS® 2016
Integrated Building Design Software

Shear Wall Design Manual

CSA A23.3-04





Shear Wall Design Manual

CSA A23.3-04

For ETABS® 2016

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Chapter 1

Introduction

This manual describes the details of the shear wall design and stress check algorithms used by the program when the user selects the CSA-A23.3-04 design code. The various notations used in this manual are described in Section 1.1.

The design is based on loading combinations specified by the user (Section 1.2). 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.

The program performs the following design, check, or analysis procedures in accordance with CSA-A23.3-04 requirements:

- Design and check of concrete wall piers for flexural and axial loads (Chapter 2)
- Design of concrete wall piers for shear (Chapter 2)
- Consideration of the boundary element requirements for concrete wall piers using an approach based on the requirements of CSA Section 21.6.7, 21.6.8 and 21.7.3.2 (Chapter 2)
- Design of concrete shear wall spandrels for flexure (Chapter 3)
- Design of concrete wall spandrels for shear (Chapter 3)

The program provides detailed output data for Simplified pier section design, Uniform pier section *design/check* Section Designer pier section *design/Check*, (Chapter 4).

1.1. Notation

Following is the notation used in this manual.

A_{cv}	Net area of a wall pier bounded by the length of the wall pier, L_p , 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_{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_v	Area of reinforcing steel required for shear, mm^2 / mm
A_{vd}	Area of diagonal shear reinforcement in a coupling beam, mm^2

A_{v-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, N
C_f	Concrete compression force in the extruding portion of a T-beam flange, N
C_s	Compression force in wall pier or spandrel reinforcing steel, N
C_w	Concrete compression force in the web of a T-beam, N
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_s	Modulus of elasticity of reinforcing steel, $N-mm^2$, assumed as 200,000 MPa
$IP-max$	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
L_p	Horizontal length of wall pier, mm. This can be different at the top and the bottom of the pier
L_s	Horizontal length of a wall spandrel, mm
LL	Live load
M_r	Nominal bending resistance, N-mm
M_f	Factored bending moment at a design section, N-mm
M_{fc}	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_{ff}	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_{fs}	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_{fw}	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, N

P_{left}	Equivalent axial force in the left edge member of a wall pier used for design, N. 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 CSA-A23.3-04, N
$P_{\text{max Factor}}$	Factor used to reduce the allowable maximum compressive design strength, unitless. CSA-A23.3-04 specifies this factor to be 0.80. This factor can be revised in the preferences.
P_r	Factored axial resistance of a design section, N
P_o	Axial load strength of a wall pier, N
P_{oc}	The maximum compression force a wall pier can carry with strength reduction factors set equal to one, N
P_{ot}	The maximum tension force a wall pier can carry with strength reduction factors set equal to one, N
P_{right}	Equivalent axial force in the right edge member of a wall pier used for design, N. This may be different at the top and the bottom of the wall pier.
P_f	Factored axial force at a design section, N
PC_{max}	Maximum ratio of compression steel in an edge member of a wall pier, unitless
PT_{max}	Maximum ratio of tension steel in an edge member of a wall pier, unitless
R_o	Overstrength related force modification factor used to compute inelastic rotational demand
R_d	Ductility related force modifications factor used to compute inelastic rotational demand
RLL	Reduced live load

T_s	Tension force in wall pier reinforcing steel, N
V_c	The portion of the shear force carried by the concrete, N
V_r	Nominal shear resistance, N
V_s	The portion of the shear force in a spandrel carried by the shear reinforcing steel, N
V_f	Factored shear force at a design section, N
WL	Wind load
a	Depth of the wall pier or spandrel compression block, mm
a_b	Depth of the compression block in a wall spandrel for balanced strain conditions, mm.
a_l	Depth of the compression block in the web of a T-beam, mm
b_s	Width of the compression flange in a T-beam, mm. This can be different on the left and right ends of the T-beam.
c	Distance from the extreme compression fiber of the wall pier or spandrel to the neutral axis, mm
d_{r-bot}	Distance from the bottom of the spandrel beam to the centroid of the bottom reinforcing steel, mm. This can be different on the left and right ends of the beam.
d_{r-top}	Distance from the top of the spandrel beam to the 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 the spandrel beam minus the cover to the centroid of the reinforcing, mm
f_y	Yield strength of the steel reinforcing, MPa. This value is used for flexural and axial design calculations.

f_{ys}	Yield strength of the steel reinforcing, MPa. This value is used for shear design calculations.
f'_c	Concrete compressive strength, MPa. This value is used for flexural and axial design calculations.
f'_{cs}	Concrete compressive strength, MPa. This value is used for shear design calculations.
f'_s	Stress in compression steel of a wall spandrel, MPa.
h_s	Height of a wall spandrel, mm. This can be different on the left and right ends of the spandrel.
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.
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.
$\sum DL$	The sum of all dead load cases
$\sum LL$	The sum of all live load cases
$\sum RLL$	The sum of all reduced live load cases
α	The angle between the diagonal reinforcing and the longitudinal axis of a coupling beam
α_1	Factor for obtaining average compressive stress in a concrete block.
β_1	Factor indicating the ability of diagonally cracked concrete to transmit tension.

ε	Reinforcing steel strain, unitless
ε_c	Limiting strain in compression, unitless. It is taken as 0.0035
ε_s	Reinforcing steel strain in a wall pier, unitless
ε'_s	Compression steel strain in a wall spandrel, unitless
θ	Angle of inclination of diagonal compressive stresses with the longitudinal axis of beam or column
ϕ	Strength reduction factor, unitless
ϕ_c	Strength reduction factor for concrete, 0.65 (CSA 8.4.2)
ϕ_s	Strength reduction factor for steel, 0.85 (CSA 8.4.3)
λ	Modification factor reflecting the reduced mechanical properties of light-weight concrete, all relative to normal weight concrete of the same compressive strength. It is equal to 1 for normal weight concrete.
σ_s	Reinforcing steel stress in a wall pier, MPa

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 a spandrel is broken 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 for 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 load combinations automatically created by the program for concrete shear wall design are given by Case 1 to Case 5 (CSA 8.3.2, Table C-1). These cases are further categorized as into subcase (a) to (d) to identify different directions of Wind and Earthquake load.

$1.4\sum DL$	Case 1
$1.25\sum DL + 1.5(\sum LL + \sum RLL)$	Case 2
$1.25\sum DL + 0.5(\sum LL + \sum RLL) + 0.4WL$	Case 2(a)
$1.25\sum DL + 0.5(\sum LL + \sum RLL) - 0.4WL$	Case 2(b)
$1.25\sum DL + 1.4WL$	Case 4(a)
$1.25\sum DL - 1.4WL$	Case 4(b)
$0.9\sum DL + 1.4WL$	Case 4(c)
$0.9\sum DL - 1.4WL$	Case 4(d)
$1.0\sum DL + 0.5(\sum LL + \sum RLL) + 1.0E$	Case 5(a)
$1.0\sum DL + 0.5(\sum LL + \sum RLL) - 1.0E$	Case 5(b)
$1.0\sum DL + 1.0E$	Case 5(c)
$1.0\sum DL - 1.0E$	Case 5(d)

In the preceding Equations,

$\sum DL$ = The sum of all dead load (DL) load cases defined for the model.

$\sum LL$ = The sum of all live load (LL) load cases defined for the model.

Note that this includes roof live loads as well as floor live loads.

$\sum RLL$ = The sum of all reducible live load (RLL) load cases defined for the model.

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

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

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.

See the description of the earthquake load component later in this chapter for additional information.

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 unreducible, 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 of CSA Case 2 and 4 will contribute multiple design load combinations, one for each wind load case that is defined.

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 of CSA Case 5 will contribute multiple design load combinations, one for each earthquake load case that is defined.

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 program 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 above, CSA Case 5(b) with negative sign for earthquake is redundant for a load combination with a response spectrum; similarly, CSA Case 5(d) with negative sign for earthquake is redundant for a load combination with a response spectrum. For this reason, the program creates default de-

sign load combinations based only on CSA Cases with positive sign for earthquake for response spectra. Default design load combinations using CSA Case 5(b) and Case 5(d) with negative sign for earthquake 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 can be for the envelope of those results or for each step of the time history. The type of time history design can be specified in the shear wall design preferences.

When envelopes are used, the design is for the maximum of each response quantity (axial load, moment, and the like) as if they occurred simultaneously. Typically, this is not the realistic case, and in some instances, it may be unconservative. Designing for each step of a time history gives the correct correspondence between different response quantities, but designing for each step can be very time consuming.

When the program gets the envelope results for a time history, it gets a maximum and a minimum value for each response quantity. Thus, for wall piers it gets maximum and minimum values of axial load, shear and moment; and for wall spandrels, it gets maximum and minimum values of shear and moment. For a design load combination in the program shear wall design module, any load combination that includes a time history load case in it is checked for all possible combinations of maximum and minimum time history design values. Thus, when checking shear in a wall pier or a wall spandrel, the time history contribution of shear to the design load combination is considered once as a maximum shear and then a second time as a minimum shear. Similarly, when checking moment in a wall spandrel, the time history contribution of moment to the design load combination is considered once as a maximum moment and then a second time as a minimum moment. When checking the flexural behavior of a wall pier, four possible combinations are considered for the contribution of time history load to the design load combination. They are:

- P_{\max} and M_{\max}
- P_{\max} and M_{\min}
- P_{\min} and M_{\max}
- P_{\min} and M_{\min}

where P is the axial load in the pier and M is the moment in the pier.

If a single design load combination has more than one time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified for the Time History Design item in the preferences.

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 performed for the last step of the static nonlinear analysis only.

1.4. 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 CSA-A23.3-04. 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.

1.5. 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 CSA-A23.3-04. Note that the available overwrites change depending on the pier section type (Uniform Reinforcing, General Reinforcing, or Simplified C and T).

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.

1.6. 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 CSA-A23.3-04 code is based on newton-millimeter-second units. For simplicity, all equations and descriptions presented in this manual correspond to **newton-millimeter-Second** units unless otherwise noted.

