COMPUTERS & STRUCTURES, INC.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS® 2015

Integrated Building Design Software

Shear Wall Design Manual

ACI 318-14

CSI

Shear Wall Design Manual

ACI 318-14

For ETABS® 2015

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Shear Wall Design

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 ACI 318-14 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 ACI 318-14 and IBC 2012 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 the code (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, and 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, in^2
A_g	Gross area of a wall pier, in^2
$A_{h\text{-min}}$	Minimum required area of distributed horizontal reinforcing steel required for shear in a wall spandrel, in^2/inch
A_s	Area of reinforcing steel, in^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, in^2
$A_{sc\text{-max}}$	Maximum area of compression reinforcing steel in a wall pier edge member, in^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, in^2
A_{st}	Area of reinforcing steel required for tension in a pier edge member, in^2
$A_{st\text{-max}}$	Maximum area of tension reinforcing steel in a wall pier edge member, in^2
A_v	Area of reinforcing steel required for shear, in^2 / in
A_{vd}	Area of diagonal shear reinforcement in a coupling beam, in^2

$A_{v\text{-min}}$ Minimum required area of distributed vertical reinforcing steel required for shear in a wall spandrel, in^2 / in

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, in^2

A'_s Area of compression reinforcing steel in a spandrel, in^2

B_1, B_2... Length of a concrete edge member in a wall with uniform thickness, in

C_c Concrete compression force in a wall pier or spandrel, lbs

C_f Concrete compression force in the extruding portion of a T-beam flange, lbs

C_s Compression force in wall pier or spandrel reinforcing steel, lbs

C_w Concrete compression force in the web of a T-beam, lbs

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, in. 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, in. 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, psi

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, in
L_p	Horizontal length of wall pier leg, in. This can be different at the top and the bottom of the pier
L_s	Horizontal length of wall spandrel, in
LL	Live load
M_n	Nominal bending strength, lb-in
M_u	Factored bending moment at a design section, lb-in
M_{uc}	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression and the tension steel, lb-in
M_{uf}	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the extruding portion of the flange and the tension steel, lb-in
M_{us}	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the compression steel and the tension steel, lb-in
M_{uw}	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the web and the tension steel, lb-in
OC	On a wall pier interaction curve the distance from the origin to the capacity associated with the point considered, in
OL	On a wall pier interaction curve the distance from the origin to the point considered, in
P_b	The axial force in a wall pier at a balanced strain condition, lbs

P_{left} Equivalent axial force in the left edge member of a wall pier used for design, lbs. 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 the code, lbs

$P_{max}Factor$ Factor used to reduce the allowable maximum compressive design strength, unitless. The code specifies this factor to be 0.80. This factor can be revised in the preferences.

P_n Nominal axial strength, lbs

P_o Nominal axial load strength of a wall pier, lbs

P_{oc} The maximum compression force a wall pier can carry with strength reduction factors set equal to one, lbs

P_{ot} The maximum tension force a wall pier can carry with strength reduction factors set equal to one, lbs

P_{right} Equivalent axial force in the right edge member of a wall pier used for design, lbs. This may be different at the top and the bottom of the wall pier.

P_u Factored axial force at a design section, lbs

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

RLL Reduced live load

T_s Tension force in wall pier reinforcing steel, lbs

V_c The portion of the shear force carried by the concrete, lbs

V_n Nominal shear strength, lbs

V_s	The portion of the shear force in a spandrel carried by the shear reinforcing steel, lbs
V_u	Factored shear force at a design section, lbs
WL	Wind load
a	Depth of the wall pier or spandrel compression block, in
a_1	Depth of the compression block in the web of a T-beam, in
b_s	Width of the compression flange in a T-beam, in. 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, in
$d_{r\text{-bot}}$	Distance from the bottom of the spandrel beam to the centroid of the bottom reinforcing steel, in. This can be different on the left and right ends of the beam.
$d_{r\text{-top}}$	Distance from the top of the spandrel beam to the centroid of the top reinforcing steel, in. This can be different on the left and right ends of the beam.
d_s	Depth of the compression flange in a T-beam, in. 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, in
f_y	Yield strength of steel reinforcing, psi. This value is used for flexural and axial design calculations.
f_{ys}	Yield strength of steel reinforcing, psi. This value is used for shear design calculations.
f'_c	Concrete compressive strength, psi. This value is used for flexural and axial design calculations.

f'_{cs}	Concrete compressive strength, psi. This value is used for shear design calculations.
f'_s	Stress in compression steel of a wall spandrel, psi.
h_s	Height of a wall spandrel, in. 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, in. This can be different at the top and bottom of the pier.
t_s	Thickness of a wall spandrel, in. This can be different on the left and right ends of the spandrel.
ΣDL	The sum of all dead load cases
ΣLL	The sum of all live load cases
ΣRLL	The sum of all reduced live load cases
α	The angle between the diagonal reinforcing and the longitudinal axis of a coupling beam
β_1	Unitless factor defined in Section 22.2.2.4.3 of the code
ε	Reinforcing steel strain, unitless
ε_s	Reinforcing steel strain in a wall pier, unitless
ε'_s	Compression steel strain in a wall spandrel, unitless
ϕ	Strength reduction factor, unitless
ϕ_t	Strength reduction factor for bending and tension controlled cases, unitless. The default value is 0.9.

ϕ_c Strength reduction factor for bending plus high axial compression in a concrete pier, unitless. The default value is 0.65.

ϕ_{vns} Strength reduction factor for shear in a nonseismic pier or spandrel, unitless. The default value is 0.75.

ϕ_{vs} Strength reduction factor for shear in a seismic pier or spandrel, unitless. The default value is 0.6.

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, psi

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" spandrels. 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 a spandrel is broken 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 the following equations (ACI 9.2.1).

$1.4D$	(ACI 5.3.1a)
$1.2D + 1.6L + 0.5L_r$	(ACI 5.3.1b)
$1.2D + 1.0L + 1.6L_r$	(ACI 5.3.1c)
$1.2D + 1.6(0.75\ PL) + 0.5L_r$	(ACI 5.3.1b, 6.4.3.3)
$1.2D + 1.6L + 0.5S$	(ACI 5.3.1b)
$1.2D + 1.0L + 1.6S$	(ACI 5.3.1c)
$0.9D \pm 1.0W$	(ACI 5.3.1f)
$1.2D + 1.0L + 0.5L_r \pm 1.0W$	(ACI 5.3.1d)
$1.2D + 1.6L_r \pm 0.5W$	(ACI 5.3.1c)
$1.2D + 1.6S \pm 0.5W$	(ACI 5.3.1c)
$1.2D + 1.0L + 0.5S \pm 1.0W$	(ACI 5.3.1d)
$0.9D \pm 1.0E$	(ACI 5.3.1g)
$1.2D + 1.0L + 0.2S \pm 1.0E$	(ACI 5.3.1e)

In the preceding Equations,

D = The sum of all dead load load cases defined for the model.

L = The sum of all live load load cases defined for the model. Note that this includes roof live loads as well as floor live loads.

L_r = The sum of all roof live load load cases defined for the model.

S = The sum of all snow load load cases defined for the model.

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

E = Any single earthquake load 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. Roof Live Load Component

The live load component of the default design load combinations consists of the sum of all roof live loads (unreducible), multiplied by the specified factor.

1.3.4. Snow Load Component

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

1.3.5. 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 ACI Equations 5.3.1c, 5.3.1d and 5.3.1f will contribute multiple design load combinations, one for each wind load case that is defined.

1.3.6. 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 ACI Equations 5.3.1e and 5.3.1g 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.