The shear wall design preferences allow the user to specify special units for concentrated and distributed areas of reinforcing. These units are then used for reinforcing 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 m, this option means concentrated areas of reinforcing are in m^2 and distributed areas of reinforcing are in m^2/m . Note that when using the "current" option, areas of distributed reinforcing are specified in $\text{Length}^2/\text{Length}$ units, where Length is the currently active length unit. For example, if you are working in kN and m units, the area of distributed reinforcing is specified in m^2/m . If you are in kN and mm, the area of distributed reinforcing is specified in mm^2/mm .

Chapter 2 Pier Design

This chapter describes how the program designs and checks concrete wall piers for flexural and axial loads using CSA-A23.3-04, which was reaffirmed in 2010. 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 Uniform Pier Reinforcing Section or General Section (i.e., Designer Section). Then we describe how the program *designs* piers that are specified by a Uniform Pier Reinforcing Section or General (Section Designer) Section.

This chapter also describes how the program designs each leg of concrete wall piers for shear using CSA-A23.3-04. Note that in this program it is not possible to 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 the design of boundary zone elements for each pier in accordance with CSA Section 21.6.7, 21.6.8 and 21.7.3.2 when a seismic load case is present in wall design load combinations.

2.1 Wall Pier Flexural Design

For both designing and checking piers, it is important to understand the local axis definition for the pier. Access the local axes assignments using the Assign menu.

2.1.1 Designing a Simplified 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 2-1. The pier geometry is defined by a length, thickness and size of the edge members at each end of the pier (if any).

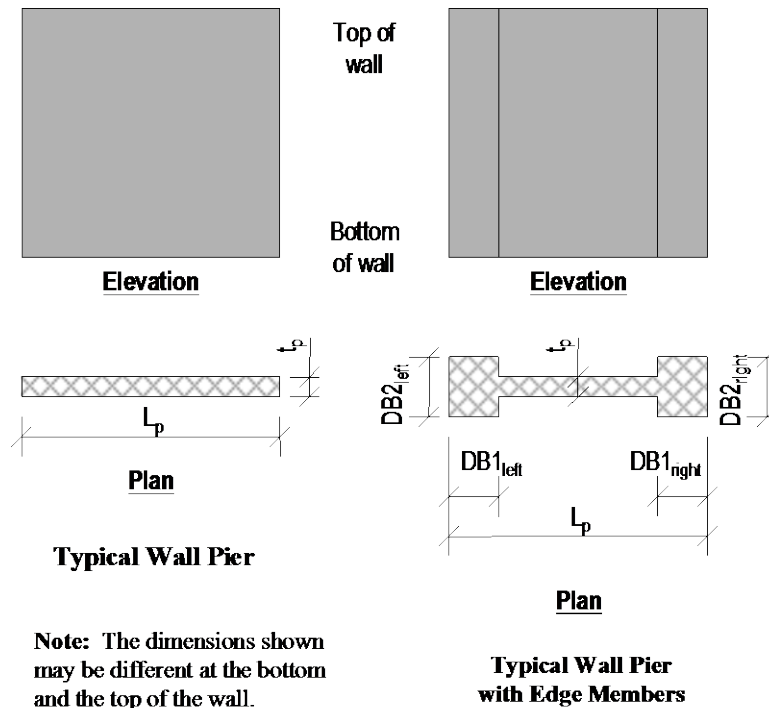


Figure 2-1: Typical Wall Pier Dimensions Used for Simplified Design

A simplified C and T 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_p . 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.
- $DB2$ 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.

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 wall pier. These conditions, illustrated in Figure 2-2, 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.

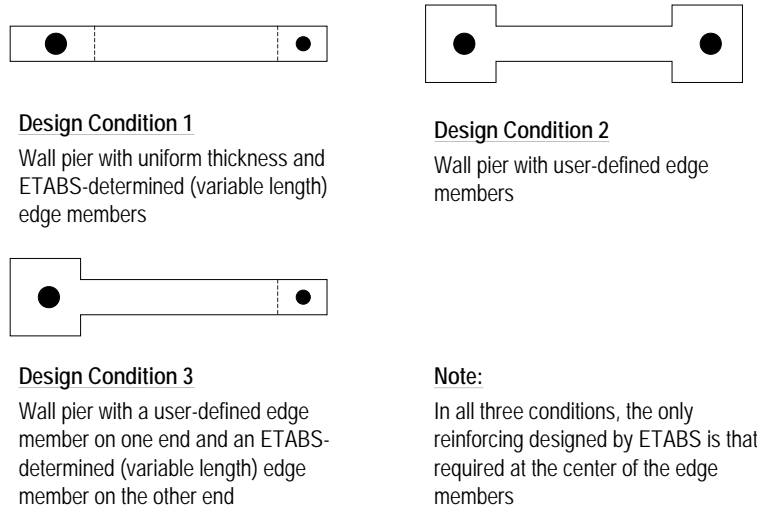


Figure 2-2: Design Conditions for Simplified Wall Piers

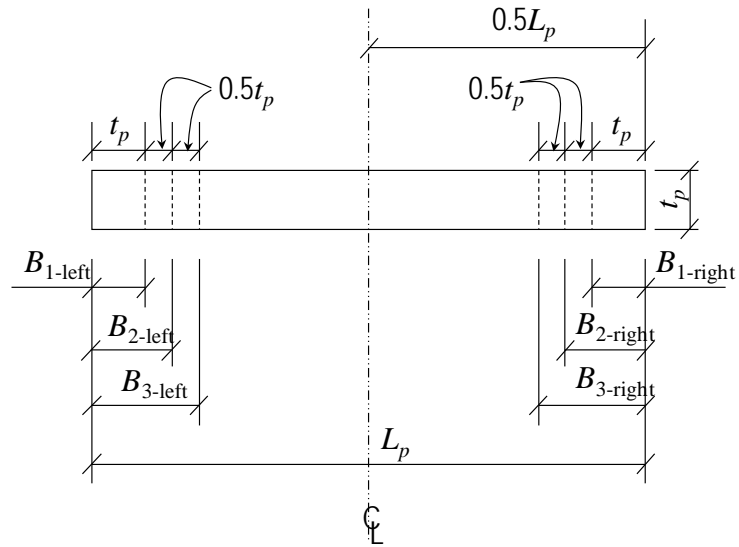
2.1.1.1 Design Condition 1

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.

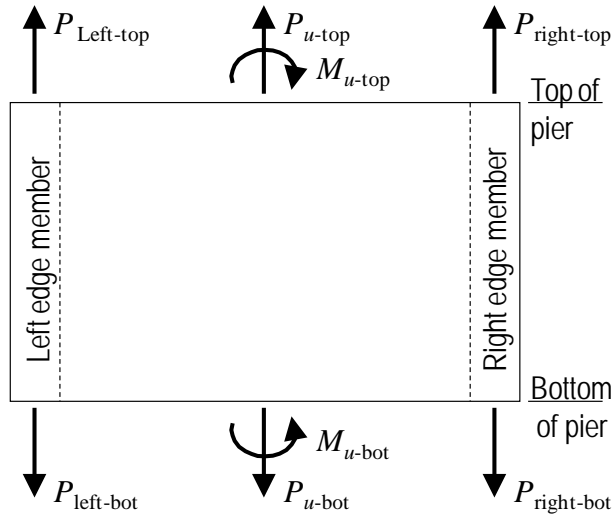
Consider the wall pier shown in Figure 2-3. For a given design section, say the top of the wall pier, the wall pier for a given design load combination is designed for a factored axial force P_{f-top} and a factored moment M_{f-top} .

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-left} , and an edge member at the right end of the wall of thickness t_p and width $B_{1-right}$. Initially $B_{1-left} = B_{1-right} = t_p$.

The moment and axial force are converted to an equivalent force set $P_{left-top}$ and $P_{right-top}$ using the relationships shown in the following equations. (Similar equations apply at the bottom of the pier.)



Wall Pier Plan



Wall Pier Elevation

Figure 2-3: Wall Pier for Design Condition 1

$$P_{\text{left-top}} = \frac{P_{f\text{-top}}}{2} + \frac{M_{f\text{-top}}}{(L_p - 0.5B_{1\text{-left}} - 0.5B_{1\text{-right}})}$$

$$P_{\text{right-top}} = \frac{P_{f\text{-top}}}{2} - \frac{M_{f\text{-top}}}{(L_p - 0.5B_{1\text{-left}} - 0.5B_{1\text{-right}})}$$

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:

$$A_{st} = \frac{P}{\phi_s f_y}. \quad (\text{CSA 8.5.3.2(b)})$$

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.

$$Abs(P) = (P_{\text{max Factor}}) [\alpha_1 \phi_c f'_c (A_g - A_{sc}) + \phi_s f_y A_{sc}] \quad (\text{CSA 10.10.4(b)})$$

where P is either $P_{\text{left-top}}$ or $P_{\text{right-top}}$, $A_g = t_p B_l$ and the $P_{\text{max Factor}}$ is defined in the shear wall design preferences (the default is 0.80). In general, we recommend the default value. From the preceding equation,

$$A_{sc} = \frac{\frac{Abs(P)}{(P_{\text{max Factor}}) \phi_c} - \alpha_1 \phi_c f'_c A_g}{\phi_s f_y - \alpha_1 \phi_c f'_c}.$$

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

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

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

Similarly, the compression reinforcing is limited by:

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

If A_{st} is less than or equal to $A_{st-\max}$ and A_{sc} is less than or equal to $A_{sc-\max}$, the program will proceed to check the next loading combination; otherwise the program will increment the appropriate B_l dimension (left, right or both, depending on which edge member is inadequate) by one-half of the wall thickness to B_2 (i.e., $1.5t_p$) 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.

2.1.1.2 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.

2.1.1.3 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.

2.1.2 Checking a General or Uniform Reinforcing Pier Section

When a General Reinforcing or Uniform Reinforcing pier section is specified 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 PMM interaction curves that are equally spaced around a 360-degree circle.

2.1.2.1 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 PMM curves are specified (the default), there is one curve every 15 degrees ($360^\circ/24 \text{ curves} = 15^\circ$). Figure 2-4 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.

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

Each PMM 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 later in the section entitled *Details of the Strain Compatibility Analysis*.

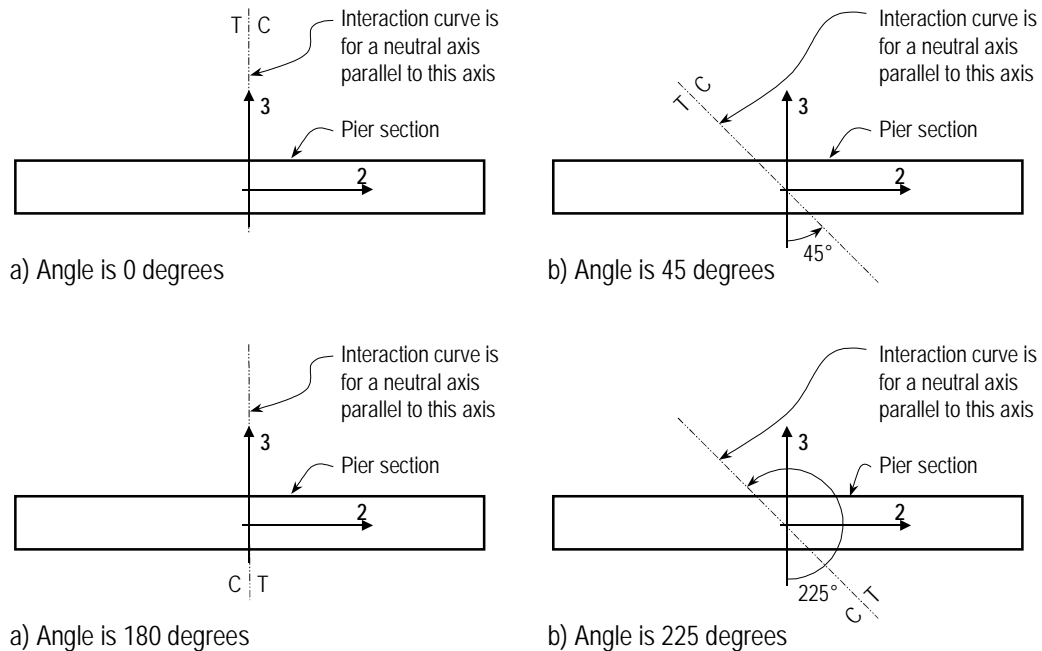


Figure 2-4: Orientation of the Pier Neutral Axis for Various Angles

By default, 11 points are used to define a PMM 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.

2.1.2.2 Formulation of the Interaction Surface

The formulation of the interaction surface in this program is based consistently on the basic principles of ultimate strength design given in Sections 10.1 and 10.10 of CSA-A23.3-04. The program uses the requirements of force equilibrium and strain compatibility to determine the nominal axial load and moment

resistance (P_r, M_{2r}, M_{3r}) of the wall pier. This nominal strength is then multiplied by the appropriate strength reduction factor ϕ_c and ϕ_s to obtain the design resistance of the pier. For the pier to be deemed adequate, the required strength (P_f, M_{2f}, M_{3f}) must be less than or equal to the design strength.

$$(P_f, M_{2f}, M_{3f}) \leq (P_r, M_{2r}, M_{3r})$$

The effects of the strength reduction factors, ϕ_c and ϕ_s , are included in the generation of the interaction curve.

ϕ_c = Strength reduction factor for concrete. The default value is 0.65 (CSA 8.4.2).

ϕ_s = Strength reduction factor for reinforcing steel. The default value is 0.85 (CSA 8.4.3).

The theoretical maximum compressive force that the wall pier can carry is designated $P_{r,max}$ and is given by the following equation:

$$P_{r,max} = 0.8[\alpha_1\phi_c f'_c (A_g - A_s) + \phi_s f_y A_s] \quad (\text{CSA 10.10.4})$$

The theoretical maximum tension force that the wall pier can carry is designated $P_{t,max}$ and is given by the following equation:

$$P_{t,max} = \phi_s f_y A_s \quad (\text{CSA 10.10.4})$$

If the wall pier geometry and reinforcing are symmetrical in plan, the moments associated with both $P_{r,max}$ and $P_{t,max}$ are zero. Otherwise, there will be a moment associated with both $P_{r,max}$ and $P_{t,max}$.

In addition to $P_{r,max}$ and $P_{t,max}$, the axial load at the balanced strain condition, i.e., P_b , is also determined. In this condition, the tension reinforcing reaches the strain corresponding to its specified factored yield strength, $\phi_s f_y$, just as the concrete reaches its assumed ultimate strain of 0.0035 (CSA 10.1.4).

Note that $P_{r,max}$ is reduced not only by the strength reduction factors but also by an additional factor of 0.80. In the preferences, this factor is called the P_{max} Factor, and its value can be changed as necessary. In all CSA-A23.3.04 code designs, it is prudent to consider this factor to be 0.80, as required by the code.

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 , $P_{r,max}$ and $P_{t,max}$ on the interaction curve. Half of the remaining number of specified points on the interaction curve occur between P_b and $P_{r,max}$ at approximately equal spacing along the P_r axis. The other half of the remaining number of specified points on the interaction curve occur between P_b and $P_{t,max}$ at approximately equal spacing along the P_r axis.

Figure 2-5 shows a plan view of an example two-dimensional wall pier. Notice that the concrete is symmetrical but the reinforcing is not symmetrical in this example. Figure 2-6 shows several interaction surfaces for the wall pier illustrated in Figure 2-5.

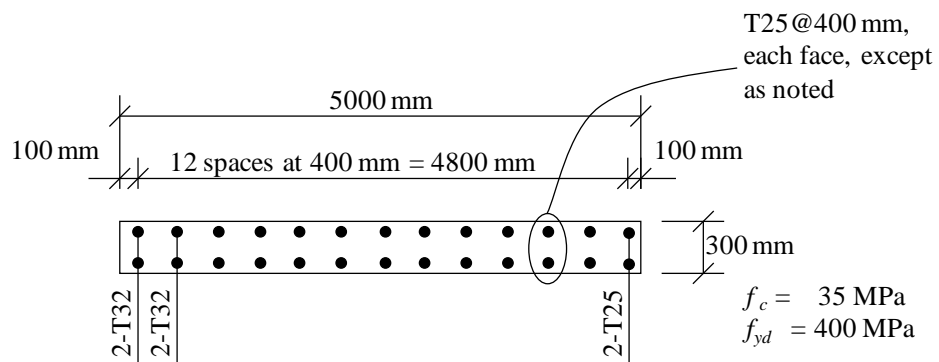


Figure 2-5: Example Two-Dimensional Wall Pier With Unsymmetrical Reinforcing

Note the following about Figure 2-6:

- Because the pier is two-dimensional, the interaction surface consists of two interaction curves. One curve is at 0° and the other is at 180° . Only M3 moments are considered because this is a two-dimensional example.
- In this program, compression is negative and tension is positive.
- The 0° and 180° interaction curves are not symmetric because the wall pier reinforcing is not symmetric.

- The smaller interaction surface (drawn with a heavier line) has both the strength reduction factors and the P_{\max} Factor, as specified by CSA-A23.3-04.
- The dashed line shows the effect of setting the P_{\max} Factor to 1.0.
- The larger interaction surface has both the strength reduction factor and the P_{\max} Factor set to 1.0.
- The interaction surfaces shown are created using the default value of 11 points for each interaction curve.

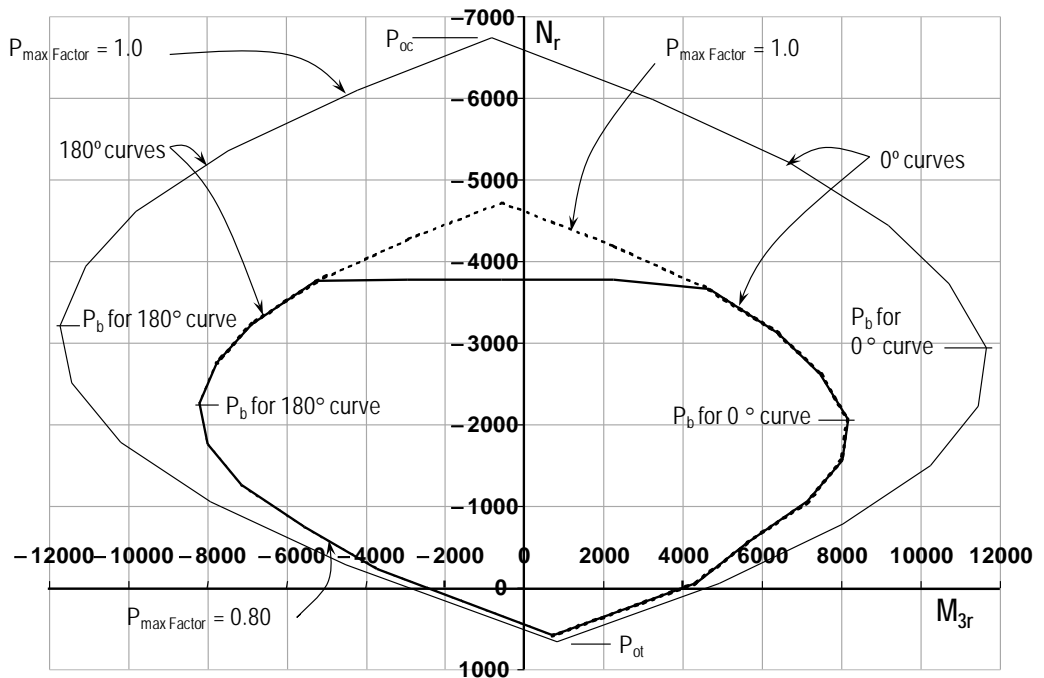


Figure 2-6 : Interaction Curves for Example Wall Pier Shown in Figure 2-5

Figure 2-7 shows the 0° interaction curves for the wall pier illustrated in Figure 2-5. Additional interaction curves are also added to Figure 2-7.