1.3.7. 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 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, ACI Equation 5.3.1e with negative sign for earthquake is redundant for a load combination with a response spectrum, and similarly, ACI Equation 5.3.1g with negative sign for earthquake is redundant for a load combination with a response spectrum. For this reason, the program creates default design load combinations based on ACI Equations 5.3.1e and 5.3.1g with only positive sign for earthquake for response spectra. Default design load combinations using ACI Equations 5.3.1e and 5.3.1g with negative sign for earthquake are not created for response spectra.

1.3.8. Combinations that Include Time History Results

The default shear wall design load combinations do not include any time history results. Therefore, user-defined load combinations should include time history forces.

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 (Appendix A).

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.

1.3.9. Combinations that Include Static Nonlinear Results

The default shear wall design load combinations do not include any static nonlinear results. Therefore, user-defined load combinations should include static nonlinear results.

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

1.4. Shear Wall Design Preferences

The shear wall design preferences are basic properties that apply to all wall pier and spandrel elements. Appendix A identifies shear wall design preferences for the code.

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 B identifies the shear wall overwrites for the code. 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 code is based on Pound-Inch-Second units. For simplicity, all equations and descriptions presented in this manual correspond to **pound-inch-second** units unless otherwise noted.

The Display Unit preferences allow the user to specify special units for concentrated and distributed areas of reinforcing. The special units specified for concentrated and distributed areas of reinforcing can be changed anytime.

The choices available in the Display Units preferences for the units associated with an area of concentrated reinforcing are in^2, cm^2, mm^2, ft^2 and m^2. 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, in^2/in, cm^2/cm, and so on.

Chapter 2
Pier Design

This chapter describes how the program designs and checks concrete wall piers for flexural and axial loads when the ACI 318-14 option is selected. First we describe how the program *designs* piers that are specified by a Simplified C & T Section. Next we describe how the program *checks* piers that are specified by a Uniform Reinforcing Pier Section or General Section (i.e., Section Designer). Then we describe how the program *designs* piers that are specified by a Uniform Reinforcing Pier Section or General Section (Section Designer).

This chapter also describes how the program designs each leg of concrete wall piers for shear when the ACI 318-14 option is selected. Note that in this program the user cannot specify shear reinforcing and then have the program check it. The program only designs the pier for shear and reports how much shear reinforcing is required. The shear design is performed at stations at the top and bottom of the pier.

This chapter also describes the design of boundary zone elements for each pier in accordance with ACI Section 18.10.6 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 C & T 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).

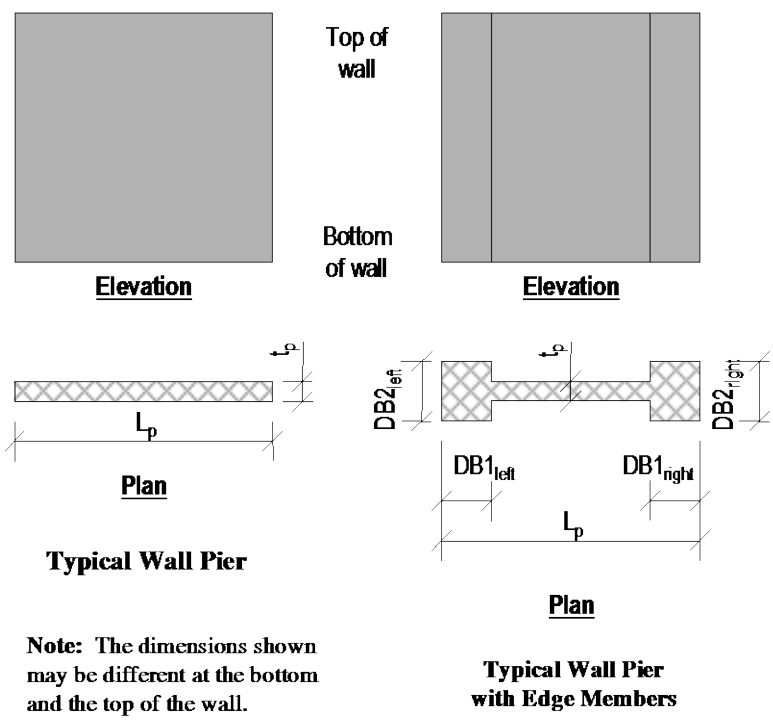

Figure 2-1: Typical Wall Pier Dimensions Used for Simplified Design

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

- The length of the wall pier is designated L_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 B), 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 thickness is the same as the thickness of 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.

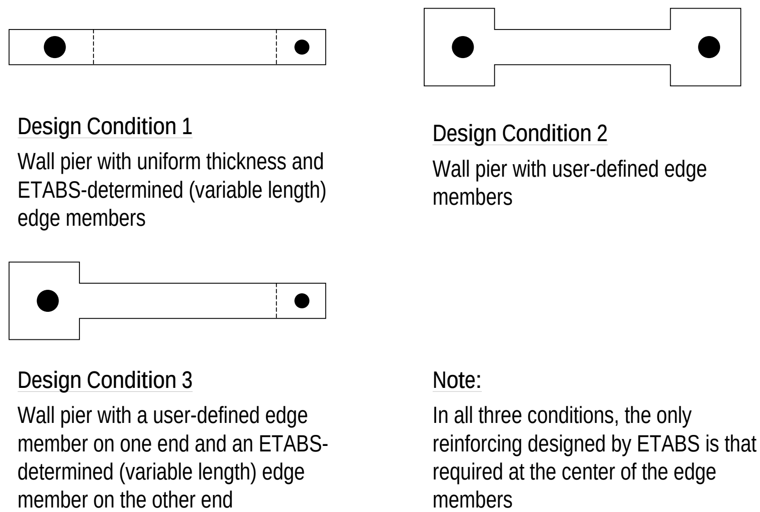

Figure 2-2: Design Conditions for Simplified Wall Piers

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 $P_{u\text{-top}}$ and a factored moment $M_{u\text{-top}}$.