The smaller, heavier curve in Figure 2-7 has the strength reduction factor and the P_{\max} Factor as specified in CSA-A23.3-04. The other three curves, which are plotted with ϕ factors as 1.0, all have P_{\max} Factors of 1.0. The purpose of showing these interaction curves is to explain how the program creates the interaction

curve. Recall that the strength reduction factors 0.65 and 0.85 are actually ϕ_c and ϕ_s , and that their values can be revised in the overwrites as required.

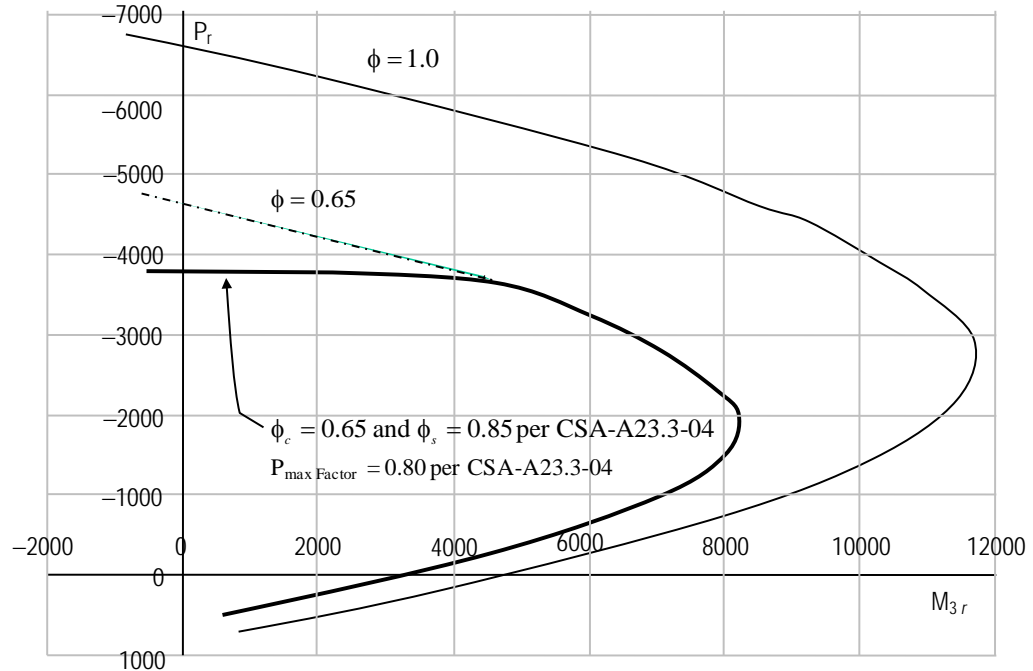


Figure 2-7: Interaction Curves for Example Wall Pier Shown in Figure 2-5

2.1.2.3 Details of the Strain Compatibility Analysis

As previously mentioned, the program uses the requirements of force equilibrium and strain compatibility to determine the nominal axial load and moment strength (P_r , M_{2r} , M_{3r}) of the wall pier. The coordinates of these points are determined by rotating a plane of linear strain on the section of the wall pier.

Figure 2-8 illustrates varying planes of linear strain such as those that the program considers on a wall pier section for a neutral axis orientation angle of 0 degrees.

In these planes, the maximum concrete strain is always taken as -0.0035 and the maximum steel strain is varied from -0.0035 to plus infinity. (Recall that in this program compression is negative and tension is positive.) When the steel strain is -0.0035 , the maximum compressive force in the wall pier, P_{oc} , is ob-

tained from the strain compatibility analysis. When the steel strain is plus infinity, the maximum tensile force in the wall pier, P_{ot} , is obtained. When the maximum steel strain is equal to the yield strain for the reinforcing, P_b is obtained.

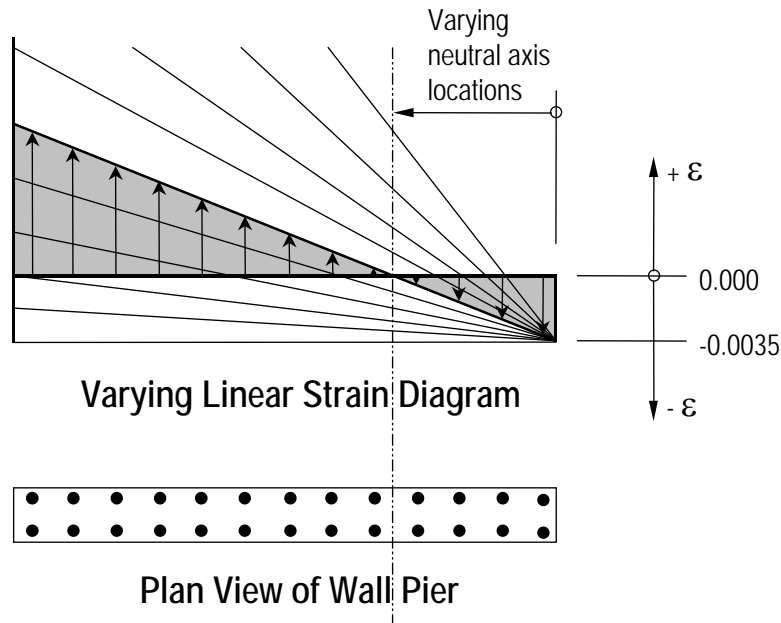


Figure 2-8: Varying Planes of Linear Strain

Figure 2-9 illustrates the concrete wall pier stress-strain relationship that is obtained from a strain compatibility analysis of a typical plane of linear strain ϕ_s shown in Figure 2-10. In Figure 2-9 the compressive stress in the concrete, C_c , is calculated (CSA 10.1.7).

$$C_c = (\alpha_1 \phi_c f'_c) \beta_1 c t_p \quad (\text{CSA 10.1.7})$$

In Figure 2-8, the value for maximum strain in the reinforcing steel is assumed. Then the strain in all other reinforcing steel is determined based on the assumed plane of linear strain. Next the stress in the reinforcing steel is calculated as follows, where ϵ_s is the strain, E_s is the modulus of elasticity, σ_s is the stress, and f_y is the yield stress of the reinforcing steel.

$$\sigma_s = \epsilon_s E_s \leq \phi_s f_y \quad (\text{CSA 8.5.3.2})$$

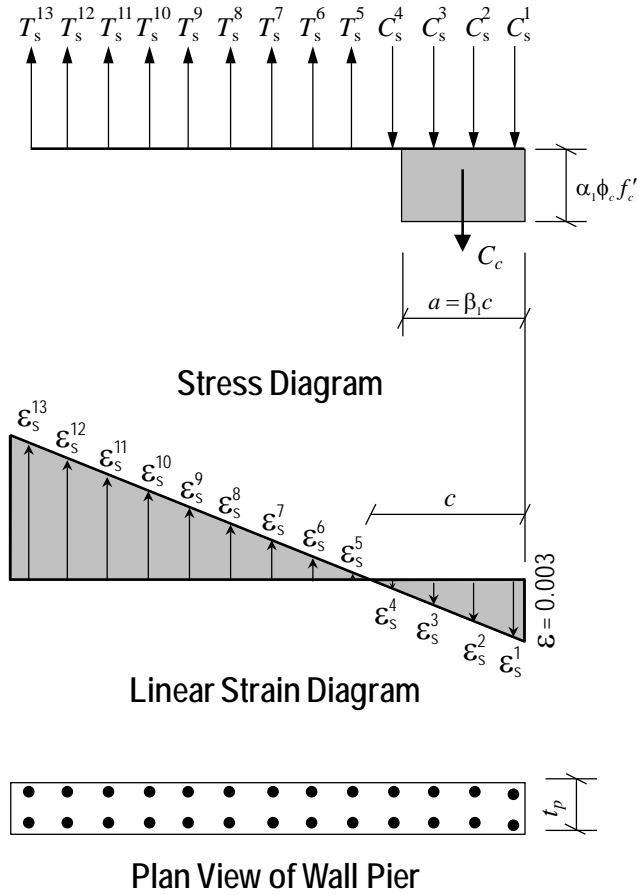


Figure 2-9: Wall Pier Stress-Strain Relationship

The force in the reinforcing steel (T_s for tension or C_s for compression) is calculated by:

$$T_s \text{ or } C_s = \sigma_s A_s$$

For the given distribution of strain, the value of P_r is calculated by.

$$P_r = \phi(\sum T_s - C_c - \sum C_s) \leq P_{\max}$$

$$P_r \leq P_{o,\max} \text{ (if compression)}$$

$$P_r \leq P_{ot,\max} \text{ (if tension)}$$

In the preceding equation, the tensile force T_s and the compressive forces C_c and C_s are all positive. If P_r is positive, it is tension, and if it is negative, it is compression. The terms $P_{oc,max}$ and $P_{ot,max}$ are calculated according to CSA Section 10.10.4. The appropriate expression of these two terms was provided previously.

The value of M_2 is calculated by summing the moments resulting from all of the forces about the pier local 2-axis. Similarly, the value of M_3 is calculated by summing the moments resulting from all of the forces about the pier local 3-axis. The forces whose moments are summed to determine M_{2r} and M_{3r} are C_c , all of the T_s forces and all of the C_s forces.

The P_r , M_{2r} and M_{3r} values calculated as described previously make up one point on the wall pier interaction diagram. Additional points on the diagram are obtained by making different assumptions for the maximum steel stress; that is, considering a different plane of linear strain, and repeating the process.

When one interaction curve is complete, the next orientation of the neutral axis is assumed and the points for the associated new interaction curve are calculated. This process continues until the points for all of the specified curves have been calculated.

2.1.3 Wall Pier Demand/Capacity Ratio

Refer to Figure 2-10, which shows a typical two-dimensional wall pier interaction diagram. The forces obtained from a given design load combination are P_u and M_{3u} . The point L, defined by (P_f, M_{3f}) , 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.