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

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

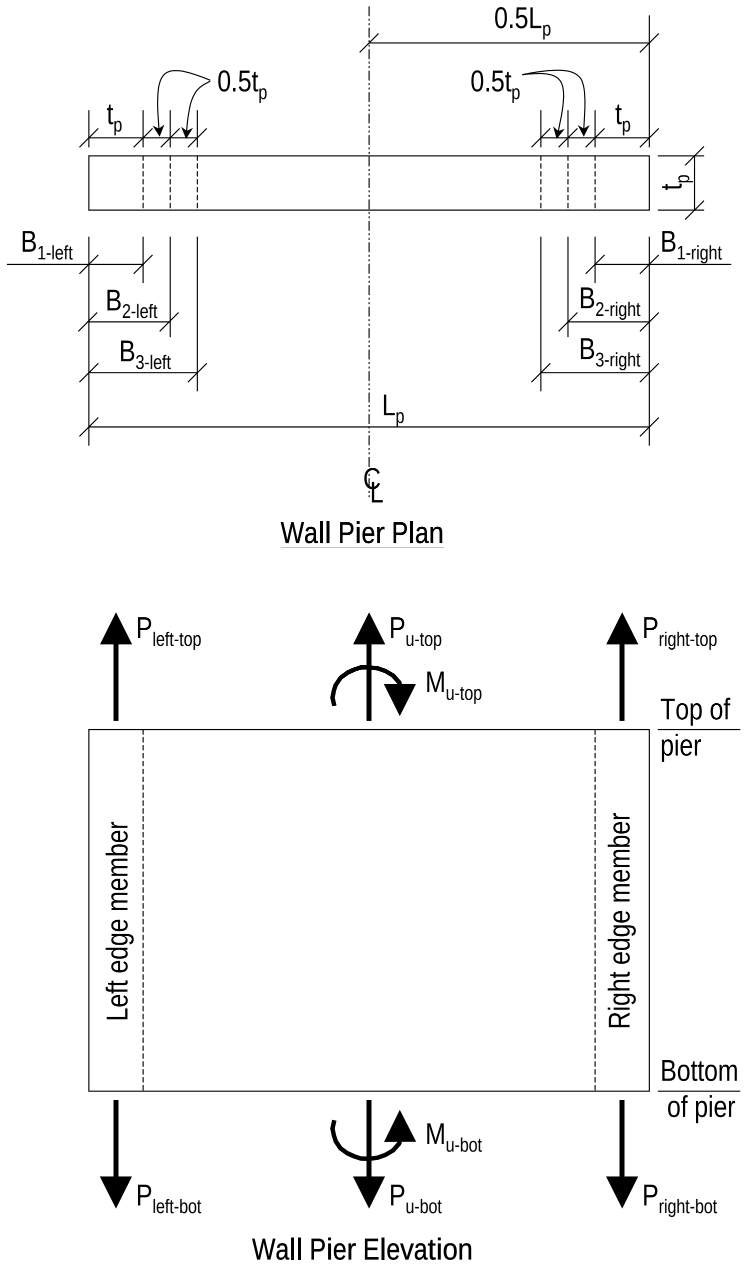

Figure 2-3: Wall Pier for Design Condition 1

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

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

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:

$$A_{st} = \frac{P}{\phi_t f_y}.$$

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.

$$Abs\,(P) = \left(P_{\text{max Factor}}\right) \phi_c \,[0.85 f_c' \,(A_g - A_{sc}) + f_y A_{sc}] \qquad \text{(ACI Table 22.4.2.1)}$$

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

$$A_{sc} = \frac{\dfrac{Abs\,(P)}{(P_{\text{max Factor}})\,\varphi_c} - 0.85 f_c' A_g}{f_y - 0.85 f_c'}.$$

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

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

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

Similarly, the compression reinforcing is limited by:

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

If A_{st} is less than or equal to $A_{st\text{-max}}$ and A_{sc} is less than or equal to $A_{sc\text{-max}}$, the program will proceed to check the next loading combination; otherwise the program will increment the appropriate B_1 dimension (left, right or both, depending on which edge member is inadequate) by one-half of the wall thickness to B_2 (i.e., $1.5t_p$) 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 of the edge members and checks if that area gives reinforcing ratios less than the user-specified maxi-mum 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 only occurs 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, M2 and M3 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°/24 curves = 15°). 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 θ+180°. Only the side of the neutral axis where the section is in tension or compression changes. We recommend that 24 interaction curves (or more) be used 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."

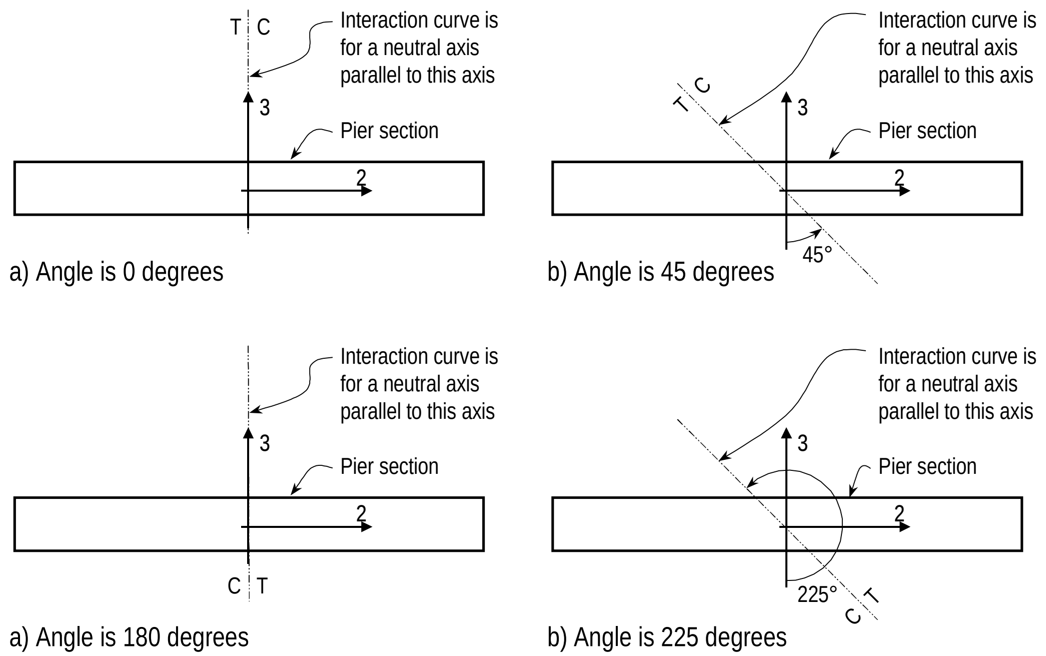

Figure 2-4: Orientation of the Pier Neutral Axis for Various Angles

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 (ACI 22.2). The program uses the requirements of force equilibrium and strain compatibility to determine the nominal axial and moment strength (P_n, M_{2n}, M_{3n}) of the wall pier. This nominal strength is then multiplied by the appropriate strength reduction factor, ϕ, to obtain the design strength (ϕP_n, ϕM_{2n}, ϕM_{3n}) of the pier. For the pier to be deemed adequate, the required strength (P_u, M_{2u}, M_{3u}) must be less than or equal to the design strength.

$$(P_u, M_{2u}, M_{3u}) \le (\phi P_n, \phi M_{2n}, \phi M_{3n})$$

The effect of the strength reduction factor, ϕ, is included in the generation of the interaction surface. The value of ϕ used in the interaction diagram varies from compression-controlled ϕ to tension-controlled ϕ based on the maximum tensile strain in the reinforcing at the extreme edge, ε_t (ACI 21.2.1, 21.2.2, Table 21.2.1, Table 21.2.2).

Sections are considered compression-controlled when the tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of $\varepsilon_{c.max}$, which is 0.003. The compression-controlled strain limit is the tensile strain in the reinforcement at balanced strain condition, which is taken as the yield strain of the steel reinforcing, f_y /E (ACI 21.2.2.1, Table 21.2.2, Fig. R21.2.2b).

Sections are tension-controlled when the tensile strain in the extreme tension steel is equal to or greater than 0.005, just as the concrete in compression reaches its assumed strain limit of 0.003 (ACI Table 21.2.2, Fig. R21.2.2b).

Sections with ε_t between the two limits are considered to be in a transition region between compression-controlled and tension-controlled sections (ACI 21.2.2, Table 21.2.2, Fig. R21.2.2b).

When the section is tension-controlled, a ϕ factor for tension-control case is used. When the section is compression-controlled, a ϕ factor for compression-control case is used. When the section falls into the transition region, ϕ is linearly interpolated between the two values (ACI 21.2.2, Table 21.2.2, Fig. R21.2.2b), as shown in the following:

$$\phi_c = \begin{cases} \phi_c & \text{if} \quad \varepsilon_t \le \varepsilon_\gamma \\ \phi_t - (\phi_t - \phi_c)\left(\dfrac{0.005-\varepsilon_t}{0.005-\varepsilon_y}\right) & \text{if} \quad \varepsilon_\gamma < \varepsilon_t \le 0.005, \\ \phi_t & \text{if} \ \varepsilon_t \ge 0.005 \end{cases} \qquad \text{(ACI 21.2.2)}$$

where,

ϕ_t = ϕ for tension controlled sections, which is 0.90 by default (ACI 21.2.1, Table 21.2.2)

ϕ_c = ϕ for compression controlled sections

= 0.65 (by default) for wall sections with tied reinforcement. (ACI 21.2.1, Table 21.2.2)

In cases involving axial tension, the strength reduction factor, ϕ, is by default equal to ϕ_t. The strength reduction factors ϕ_c and ϕ_t can be revised in the preferences and the overwrites (Appendix A).

The theoretical maximum nominal compressive force that the wall pier can carry, assuming the ϕ_c factor is equal to 1, is designated P_{oc} and is given by.

$$P_{oc} = [0.85f'_c\,(A_g - A_s) + f_yA_s] \qquad \text{(ACI 21.4.4.2)}$$

The theoretical maximum nominal tension force that the wall pier can carry, assuming the ϕ_t factor is equal to 1, is designated P_{ot} and is given by.

$$P_{ot} = f_yA_s$$

If the wall pier geometry and reinforcing is symmetrical in plan, the moments associated with both P_{oc} and P_{ot} are zero. Otherwise, a moment associated will be with both P_{oc} and P_{ot}.

The code limits the maximum compressive design strength, ϕ_cP_n, to the value given by P_{max}

$$\phi P_{max} = 0.80\phi_cP_{oc} = 0.80\phi[0.85f'_c\,(A_g - A_s) + f_yA_s] \qquad \text{(ACI 22.4.2.1)}$$

Note that the equation defining P_{max} reduces P_{oc} not only by a strength reduction factor, ϕ_c, but also by an additional factor of 0.80. In the preferences, this factor is called the $P_{max\ Factor}$, and different values for it can be specified, as required. In all 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.

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

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.

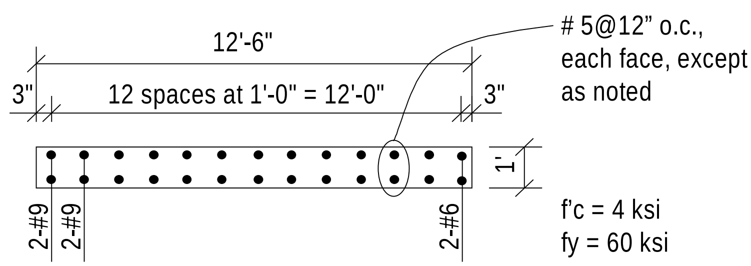

Figure 2-5: Example Two-Dimensional Wall Pier With Unsymmetrical Reinforcing

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 $P_{max\ Factor}$, applied as specified by the code.
- 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.

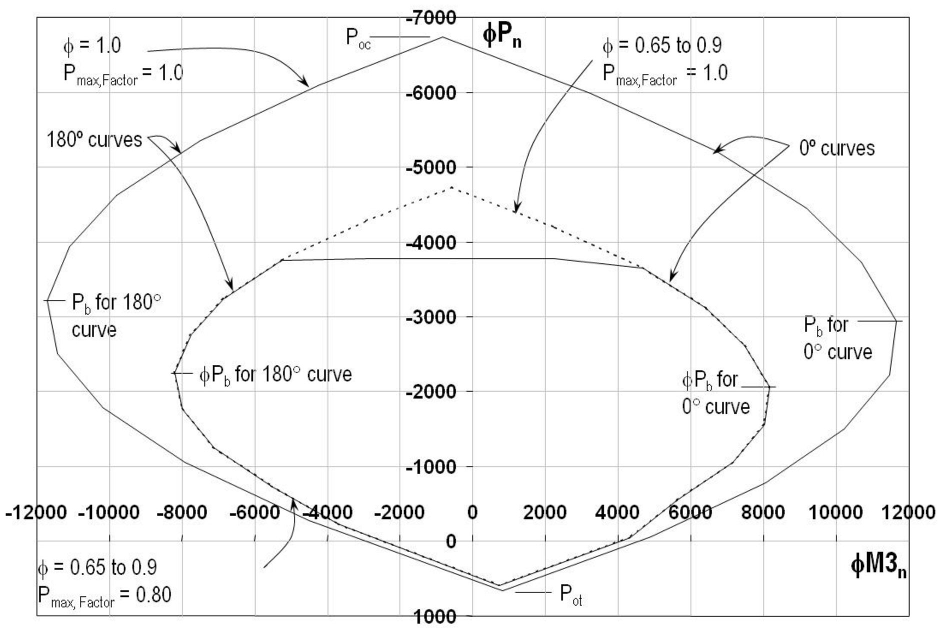

Figure 2-6 : Interaction Curves for Example Wall Pier Shown in Figure 2-5

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 ACI 318-14. The other three curves, which are plotted for ϕ = 0.65, 0.9 and 1.0, all have $P_{max,Factor}$ 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 0.9 are actually ϕ_c and ϕ_t, and that their values can be revised in the preferences as required.

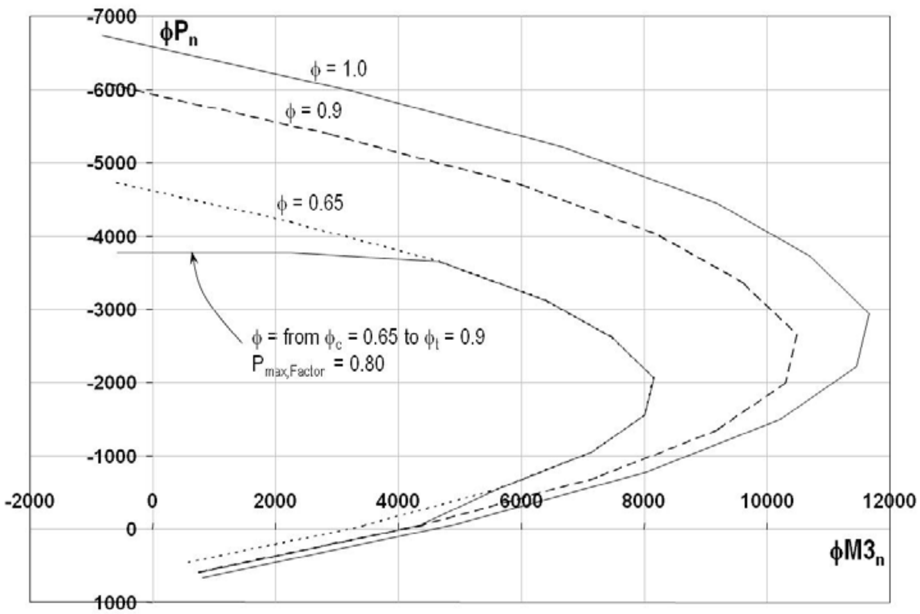

Figure 2-7: Interaction Curves for Example Wall Pier Shown in Figure 2-5

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 strength and moment strength (P_n, M_{2n}, M_{3n}) 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 −0.003 and the maximum steel strain is varied from −0.003 to plus infinity. (Recall that in this program compression is negative and tension is positive.) When the steel strain is −0.003, the maximum compressive force in the wall pier, P_{oc}, is obtained 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 (e.g., 0.00207 for f_y = 60 ksi), P_b is obtained.