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. The 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 demand/capacity ratio:

- If $OL = OC$ (or $D/C = 1$), the point (P_f, M_{3f}) lies on the interaction curve and the wall pier is stressed to capacity.
- If $OL < OC$ (or $D/C < 1$), the point (P_f, M_{3f}) lies within the interaction curve and the wall pier capacity is adequate.
- If $OL > OC$ (or $D/C > 1$), the point (P_f, M_{3f}) lies outside of the interaction curve and the wall pier is overstressed.

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

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

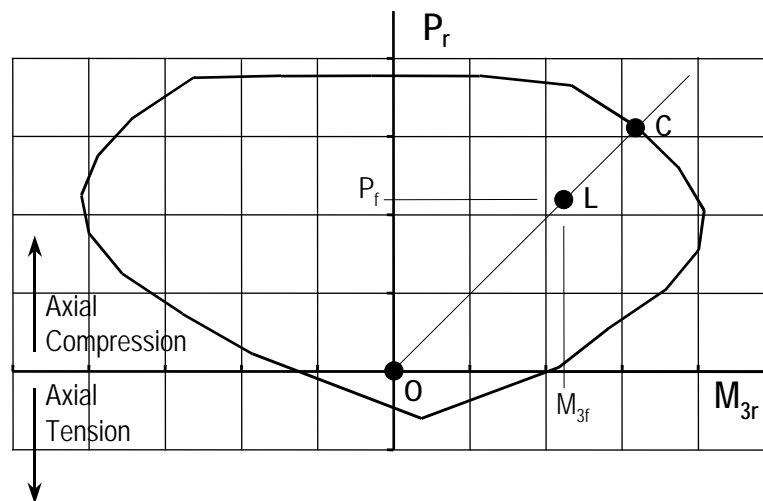


Figure 2-10: Two-Dimensional Wall Pier Demand/Capacity Ratio

2.1.4 Designing a General Reinforcing 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 2-1 illustrates the spacing, both in general terms and for a specific example, when the minimum reinforcing ratio, IPmin, is 0.0025 and the maximum, IPmax, is 0.02.

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

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

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 the section entitled *Checking a General or Uniform Reinforcing Pier Section*.

Next, for a given design load combination, the program generates a demand/capacity 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 demand/capacity ratio of 1 (actually the program uses 0.99 instead of 1). 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.

2.2 Wall Pier Shear Design

The wall pier shear reinforcing is designed for each of the design load combinations. The following steps are involved in designing the shear reinforcing for a particular wall pier section for a particular design loading combination.

- Determine the factored forces P_f , M_f and V_f that are acting on the wall pier section. Note that P_f and M_f are required for the calculation of V_c .

In addition to designing for factored shear force, each leg of the shear wall is designed for enhanced factored seismic loading. In the enhanced factored forces, the seismic load factors are multiplied by force modification factors (CSA 21.6.9.1, 21.7.3.4.1) provided in Shear Wall Design Preferences. The force modification factor (CSA 21.6.9.1, 21.7.3.4.1) refers to shear corresponding to the development of the probable moment capacity of the wall system at its plastic hinge location. This is a user specified factor in the program and a default value of 2.0 is used.

The program does not amplify the shear resulting from design load combinations that include earthquake, with load effects calculated using R_d and R_o ((21.6.9.1(b), 21.7.3.4.1(b)) as this condition does not govern for shear design. The program also ignores the magnification of the shear due to inelastic effects of higher modes.

The ductility related force modifications factor, R_d and overstrength related force modification factor, R_o are used to compute inelastic rotational demand (CSA 21.6.7.2, 21.6.8.2 and 21.7.3.2). The inelastic rotational demand is used for computing the β factor.

The ductility related force modifications factor, R_d and overstrength related force modification factor, R_o reflect the capacity of a structure to dissipate energy through inelastic behavior. As given in the National Building Code of Canada, the value of R_d and R_o are taken as follows (CSA 21.6, NBCC 4.1.8.9, Table 4.1.8.9)

for Ordinary wall	$R_d = 1.0$ and $R_o = 1.0$
for Ductile Flexural wall ($h_w/l_p > 2.0$)	$R_d = 3.5$ or 4.0
for Ductile Shear wall	$R_d = 3.5$ and $R_o = 1.6$
for Ductile Coupled wall	$R_d = 4.0$ and $R_o = 1.7$
for Ductile Partially Coupled wall	$R_d = 3.5$ and $R_o = 1.7$
for Moderately Ductile Shear wall ($h_w/l_p > 2.0$)	$R_d = 2.0$ and $R_o = 1.4$

- Determine the shear force, V_c , that can be carried by the concrete.
- Determine the required shear reinforcing to carry the balance of the shear force.

Step 1 needs no further explanation. The following two sections describe in detail the algorithms associated with the Steps 2 and 3.

2.2.1 Determine the Concrete Shear Capacity

For designing ordinary shear wall or any other type of wall for nonseismic load, V_c is calculated as follows:

$$V_c = \phi_c \lambda \beta \sqrt{f'_c} b_w d, \quad (\text{CSA 11.3.4})$$

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

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

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

β is the factor for accounting for the shear resistance of cracked concrete (CSA 2.2). Its value is normally between 0.1 and 0.4. It is determine in accordance with section 11.3.6.4 of the Code, which is described in the sections that follow.

t_p is the thickness of wall pier resisting the shear perpendicular to the shear force direction.

d_v is the effective shear depth. It is conservatively taken to be $0.8L_p$.

$$d_v = 0.8 L_p \quad (\text{CSA 11.0})$$

The value of the β factor is determined using the General method (CSA 11.3.6.4).

$$\beta = \frac{0.40}{(1 + 1500\epsilon_x)} \bullet \frac{1300}{(1000 + S_{ze})} \quad (\text{CSA 11.3.6.4})$$

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

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

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

$$\varepsilon_x = \frac{M_f/d_v + V_f + 0.5N_f}{2(E_s A_s)} \quad (\text{CSA 11.3.6.4})$$

In evaluating the ε_x , the following conditions apply:

$$V_f \text{ and } M_f \text{ are taken as positive quantities} \quad (\text{CSA 11.3.6.4a})$$

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

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

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

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

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

An upper limit on ε_x is imposed as follows:

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

ε_x is positive for tensile action.

N_f is positive for tensile action.

In the preceding equation, d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.8 L_p$.

$$d_v = 0.8 L_p \quad (\text{CSA 11.0})$$

The term λ that is used as a multiplier on all $\sqrt{f'_c}$ terms in this chapter is a shear strength reduction factor that applies to light-weight concrete (CSA 11.0, CSA 8.6.5). It is equal to 1 for normal weight concrete. This factor is specified in the concrete material properties.

- For designing Ductile Flexural walls, Ductile Coupled walls, and Ductile Partially Coupled walls subjected to seismic loads, the following additional clauses are checked by program:

- (i) When the inelastic rotational demand on the wall, $\theta_{id} \leq 0.005$

$$V_f \leq 0.15\phi_c f'_c t_p d_v \quad (\text{CSA 21.6.9.6 (a)})$$

$$\beta = 0.18 \quad (\text{CSA 21.6.9.6 (b)})$$

- (ii) When the inelastic rotational demand on the wall, $\theta_{id} \geq 0.015$

$$V_f \leq 0.10\phi_c f'_c t_p d_v \quad (\text{CSA 21.6.9.6 (a)})$$

$$\beta = 0 \quad (\text{CSA 21.6.9.6 (b)})$$

- (iii) A linear interpolation is used for determining the V_f and β when the inelastic rotational demand, θ_{id} , on the wall is between 0.005 to 0.015.

- (iv) The value of θ is taken as 45 degree.

With those modification, the shear design calculation proceeds in the same way as that for Ordinary shear walls.

For designing shear walls with Moderately Ductile Shear walls subjected to seismic loads, V_c is computed based on the assumption that

$$\beta = 0.1 \text{ and } \theta = 45 \text{ degrees} \quad (\text{CSA 21.73.4.2})$$

Otherwise the procedure for computing V_c is the same as that for Ordinary moment resisting frames (CSA 11.4).

2.2.2 Determine the Required Shear Reinforcing

Given V_f and v_c , the following procedure provides the required shear reinforcing in area per unit length.

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

$$v = \frac{V_f}{t_p d_v}$$

In the preceding equation, d_v , the distance between the resultants of the tensile and compressive forces, is conservatively taken to be $0.8 L_p$.

$$d_v = 0.8 L_p \quad (\text{CSA 11.0})$$

- The average shear stress, V_f , is limited to a maximum limit, $V_{r,\max}$, given by

$$V_{r,\max} = 0.25 \phi_c f'_c \quad (\text{CSA 11.3.3})$$

The shear reinforcement per unit spacing is computed as follows:

- If $V_f \leq V_c$,

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

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

$$\frac{A_v}{s} = \frac{(V_f - V_c) \tan \theta}{\phi_s f_y d_v} \quad (\text{CSA 11.3.3, 11.3.5.1})$$

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

a failure condition is declared. (CSA 11.3.3)

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

$$\frac{A_v}{s} \geq \frac{\sqrt{f'_c}}{f_y} b_w \quad (\text{CSA 11.2.8.2})$$

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

Similar to the β factor, which was described previously, the value of θ is preferably determined using the Simplified method (CSA 11.3.6.3), whenever applicable. The program uses the General method when the conditions for the Simplified methods are not satisfied (CSA 11.3.6.4). For designing concrete column sections for shear forces, the special value of θ does not apply (CSA 11.3.6.2).

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

$$\theta = 35^\circ \text{ for } P_f \leq 0 \text{ or } f_y \leq 400 \text{ MPa or } f'_c \leq 60 \text{ MPa.} \quad (\text{CSA 11.3.6.4})$$

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

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

$$\text{for } P_f < 0 \text{ or } f_y > 400 \text{ MPa or } f'_c > 60 \text{ MPa} \quad (\text{CSA 11.3.6.4})$$

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

The maximum of all the calculated A_v/s values, obtained from each load combination, is reported for each leg of the wall along with the controlling shear force and associated load combination number. The output units for the distributed shear reinforcing can be set in the shear wall design preferences.