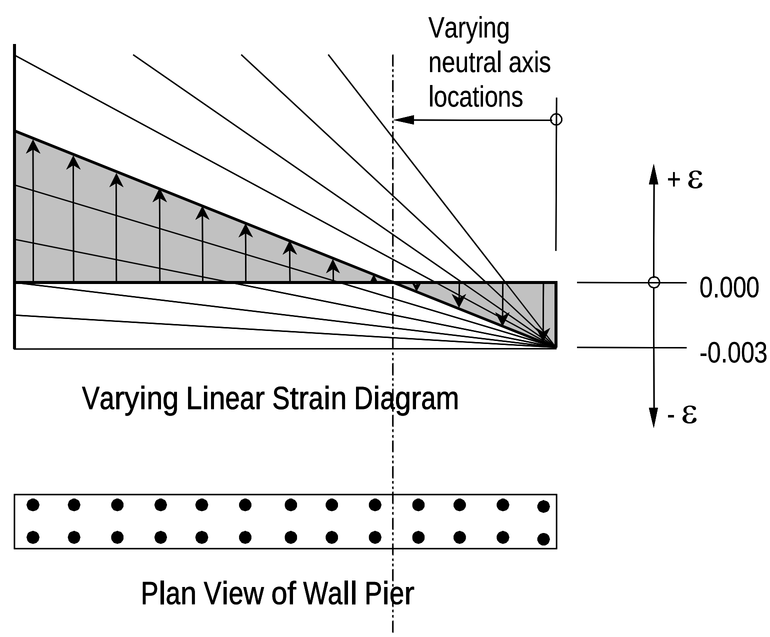

Figure 2-8: Varying Planes of Linear Strain

Figure 2-9 illustrates the concrete wall pier strain, stress, and force that is obtained from a strain compatibility analysis of a typical plane of linear strain shown in Figure 2-8. In Figure 2-9 the compressive stress in the concrete, C_c, is calculated (ACI 21.2.2.4.1).

$$C_c = 0.85 f'_c \beta_1 c t_p \qquad \text{(ACI 21.2.2.4.1)}$$

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.

$$\sigma_s = \varepsilon_s E_s \leq f_y \qquad \text{(ACI 20.2.2)}$$

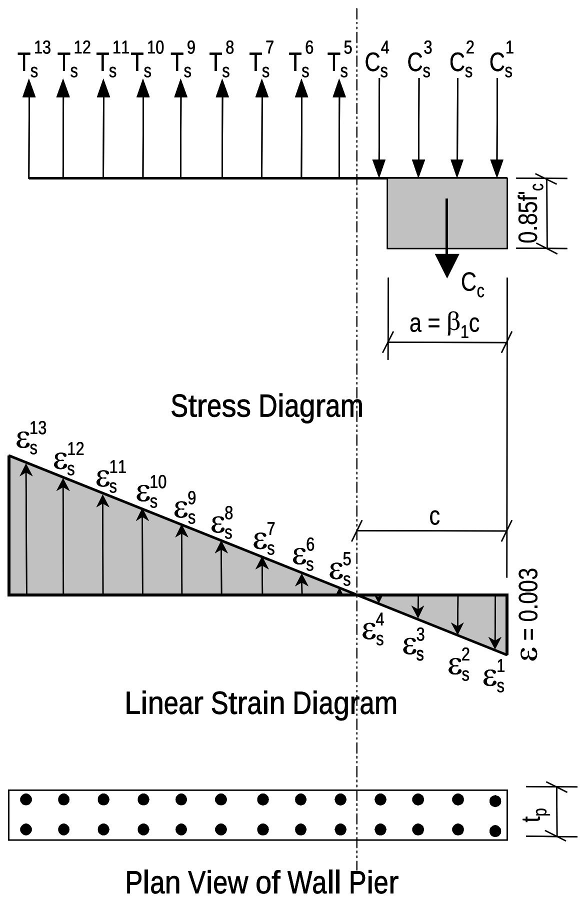

Figure 2-9: Wall Pier Stress-Strain Relationship

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 ϕP_n is calculated by.

$$\phi P_n = \phi(\Sigma T_s - C_c - \Sigma C_s) \leq \phi P_{max}$$

In the preceding equation, the tensile force T_s and the compressive forces C_c and C_s are all positive. If ϕP_n is positive, it is tension, and if it is negative, it is compression.

The value of ϕM_{2n} is calculated by summing the moments due to all of the forces about the pier local 2 axis. Similarly, the value of ϕM_{3n} is calculated by summing the moments due to all of the forces about the pier local 3 axis. The forces whose moments are summed to determine ϕM_{2n} and ϕM_{3n} are ϕC_c, all of the ϕT_s forces and all of the ϕC_s forces.

The ϕP_n, ϕM_{2n} and ϕM_{3n} values calculated as described in the preceding paragraph 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 strain; 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 $M3_u$. The point L, defined by (P_u, M_{3u}), 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_u, M_{3u}) lies on the interaction curve and the wall pier is stressed to capacity.
- If OL < OC (or D/C < 1), the point (P_u, M_{3u}) lies within the interaction curve and the wall pier capacity is adequate.
- If OL > OC (or D/C > 1), the point (P_u, M_{3u}) 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.

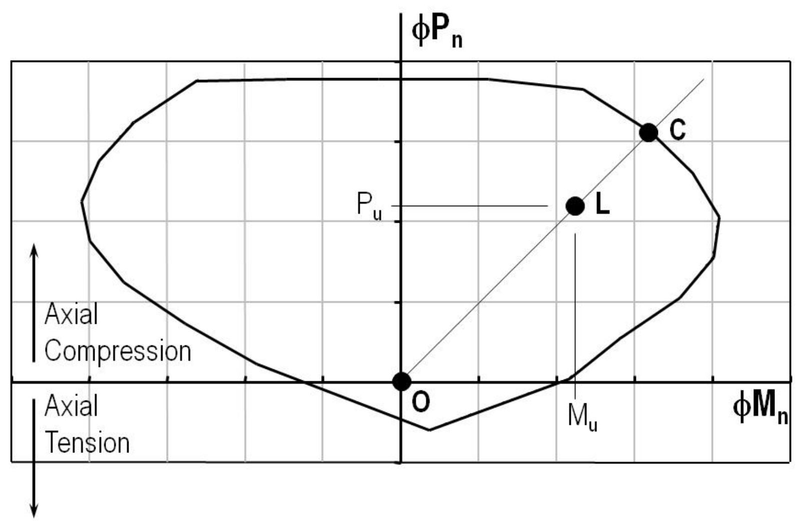

Figure 2-10: Two-Dimensional Wall Pier Demand/Capacity Ratio

2.1.4 Designing a General Reinforcing Pier Section

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

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

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

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

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

The eight reinforcing ratios used are the maximum and the minimum ratios plus six more ratios. The spacing between the reinforcing ratios is calculated as an increasing arithmetic series in which the space between the first two ratios is equal to one-third of the space between the last two ratios. Table 1 illustrates the spacing, both in general terms and for a specific example, when the minimum reinforcing ratio, 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	$\text{IPmin} + \dfrac{\text{IPmax} - \text{IPmin}}{14}$	0.0038
3	$\text{IPmin} + \dfrac{7}{3}\left(\dfrac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0054
4	$\text{IPmin} + 4\left(\dfrac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0075
5	$\text{IPmin} + 6\left(\dfrac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0100
6	$\text{IPmin} + \dfrac{25}{3}\left(\dfrac{\text{IPmax} - \text{IPmin}}{14}\right)$	0.0129
7	$\text{IPmin} + 11\left(\dfrac{\text{IPmax} - \text{IPmin}}{14}\right)$	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 an 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 leg by leg (panel by panel) 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_u, M_u and V_u that are acting on a leg of the wall pier section. Note that P_u and M_u are required for the calculation of V_c.
- Determine the shear force, V_c, that can be carried by the concrete of the leg (panel).
- 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 of the Leg

Given the design force set P_u, M_u and V_u acting on a wall pier section, the shear force carried by the concrete, V_c, is calculated using the minimum from the following two equations (ACI 11.5.4.6).

$$V_c = 3.3\lambda\sqrt{f'_c}\, t_p d + \frac{N_u d}{4L_p} \qquad \text{(ACI 11.5.4.6, Table 11.5.4.6)}$$

$$V_c \le \left[0.6\lambda\sqrt{f'_c} + \frac{L_p\left(1.25\lambda\sqrt{f'_c} + 0.2\dfrac{N_u}{L_p t_p}\right)}{Abs\left(\dfrac{M_u}{V_u}\right) - \dfrac{L_p}{2}} \right] t_p d \qquad \text{(ACI 11.5.4.6, Table 11.5.4.6)}$$

The last of the two above equations does not apply if $Abs\left(\frac{M_u}{V_u}\right) - \frac{L_p}{2}$ is negative or zero, or if V_u is zero.

In the preceding equations, N_u is the axial force, and N_u is positive for compression and negative for tension. The effective shear depth, d, is taken as follows:

$$d = 0.8\, L_p \qquad \text{(ACI 11.5.2)}$$

A limit of 100 psi on $\sqrt{f'_c}$ is imposed,

$$\sqrt{f'_c} \le 100 \text{ psi} \qquad \text{(ACI 22.5.3.1)}$$

If the tension is large enough that any of the above two equations for V_c results in a negative number, V_c is set to zero.

Note that 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. The factor λ shall normally be 0.75 for lightweight concrete and 1.0 for normal weight concrete (ACI Table 19.2.4.2). However, the program allows the user to define the value while defining concrete material property. The program uses the user input value for λ.

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

- The shear force is limited to a maximum of

$$V_{\max} = \left(10\sqrt{f'_c}\right) t_{cp} d, \quad \text{where} \qquad \text{(ACI 11.5.4.3)}$$

$$d = 0.8L_p \qquad \text{(ACI 11.5.2)}$$

- The required horizontal shear reinforcement per unit spacing, A_v /s, is calculated as follows:

If $V_u \le \phi\left(V_c/2\right)$,

$$\frac{A_v}{s} = \rho_{t,\min} t_{cp}, \qquad \text{(ACI 11.6.1, Table 11.6.1)}$$

else if $\phi V_c < V_u \le \phi V_{\max}$,

$$\frac{A_v}{s} = \frac{\left(V_u - \phi V_c\right)}{\phi f_{ys} d}, \qquad \text{(ACI 11.5.4.8, 11.5.4.4)}$$

$$\frac{A_v}{s} \ge \rho_{t,\min} t_p \qquad \text{(ACI 11.6.2)}$$

else if $V_u > \phi V_{\max}$,

a failure condition is declared. (ACI 11.5.4.3)

In the preceding equations, the strength reduction factor ϕ is taken as 0.75 for non-seismic cases ϕ_{vs} (ACI 21.2.1), and as 0.6 for seismic cases ϕ_{vs} (ACI 21.2.1, 21.2.4). However, those values may be overwritten by the user if so desired.

If V_u exceeds the maximum permitted value of $\phi V_{\max}$, the shear wall section should be increased in size (ACI 11.5.4.3).

The minimum horizontal volumetric shear rebar ratio, $\rho_{t,\min}$, and the minimum vertical volumetric shear rebar ratio, $\rho_{l,\min}$, are calculated as follows:

If $V_u \leq \phi\left(V_c/2\right)$,

$$\rho_{t,\min} = 0.0025 \qquad \text{(ACI 11.6.1, Table 11.6.1)}$$

$$\rho_{l,\min} = 0.0025 \qquad \text{(ACI 11.6.1, Table 11.6.1)}$$

else if $V_u > \phi\left(V_c/2\right)$,

$$\rho_{t,\min} = 0.0025 \qquad \text{(ACI 11.6.2(b))}$$

$$\rho_{l,\min} = 0.0025 + 0.5\left(2.5 - h_w/L_p\right)\left(\rho_t - 0.0025\right) \geq 0.0025 \qquad \text{(ACI 11.6.2(a))}$$

where,

$$\rho_t = \frac{A_v}{st_p} \qquad \text{(ACI R11.6.1)}$$

$$\rho_l = \frac{A_{st}}{L_p t_p} \qquad \text{(ACI R11.6.1)}$$

h_w = Story height,

t_p = Thickness of the shear wall panel, and

L_p = Length of the shear wall panel.

For shear design of special seismic wall pier legs for seismic load, the procedure given in this section is modified with the following exceptions.

- The concrete shear capacity is taken as follows (ACI 18.10.4.1):

$$V_c = \alpha_c \lambda \sqrt{f'_c}\, A_{cv} \qquad \text{(ACI 18.10.4.1)}$$

where,

$$\alpha_c = \begin{cases} 3.0 & \text{for } h_w/L_p \leq 1.5, \\ 2.0 & \text{for } h_w/L_p > 2.0, \\ \text{interpolated} & \text{for } 1.5 < h_w/L_p < 2.0. \end{cases} \qquad \text{(ACI 18.10.4.1)}$$

$$A_{cv} = L_p t_p \qquad \text{(ACI 18.10.4)}$$

- The maximum shear that can be carried by the wall segment irrespective of the amount of reinforcing bar provided is taken as follows (ACI 18.10.4.4):

$$V_{max} = 8\sqrt{f'_c}\, L_p t_p \qquad \text{(ACI 18.10.4.1)}$$

- The expression for required shear rebar is modified as follows:

$$\frac{A_v}{s} = \frac{V_u - \left(\alpha_c \lambda \sqrt{f'_c} L_p t_p\right)\phi_{vs}}{f_{ys} L_p} \qquad \text{(ACI 21.9.4.1)}$$

- If V_u exceeds $\lambda\sqrt{f'_c}\left(L_p t_p\right)$, the $\rho_{t,\min}$ and $\rho_{l,\min}$ is modified as follows:

$$\rho_{t,\min} = 0.0025 \qquad \text{(ACI 18.10.2.1)}$$

$$\rho_{l,\min} = 0.0025 \qquad \text{(ACI 18.10.2.1)}$$

The maximum of all of the calculated A_v /s values, obtained from each design load combination, is reported along with the controlling shear force and associated design load combination name.

The pier horizontal shear reinforcement requirements reported by the program are based purely on shear strength considerations. Any minimum shear rebar requirements to satisfy spacing consideration must be investigated independently of the program by the user.

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 the code when the Special Structural Wall option is chosen. The program uses an approach based on the requirements of Section 18.10.6 of the code. The program does not compute boundary zone requirement when maximum extreme fiber compressive stress is less than 0.2 f'_c (ACI 18.10.6.3). When the extreme fiber compressive stress is equal to or greater than 0.2 f'_c (ACI 18.10.6.3), the program also checks ACI Section 18.10.6.2 and reports the boundary zone requirement when the depth of the compression zone exceeds a limit (ACI 18.10.6.2).

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 made available for the boundary element check:

- The design forces P_u, V_u, and M_u for the pier section.
- The story height, h_w, length of the wall pier panel, 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_u by multiplying the displacement from load combination with the C_d factor provided in the shear wall design preferences (Appendix B).
- 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., the left side of the pier is the same as the right side of the pier). Only the geometry of the pier is considered, not the reinforcing, 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.

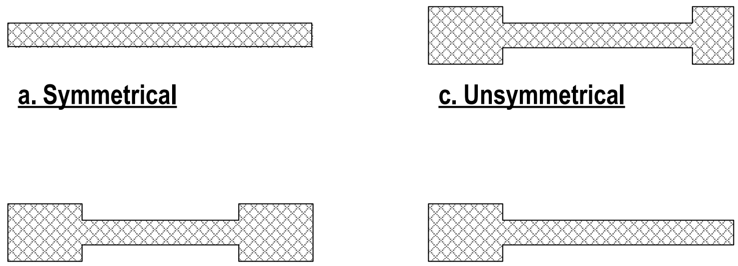

Figure 2-11 Example Plan Views of Symmetrical and Unsymmetrical Wall Piers

Using this information, the program calculates the maximum compressive stress at extreme fiber of concrete pier for the specified load combination.