2.3 Wall Pier Boundary Elements

This section describes how the program considers the boundary element requirements for each leg of concrete wall piers using CSA-A23.3-04 when the Ductile Flexural wall, Ductile Coupled wall, Ductile Partially Coupled wall or Moderately Ductile Shear wall option is chosen. The program uses an approach based on the requirements of Section 21.6.7, 21.6.8 and 21.7.3.2 of CSA-A23.3-04. The program does not compute boundary zone requirement when inelastic rotational demand exceeds the inelastic rotational capacity of the wall.

Note that the boundary element requirements are considered separately for each design load combination that includes seismic load.

2.3.1 Details of Check for Boundary Element Requirements

The following information is available for the boundary element check:

- The design forces P_f , V_f , and M_f for the pier section.
- The height of the entire wall, h_w , length of the wall pier, L_p , the gross area of the pier, A_g , and the net area of the pier, A_{cv} . The net area of the pier is the area bounded by the web thickness, t_p , and the length of the pier. (Refer to Figure 2-3 earlier in this chapter for an illustration of the dimensions L_p and t_p .)
- The program also computes the design displacement Δ_f by multiplying the displacement from a load combination with the $R_d R_o$ factor provided in the shear wall design preferences (Appendix C).
- The area of reinforcement in the pier, A_s . This area of steel is calculated by the program or it is provided by the user.
- The material properties of the pier, f'_c and f_y .
- The symmetry of the wall pier (i.e., is the left side of the pier the same as the right side of the pier). Only the geometry of the pier is considered, not the re-

inforcing, when determining if the pier is symmetrical. Figure 2-11 shows some examples of symmetrical and unsymmetrical wall piers. Note that a pier defined using Section Designer is assumed to be unsymmetrical, unless it is made up of a single rectangular shape.

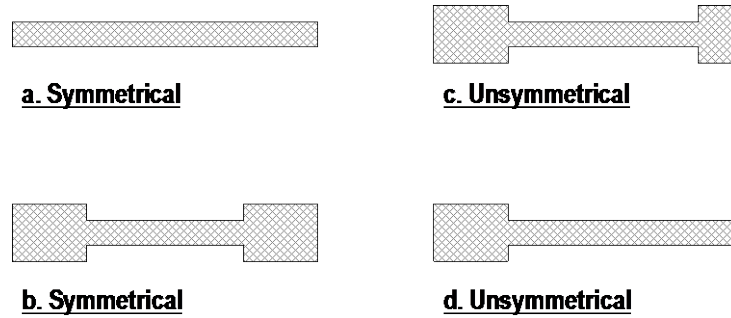


Figure 2-11 Example Plan Views of Symmetrical and Unsymmetrical Wall Piers

Using this information, the program calculates the inelastic rotation demand, θ_{id} , as follows:

- (i) for Ductile Flexural wall (CSA 21.6.7.2)

$$\theta_{id} = \frac{(\Delta_f R_d R_o - \Delta_f \gamma_w)}{h_w - \frac{l_w}{2}} \geq 0.004$$

- (ii) for Ductile Coupled and partially Coupled wall (CSA 21.6.8.2)

$$\theta_{id} = \frac{\Delta_f R_d R_o}{h_w} \geq 0.004$$

- (iii) for Moderately Ductile Flexural wall (CSA 21.7.3.2)

$$\theta_{id} = \frac{(\Delta_f R_d R_o - \Delta_f \gamma_w)}{h_w - \frac{l_w}{2}} \geq 0.003$$

where,

$$\Delta_f R_d R_o = \text{the design displacement}$$

$\Delta_f \gamma_w$ = the elastic portion of displacement. γ_w is taken as 1.3.

l_w = length of the wall

The inelastic rotational capacity of the wall, θ_{ic} , for Ductile Flexural wall, Ductile Coupled and Ductile Partially coupled and Moderately Ductile shear wall (CSA 21.6.7.3, 21.6.8.3, 21.7.3.2) is computed as follows:

$$\theta_{ic} = \left(\frac{\varepsilon_{cu} l_w}{2c} - 0.002 \right) \leq 0.025 \quad (\text{CSA 21.6.7.3})$$

where,

ε_{cu} is taken as 0.0035.

The depth of neutral axis, c , is determine using CSA Eqn. 21-12.

$$c = \frac{P_s + P_n + P_{ns}}{\alpha_1 \beta_1 \phi_c f_c' t_p} \quad (\text{CSA 21.6.7.3})$$

If boundary elements are required, the program calculates the minimum required length of the boundary zone at each end of the wall, L_{BZ} , in accordance with the requirements of Section 21.6.6.4 in CSA-A23.3-04. The code requires that L_{BZ} shall not be less than $0.0015l_w$ (CSA 21.6.6.4). Figure 2-12 illustrates the boundary zone length L_{BZ} .

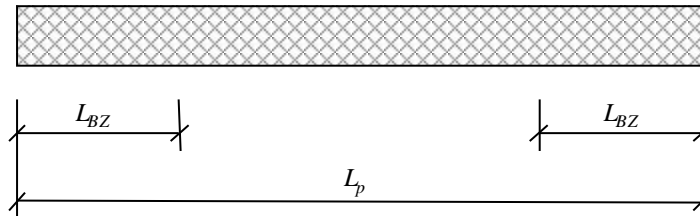


Figure 2-12: Illustration of Boundary Zone Length, L_{BZ}

Chapter 3 Spandrel Design

This chapter describes how the program designs concrete shear wall spandrels for flexure and shear when CSA-A23.3-04 is the selected design code. 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 the center (mid-length) of the spandrel. The program does not allow shear reinforcing to be specified and then checked. The program only designs the spandrel for shear and reports how much shear reinforcing is required.

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 spandrels 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 only 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_f .
- Determine the required flexural reinforcing.

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 first obtained.

The beam section is then 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 in the spandrel design overwrites (Appendix C).

It is assumed that the compression carried by the concrete is less than that which can be carried at the balanced condition (CSA 10.1.4). When the applied moment exceeds the moment capacity at the balanced condition, the program calculates an area of compression reinforcement assuming that the additional moment is carried by compression reinforcing and additional tension reinforcing.

In designing the spandrel flexural reinforcement, the limit of f_c' is taken to be 80 MPa for Ordinary, Ductile Coupled, and Ductile Partially Coupled spandrel walls.

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

$$f_c' \leq 80 \text{ MPa (Ductile Coupled, and Ductile Partially Coupled spandrel walls.)} \quad (\text{CSA 21.2.6.2})$$

The limit of f_y is taken to be 500 MPa for Ordinary spandrels and 400 MPa for Ductile Coupled and Ductile Partially Coupled spandrel walls.

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

$$f_y \leq 400 \text{ MPa (Ductile Coupled, and Ductile Partially Coupled spandrel walls.)} \quad (\text{CSA 21.2.7.1})$$

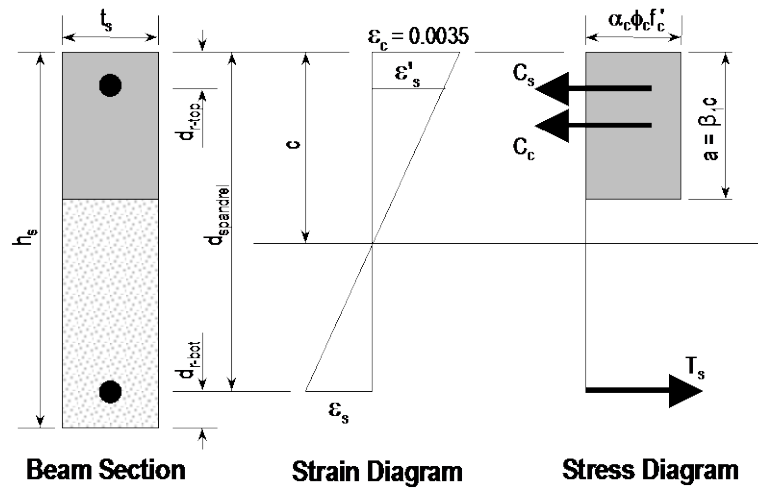


Figure 3-1 Rectangular Spandrel Beam Design, Positive Moment

The procedure used by the program for both rectangular and T-beam sections is given in the subsections that follow.

3.1.2.1 Rectangular Beam Flexural Reinforcing

Refer to Figure 3-1. For a rectangular beam, the factored moment, M_u , is resisted by a couple between the concrete in compression and the tension in reinforcing steel. This is expressed as follows:

$$M_f = C_c \left(d_{\text{spandrel}} - \frac{a}{2} \right)$$

where $C_c = \alpha_1 \phi_c f_c' a t_s$ and d_{spandrel} is equal to $h_s - d_{\text{r-bot}}$ for positive bending and $h_s - d_{\text{r-top}}$ for negative bending.

In designing for a factored negative or positive moment, M_u (i.e., designing top or bottom steel), the depth of the compression block is given by a :

$$a = d_{\text{spandrel}} - \sqrt{d_{\text{spandrel}}^2 - \frac{2M_f}{\alpha_1 \phi_c f_c' t_s}} \quad (\text{CSA 10.1})$$

where the value of ϕ_c is 0.65 (CSA 8.4.2) in these equations. Also α_1 is calculated as follows:

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

The program uses the previous equation to determine the depth of the compression block, a .

The depth of the compression block, a , is compared with β_1 (CSA 10.1.7), where,

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

c_b , the distance from the extreme compression fiber to the neutral axis for balanced strain conditions, is given by the following equation:

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

Note: If the required tension reinforcing exceeds the balanced reinforcing, the program provides compression steel to help resist the applied moment.

3.1.2.1.1 Tension Reinforcing Only Required

- If $a \leq \beta_1 c_b$ (CSA 10.5.2), the area of tensile steel reinforcement is then given by

$$A_s = \frac{M_f}{\phi_s f_y \left(d_{\text{spandrel}} - \frac{a}{2} \right)}$$

The steel is placed at the bottom for positive moment and at the top for negative moment.

Note: The program reports the ratio of top and bottom steel required in the web area. When compression steel is required, those ratios may be large because there is no limit on them. However, the program reports an overstress when the ratio exceeds 4%.