After the compressive stress at the extreme fiber of the concrete pier is known, the program calculates the following quantities that are used to determine the boundary zone requirements. These quantities are: b_c, $0.2 f_c'$, δ_u / h_w, c.

When the extreme fiber compressive stress, b_c, exceeds $0.2 f_c'$, boundary elements are required (ACI 18.10.6.3), or when the neutral axial depth calculated from the factored axial force and nominal moment strength are consistent with design displacement, δ_u, and exceed the following limit:

$$c \geq \frac{l_w}{600\left(1.5\ \delta_u / h_w\right)} \qquad \text{(ACI 18.10.6.2)}$$

where,

$$\delta_u = \delta_{u,\text{elastic analysis}} \left(\frac{C_d}{I}\right) \qquad \text{(ASCE 7-10, 12.8-15)}$$

C_d = The deflection amplification factor as taken from ASCE 7-10 Table 12.2-1. It is input in the preferences.

I = The Importance factor determined in accordance with Section ASCE 11.5.1. It is input in the preferences.

$$\delta_u / h_w \leq 0.005 \qquad \text{(ACI 18.10.6.2(a))}$$

boundary elements are required (ACI 18.10.6).

The program also reports the largest neutral axis depth for each leg and the boundary zone length computed using ACI 18.10.6.4(a) when the boundary zone is **Not Needed**. This information is provided so the user can satisfy the requirement of ACI Section 18.10..6.4(a) and 18.10..6.5 when the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$.

If boundary elements are required, the program calculates the minimum required length of the boundary zone at each end of the wall, L_{BZ}, which is calculated as follows:

$$L_{BZ} = \max\left\{c/2,\, c - 0.1L_w\right\}. \quad \text{(ACI 18.10.6.4(a))}$$

Figure 2-12 illustrates the boundary zone length L_{BZ}.

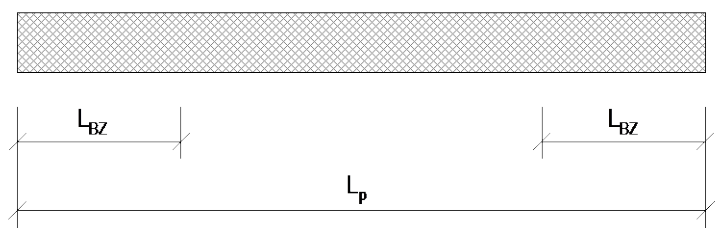

Figure 2-12: Illustration of Boundary Zone Length, L_{BZ}

2.3.2 Transverse Reinforcement for Boundary Elements

Where special boundary elements are required by ACI Sections 18.10.6.2 or 18.10.6.3, the wall boundary elements need the total cross-sectional area of rectangular hoop reinforcement as follows (ACI 18.10.6.4(c), 18.10.6.4(b)):

$$A_{sh}/s = 0.09t_p f'_c / f_{yt} \quad \text{(ACI 21-5)}$$

Where special boundary elements are not required by ACI Sections 18.10.6.2 or 21.9.6.3, and the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, the user should independently satisfy the requirements of ACI Sections 18.10.6.1(c), 18.10.4.2, and 18.10.6.5.

However, the program does not calculate and report these values. The user should independently satisfy the requirements of the sections.

Chapter 3
Spandrel Design

This chapter describes how the program designs concrete shear wall spandrels for flexure and shear when ACI 318-14 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_u.
- 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. To design a spandrel as a T-beam, specify the appropriate slab width and depth dimensions in the spandrel design overwrites (Appendix B).

The flexural design procedure is based on a simplified rectangular stress block, as shown in Figure 3-1. The maximum depth of the compression zone, c_{max}, is calculated based on the limitation that the tensile steel tension shall not be less than $\varepsilon_{s,min}$, which is equal to 0.005 for tension controlled behavior (ACI 9.3.3.1, 21.2.2, Fig. R21.2.2b):

$$c_{max} = \frac{\varepsilon_{c,max}}{\varepsilon_{c,max} + \varepsilon_{s,min}} d \qquad \text{(ACI 21.2.2)}$$

where,

$\varepsilon_{c,max} = 0.003$ (ACI 21.2.2, Fig. R21.2.2b)

$\varepsilon_{s,min} = 0.005$ (ACI 21.2.2, Fig. R21.2.2b)

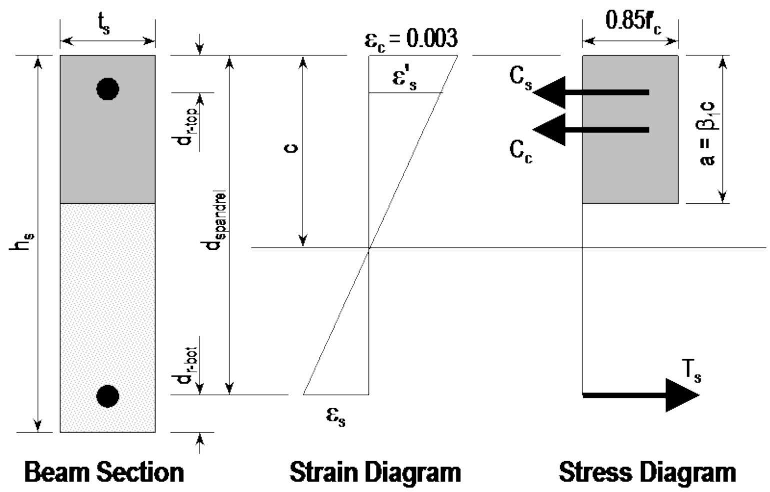

Figure 3-1 Rectangular Spandrel Beam Design, Positive Moment

The maximum allowable depth of the rectangular compression block, a_{max}, is given by

$$a_{max} = \beta_1 c_{max} \qquad \text{(ACI 22.2.2.4.1)}$$

where β_1 is calculated as follows:

$$\beta_1 = 0.85 - 0.05\left(\frac{f'_c - 4000}{1000}\right),\ 0.65 \le \beta_1 \le 0.85 \qquad \text{(ACI 22.2.2.4.3)}$$

It is assumed that the compression depth carried by the concrete is less than or equal to a_{max}. When the applied moment exceeds the moment capacity at a_{max}, the program calculates an area of compression reinforcement assuming that the additional moment is carried by compression reinforcing and additional tension reinforcing.

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 with tensile side reinforcement only, the factored moment, M_u, is resisted by a couple between the concrete in compression and the tension in reinforcing steel. This expression is given as follows.

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

Where C_c = $0.85\phi_b f'_c a t_s$ 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.

$$a = d_{\text{spandrel}} - \sqrt{d^2_{\text{spandrel}} - \frac{2M_u}{0.85 f'_c \phi_b t_s}} \qquad \text{(ACI 22.2)}$$

The program uses the preceding equation to determine the depth of the compression block, a. The depth of the compression block, a, is compared with a_{max}.

3.1.2.1.1 Tension Reinforcing Only Required

- If $a \le a_{\text{max}}$ (ACI 9.3.3.1, 21.2.2, 22.2.2.4.1), the area of tensile steel reinforcement is then given by

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

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 $a > a_{max}$, compression reinforcement is required (ACI 9.3.3.1, 21.2.2, 22.2.2.4.1) and is calculated as follows:

The depth of the concrete compression block, a, is set equal to $a = a_{\max}$. The compressive force developed in the concrete alone is given by

$$C_c = 0.85 f_c{}' a_{\max} t_s \,. \qquad \text{(ACI 22.2.2.4.1)}$$

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

$$M_{uc} = \phi_b C_c \left(d_{\text{spandrel}} - \frac{a_{\max}}{2} \right).$$

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

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

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

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

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

$$\varepsilon_s{}' = \frac{\varepsilon_{c,\max}\left(c - d_r\right)}{c}\,.$$

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

$$f_s{}' = E_s \varepsilon_s{}' = \frac{\varepsilon_{c,\max} E_s \left(c - d_r\right)}{c}. \qquad \text{(ACI 9.2.1.2, 9.5.2.1, 20.2.2, 22.2.1.2)}$$

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 $a_{\max}/\beta_1$.