3.1.2.1.2 Tension and Compression Reinforcing Required

- If $a > \beta_1 c_b$ (CSA 10.5.2), compression reinforcement is required and is calculated as follows.

The compressive force developed in concrete alone is given by

The depth of the concrete compression block, a , is set equal to $a_b = \beta_1 c_b$. The compressive force developed in the concrete alone is given by:

$$C_c = \alpha_1 \phi_c f'_c t_s a_b. \quad (\text{CSA 10.17})$$

The moment resisted by the couple between the concrete in compression and the tension steel, M_{fc} , is given by:

$$M_{fc} = C_c \left(d_{\text{spandrel}} - \frac{a_b}{2} \right).$$

Therefore, the additional moment to be resisted by the couple between the compression steel and the additional tension steel, M_{fs} , is given by:

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

The force carried by the compression steel, C_s , is given by:

$$C_s = \frac{M_{fs}}{d_{\text{spandrel}} - d_r}.$$

Referring to Figure 3-1, the strain in the compression steel, ϵ'_s , is given by:

$$\epsilon'_s = \frac{0.0035(c - d_r)}{c}. \quad (\text{CSA 8.5.3.2, CSA 10.1.3})$$

The stress in the compression steel, f'_s , is given by

$$f'_s = E_s \varepsilon'_s = \frac{0.0035 E_s (c - d_r)}{c} \leq f_y \quad (\text{CSA 8.5.3.2, CSA 10.1.3})$$

The term d_r in the preceding equations is equal to $d_{r\text{-top}}$ for positive bending and equal to $d_{r\text{-bot}}$ for negative bending. The term c is equal to c_b (CSA 10.1.7).

The total required area of compression steel, A'_s , is calculated using the following equation.

$$A'_s = \frac{C_s}{\left(\phi_s f'_s - \phi_c \alpha_1 f'_c \right)}$$

The required area of tension steel for balancing the compression in the concrete web, A_{sw} , is:

$$A_{sw} = \frac{M_{fc}}{\phi_s f_y \left(d_{\text{spandrel}} - \frac{a_{\text{max}}}{2} \right)}$$

The required area of tension steel for balancing the compression steel, A_{sc} , is:

$$A_{sc} = \frac{M_{fs}}{\phi_s f_y (d_{\text{spandrel}} - d_r)}$$

In the preceding Equations, d_{spandrel} is equal to $h_s - d_{r\text{-bot}}$ for positive bending and $h_s - d_{r\text{-top}}$ for negative bending. d_r is equal to $d_{r\text{-top}}$ for positive bending and $d_{r\text{-bot}}$ for negative bending.

The total tension reinforcement A_s is given by:

$$A_s = A_{sw} + A_{sc}$$

The total tension reinforcement A_s is to be placed at the bottom of the beam and total compression reinforcement A'_s at the top for positive bending and vice versa for negative bending.

3.1.2.2 T-Beam Flexural Reinforcing

T-beam action is considered effective for positive moment only. When designing T-beams for negative moment (i.e., designing top steel), the calculation of required steel is as described in the previous section for rectangular sections. No T-beam data is used in this design. The width of the beam is taken equal to the width of the web, t_s .

For positive moment, the depth of the compression block, a , and the method for calculating the required reinforcing steel relates the compression block depth, a , is previously described in Section 3.1.2, to the depth of the T-beam flange, d_s . See Figure 3-2.

- If $a \leq d_s$, the subsequent calculations for the reinforcing steel are exactly the same as previously defined for rectangular section design. However, in that case, the width of the compression block is taken to be equal to the width of the compression flange, b_s . Compression reinforcement is provided when the dimension " a " exceeds $\beta_1 c_b$, where β_1 and c_b are given by previous equations for rectangular beam.
- If $a > d_s$, the subsequent calculations for the required area of reinforcing steel are performed in two parts. First, the tension steel required to balance the compressive force in the flange is determined, and second, the tension steel required to balance the compressive force in the web is determined. If necessary, compression steel is added to help resist the design moment.

The remainder of this section describes in detail the design process used by the program for T-beam spandrels when $a > d_s$.

Refer to Figure 3-2. The protruding portion of the flange is shown cross-hatched. The compression force in the protruding portion of the flange, C_f , is given by:

$$C_f = \alpha_1 f_c' (b_s - t_s) d_s$$

Note: T-beam action is considered for positive moment only.

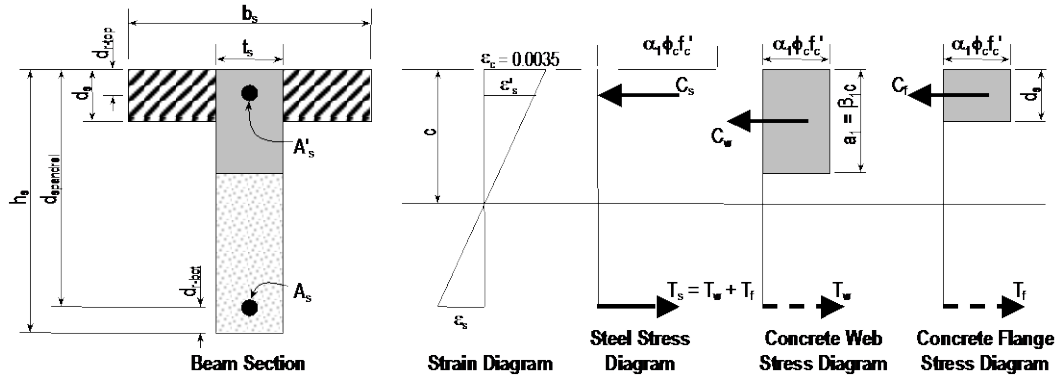


Figure 3-2: Design of a Wall Spandrel with a T-Beam Section, Positive Moment

The required area of tension steel for balancing the compression force in the concrete flange, A_{sf} , is:

$$A_{sf} = \frac{\phi_c C_f}{\phi_s f_y}$$

The portion of the total moment, M_u , that is resisted by the flange, M_{ff} , is given by:

$$M_{ff} = \phi_c C_f \left(d_{\text{spandrel}} - \frac{d_s}{2} \right)$$

Therefore, the balance of the moment to be carried by the web, M_{fw} , is given by:

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

The web is a rectangular section of width t_s and depth h_s for which the design depth of the compression block, a_1 , is recalculated as:

$$a_1 = d_{\text{spandrel}} - \sqrt{d_{\text{spandrel}}^2 - \frac{2M_{fw}}{\alpha_1 \phi_c f'_c t_s}} \quad (\text{CSA 10.1})$$

3.1.2.2.1 Tension Reinforcing Only Required

If $a_1 \leq \beta_1 c_b$ (CSA 10.5.2), no compression reinforcing is required and the program calculates the area of tension steel for balancing the compression force in the concrete web, A_{sw} , using the following equation.

$$A_{sw} = \frac{M_{fw}}{\phi_s f_y \left(d_{\text{spandrel}} - \frac{a_1}{2} \right)}$$

The total tension reinforcement A_s is given by:

$$A_s = A_{sf} + A_{sw}$$

The total tension reinforcement, A_s , is to be placed at the bottom of the beam for positive bending.

3.1.2.2.2 Tension and Compression Reinforcing Required

If $a_1 > \beta_1 c_b$ (CSA 10.5.2), compression reinforcing is required. In that case, the required reinforcing is computed as follows.

The depth of the concrete compression block, a , is set equal to $a_b = \beta_1 c_b$. The compressive force developed in the web concrete alone is given by

$$C_w = \alpha_1 \phi_c f'_c a t_s$$

The moment resisted by the couple between the concrete web in compression and the tension steel, M_{fc} , is given by

$$M_{fc} = C_w \left(d_{\text{spandrel}} - \frac{a_b}{2} \right)$$

Therefore, the additional moment to be resisted by the couple between the compression steel and the tension steel, M_{fs} , is given by:

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

Referring to Figure 3-2, the force carried by the compression steel, C_s , is given by:

$$C_s = \frac{M_{fs}}{d_{\text{spandrel}} - d_{r\text{-top}}}. \quad (\text{CSA 10.1.7})$$

The strain in the compression steel, ε_s' , is given by:

$$\varepsilon_s' = \frac{0.0035(c - d_{r\text{-top}})}{c}. \quad (\text{CSA 8.5.3.2, CSA 10.1.3})$$

The stress in the compression steel, f_s' , is given by:

$$f_s' = E_s \varepsilon_s' = \frac{0.0035 E_s (c - d_{r\text{-top}})}{c} \leq f_y. \quad (\text{CSA 8.5.3.2, CSA 10.1.3})$$

The term c is equal to a_b / β_1 .

The required area of compression steel, A_s' , is calculated using

$$A_s' = \frac{C_s}{\phi_s f_s' - \phi_c \alpha_1 f_c'}$$

The required area of tension steel for balancing the compression in the concrete web, A_{sw} , is:

$$A_{sw} = \frac{M_{fc}}{\phi_s f_y \left(d_{\text{spandrel}} - \frac{a}{2} \right)}$$

The required area of tension steel for balancing the compression steel, A_{sc} , is:

$$A_{sc} = \frac{M_{fs}}{\phi_s f_y (d_{\text{spandrel}} - d_{r\text{-top}})} 4$$

The total tension reinforcement A_s is given by:

$$A_s = A_{sf} + A_{sw} + A_{sc}.$$

The total tension reinforcement, A_s is to be placed at the bottom of the beam, and total compression reinforcement, A_s' at the top of the beam.

3.1.2.2.3 Minimum and Maximum Tensile Reinforcement

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

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

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

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

$$A_s \leq 0.04 t_s d_{\text{spandrel}}$$

$$A_s' \leq 0.04 t_s d_{\text{spandrel}}$$

For Ductile Flexural walls, Ductile Coupled walls, and Ductile Partially Coupled walls, when designing for seismic load combinations, the following additional conditions are satisfied:

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

$$A_{s(\text{min})} \geq \frac{1.4}{f_y} t_s d_{\text{spandrel}} \quad (\text{CSA 21.3.2.1})$$

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

$$A_s \leq 0.025 t_s d_{\text{spandrel}} \quad (\text{CSA 21.3.2.1})$$

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 axial forces, 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 following steps are involved in designing the shear reinforcing for a particular wall spandrel section for a particular design loading combination at a particular station.

- Determine the factored shear force V_f .
- Determine the shear force, V_c , that can be carried by the concrete.
- Determine the required shear reinforcing to carry the balance of the shear force.

Note: In the overwrites, V_c can be specified to be ignored (set to zero) for spandrel shear calculations.

Step 1 needs no further explanation. The following two sections describe in detail the algorithms associated with Steps 2 and 3.

3.2.1.1 Determine Factored Forces

The factored forces P_f , M_f , and V_f that are acting on the spandrel section are determined from the basic forces for each load case and the load combination factors. Then the spandrel is designed for the factored forces.

In addition to designing for the factored shear force, the spandrel section is designed for enhanced factored forces if the design load combination involves any seismic loading. In the enhanced factored forces, the seismic load factors are multiplied by a force modification factor. The force modification factor is a user defined parameter which needs to be specified in Shear Wall Design Preferences for CSA A23.3-04 code. This modification factor reflects the capability of a structure to dissipate energy through inelastic behaviors. As given in the National Building Code of Canada.

3.2.2 Determine the Concrete Shear Capacity

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

$$V_c = \phi_c \lambda \beta \sqrt{f'_c} t_s d_v, \quad (\text{CSA 11.3.4})$$

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

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

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

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

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

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

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

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

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

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

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

- When the section contains at least the minimum transverse reinforcement, β is taken as 0.18 (CSA 11.6.3.a).

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

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

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

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

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

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

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

$$\text{where, } s_{ze} = \frac{35}{15 + a_g} s_z \geq 0.85s_z \quad (\text{CSA 11.3.6.3.c})$$

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

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

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

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

$$\beta = \frac{0.40}{(1 + 1500\varepsilon_x)} \bullet \frac{1300}{(1000 + s_{ze})} \quad (\text{CSA 11.3.6.4})$$

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

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

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

$$\varepsilon_x = \frac{M_f/d_v + V_f}{(E_s A_s)} \quad (\text{CSA 11.3.6.4})$$

In evaluating the ε_x , the following conditions apply:

$$V_f \text{ and } M_f \text{ are taken as positive quantities} \quad (\text{CSA 11.3.6.4a})$$

A_s is taken as the total area of longitudinal reinforcement in the spandrel section. It is taken as the envelope of reinforcement required for all design load combinations. Actual provided reinforcement might be slightly higher than this quantity. The rebar should be developed to achieve full strength (CSA 11.3.6.3 b).

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

An upper limit on ε_x is imposed as follows:

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

ε_x is positive for tensile action.

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

3.2.3 Determine Required Shear Reinforcement

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

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

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

If $V_f \leq V_c$

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

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

$$\frac{A_v}{s} = \frac{(V_f - V_c) \tan \theta}{\phi_s f_y d_v} \quad (\text{CSA 11.3.3, 11.3.5.1})$$

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

a failure condition is declared. (CSA 11.3.3)

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

- (a) in regions of flexural members where the factored shear force V_f exceeds V_c ,
- (b) in regions of beams and columns with an overall depth greater than 750 mm, and
- (c) in regions of beams and columns where the factored torsion T_f exceeds 0.25.

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

$$\frac{A_v}{s} \geq 0.06 \frac{\sqrt{f'_c}}{f_y} t_s \quad (\text{CSA 11.2.8.2})$$

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

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

- If the overall beam depth, h , is less than 250 mm or if the depth of the T-beam below the slab is not greater than one-half of the width of web or 350 mm, θ is taken as 42 degree (CSA 11.3.6.2).
- If the specified yield strength of the longitudinal reinforcing f_y does not exceed 400 MPa, and the specified concrete strength f'_c does not exceed 60 MPa, θ is taken to be 35 degree (CSA 11.3.6.3).

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

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

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

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

In addition, the minimum vertical shear reinforcing is checked with its minimum permitted ratio of 0.0020 for Ordinary spandrels (CSA 14.3.3, CSA 14.2.2), 0.0025 for Ductile spandrels (CSA 21.5.5.1), and 0.0025 for Moderately Ductile spandrels (CSA 21.9.3.3.1).

The limit of f'_c is taken to be 80 MPa for all spandrels:

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

$$f'_c \leq 80 \text{ MPa (Ductile and partially Ductile)} \quad (\text{CSA 21.2.6.1})$$

The limit of f_y is taken to be 500 MPa for all spandrels.

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

$$f_y \leq 400 \text{ MPa (Ductile and Partially Ductile)} \quad (\text{CSA 21.2.7.1})$$

The maximum of all the calculated A_v / s values, obtained from each load combination, is reported for the major direction of the spandrel along with the controlling shear force and associated load combination number. You can set the output units for the distributed shear reinforcing in the shear wall design preferences.

Appendix A Supported Design Codes

Only one design code may be used in any one design run. That is, it is not possible to design some beams for one code and others for a different code in the same design run. However, it is possible to perform different design runs using different design codes without rerunning the analysis.

The program supports the following shear wall design codes:

- ACI 318-99
- ACI 318-02
- ACI 318-05/IBC 2006
- ACI 318-05/IBC 2006
- BS 8110-89
- BS 8110-97
- CSA A23-3-94
- Indian IS 456-2000
- UBC97

Appendix B

Shear Wall Design Preferences

The preference options are described in Table B1. An explanation of how to change a preference is provided at the end of this appendix.

Table B1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program	UBC 97	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.
Phi (Tension Controlled)	> 0	0.9	The strength reduction factor for bending in a wall pier or spandrel in tension controlled section
Phi (Compression Controlled)	> 0	0.65	The strength reduction factor for axial compression in a wall pier
Phi (Shear and/or Torsion)	> 0	0.75	The strength reduction factor for shear in a wall pier or spandrel for a nonseismic condition
Phi (Shear Seismic)	> 0	0.6	The strength reduction factor for shear in a wall pier or spandrel for a seismic condition

Table B1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
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	\geq 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	Stress ratios that are less than or equal to this value are considered acceptable.

The preference options are displayed in a two-column spreadsheet. The left column of the spreadsheet displays the preference item name. The right column of the spreadsheet displays the preference item value. To change a preference item, left click a preference item in either the left or right column of the spreadsheet. This activates a drop-down box or highlights the current preference value. If the drop-down box appears, select a new value. If the cell is highlighted, type in the desired value. The preference value will update accordingly.

Appendix C Design Procedure Overwrites

Table C-1 identifies shear wall overwrites for piers and Table C-2 identifies overwrites for spandrels applicable to ACI 318-05/IBC 2006. An explanation of how to change the overwrites is provided at the end of this appendix.

Table C-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 of Structure 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" for more information.

Table C-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
Design is Seismic	Yes or No	Yes	Toggle for design as seismic or nonseismic. Additional design checks are performed for seismic elements compared to nonseismic elements. Also, in some cases, the strength reduction factors are different.
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	12"	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	1.5"	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 C-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, $DB2_{left}$. See Figure 1 in Shear Wall Design Technical Note 6 Wall Pier Design Section. See the subsection entitled "User-Defined Edge Members" 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 top of pier, t_p . Inputting 0 means the item is to be program calculated.
LengthTop	Program calculated, or > 0	Program calculated	Wall pier length at top of 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, $DB1_{right}$.
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, $DB2_{right}$.

Table C-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
Material	Any defined concrete material property	See "Material Properties" in Shear Wall Design Technical Note 6 Wall Pier Design 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} .

C.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.

Note that the **Define menu** can be used to specify that a load case is a 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.

C.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.

Table C-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 of Structure 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" for more information.
Design is Seismic	Yes or No	Yes	Toggle for design as seismic or nonseismic. Additional design checks are performed for seismic elements compared to nonseismic elements. Also, in some cases the strength reduction factors are different.
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 left side of spandrel, t_s . Inputting 0 means the item is to be program calculated.
DepthLeft	Program calculated, or > 0	Program calculated	Wall spandrel depth at left side of spandrel, h_s . Inputting 0 means the item is to be program calculated.
CoverBotLeft	Program calculated, or > 0	Program calculated	Distance from bottom of spandrel to centroid of bottom reinforcing, $d_{r-bot\ left}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.

Table C-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
CoverTopLeft	Program calculated, or > 0	Program calculated	Distance from top of spandrel to centroid of top reinforcing, $d_{r-top\ left}$ 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-beam at left end of spandrel, b_s .
SlabDepthLeft	≥ 0	0	Slab depth for T-beam at left end of spandrel, d_s .
ThickRight	Program calculated, or > 0	Program calculated	Wall spandrel thickness at right side of spandrel, t_s . Inputting 0 means the item is to be program calculated.
DepthRight	Program calculated, or > 0	Program calculated	Wall spandrel depth at right side of spandrel, h_s . Inputting 0 means the item is to be program calculated.
CoverBotRight	Program calculated, or > 0	Program calculated	Distance from bottom of spandrel to centroid of bottom reinforcing, $d_{r-bot\ right}$ on right side of 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 top of spandrel to centroid of top reinforcing, $d_{r-top\ right}$ on right side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
SlabWidthRight	≥ 0	0	Slab width for T-beam at right end of spandrel, b_s .
SlabDepthRight	≥ 0	0	Slab depth for T-beam at right end of spandrel, d_s .
Material	Any defined concrete material property	See "Default Design Material Property" in Shear Wall Design Technical Note 7 Wall Spandrel Design Sections	Material property associated with the spandrel.

Table C-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
Consider V_c	Yes or No	Yes	Toggle switch to consider V_c (concrete shear capacity) when computing the shear capacity of the spandrel.

Appendix D

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. Two 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 the analysis section pier up 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 only used for the flexural design/check.

- **General Reinforcing Section:** For flexural designs and/or checks, the pier geometry and the reinforcing is 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 up 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 only used for the flexural design/check.

- **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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