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

$$A_s{}' = \frac{C_s}{\phi_b (f_s{}' - 0.85 f_c{}')}\,.$$

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

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

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

$$A_{sc} = \frac{M_{us}}{\phi_b f_y \left(d_{\text{spandrel}} - d_r \right)}.$$

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 spandrel 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.

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, as previously described in Section 3.1.2, to the depth of the T-beam flange, d_s. See Figure 3-2.

- If $a \le d_s$, 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 a_{max}.

- 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.

$$C_f = 0.85 f'_c \left(b_s - t_s\right) d_s \qquad \text{(ACI 22.2.2.4.1)}$$

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

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

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

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

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

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

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

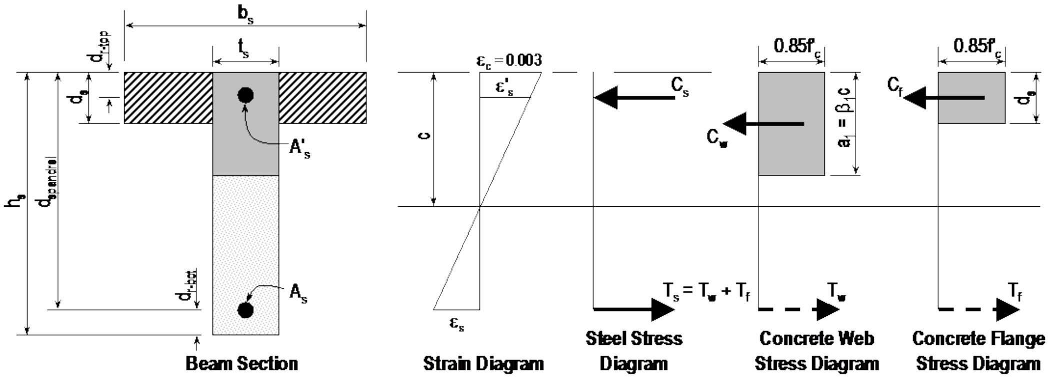

Figure 3-2: Design of a Wall Spandrel with a T-Beam Section, Positive Moment

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:

$$a_1 = d_{\text{spandrel}} - \sqrt{d_{\text{spandrel}}^2 - \frac{2M_{uw}}{0.85\, f_c'\, \phi_b\, t_s}}\ .$$

3.1.2.2.1 Tension Reinforcing Only Required

If $a_1 \le a_{\max}$, 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.

$$A_{sw} = \frac{M_{uw}}{\phi_b\, f_y \left(d_{\text{spandrel}} - \frac{a_1}{2} \right)}\ .$$

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 $a_1 > a_{max}$, 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 $a = a_{max}$. The compressive force developed in the web concrete alone is given by

  $$C_w = 0.85 f_c{}' a t_s \, .$$

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

  $$M_{uc} = \phi_b C_w \left(d_{\text{spandrel}} - \frac{a}{2} \right).$$

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

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

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

  $$C_s = \frac{M_{us}}{d_{\text{spandrel}} - d_{\text{r-top}}}$$

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

  $$\varepsilon_s{}' = \frac{\varepsilon_{c,\max} \left(c - d_{\text{r-top}} \right)}{c} \, .$$

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

  $$f_s{}' = E_s \varepsilon_s{}' = \frac{\varepsilon_{c,\max} E_s \left(c - d_{\text{r-top}} \right)}{c} \, .$$

  The term c is equal to $a_{\max} / \beta_1$.

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

$$A_s' = \frac{C_s}{\phi_b f_s'}$$

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

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

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

$$A_{sc} = \frac{M_{us}}{\phi_b f_y \left(d_{\text{spandrel}} - d_{\text{r-top}} \right)}.$$

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.

3.2 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.

1. Determine the factored shear force V_u.

2. Determine the shear force, V_c, that can be carried by the concrete.

3. 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 Determine the Concrete Shear Capacity

The shear force carried by the concrete, V_c, is given by

$$V_c = 2\lambda\sqrt{f_c'}\,t_s d_{\text{spandrel}} \qquad \text{(ACI 22.5.5.1)}$$

The shear force carried by the concrete, V_c, is calculated using the following equation when the spandrel is subjected to axial compression.

$$V_c = 2\lambda\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f_c'}\;t_s d_{\text{spandrel}} \qquad \text{(ACI 22.5.6.1, Table 22.5.6.1(b))}$$

The shear force carried by the concrete, V_c, is calculated using the following equation when the spandrel is subjected to axial tension. N_u is negative for tension.

$$V_c = 2\lambda\left(1+\frac{N_u}{500A_g}\right)\sqrt{f_c'}\;t_s d_{\text{spandrel}} \geq 0 \qquad \text{(ACI 22.5.7.1)}$$

Note that an overwrite is available that can be used to ignore the concrete contribution to the shear strength of the spandrel. If this overwrite is activated, the program sets V_c to zero for the spandrel.

In all of the preceding cases, a limit on $\sqrt{f_c'}$ is imposed as 100 psi.

$$\sqrt{f_c'} \leq 100 \text{ psi} \qquad \text{(ACI 22.5.3.1)}$$

The factor λ shall normally be 0.75 for lightweight concrete and 1.0 for normal weight concrete (ACI Table 19.2.4.2). However, the program allows the user to define the value while defining concrete material property. The program uses the user input value for λ.

3.2.2 Determine the Required Shear Reinforcing

One of the terms used in calculating the spandrel shear reinforcing is d_{spandrel}, which is the distance from the extreme compression fiber to the centroid of the tension steel. For shear design, the program takes d_{spandrel} to be equal to the smaller of $h_s - d_{r\text{-top}}$ and $h_s - d_{r\text{-bot}}$.

3.2.2.1 Seismic and Nonseismic Spandrels

In this entire subsection the term ϕ is equal to ϕ_{vns} for nonseismic spandrels and to ϕ_{vs} for seismic spandrels.

Given V_u and V_c, the required force to be carried by the shear reinforcing, V_s, is given by (ACI 22.5.1.1, 22.5.10.1).

$$V_s = V_n - V_c = \frac{V_u}{\phi} - V_c \qquad \text{(ACI 22.5.1.1, 22.5.10.1)}$$

If V_s as calculated exceeds $8\sqrt{f_c'}\,t_s d_{\text{spandrel}}$, a failure condition is reported (ACI 22.5.1.2).

Given V_s, initially calculate the required vertical shear reinforcing in area per unit length (e.g., in^2/in) for both seismic and nonseismic wall spandrels (as indicated in the preferences). Note that additional requirements that are checked for both seismic and nonseismic wall spandrels are given by the following equation (ACI 20.5.10.5.3):

$$\frac{A_v}{s} = \frac{V_s}{f_{ys} d_{\text{spandrel}}} = \frac{V_u/\phi - V_c}{f_{ys} d_{\text{spandrel}}} \qquad \text{(ACI 20.5.10.5.3, 11.5.4.8)}$$

The output units for the distributed shear reinforcing can be set in the Display Units preferences.

The following additional checks also are performed for both seismic and nonseismic spandrels.

- When $\dfrac{L_s}{d_{\text{spandrel}}} > 4$, the program verifies:

$$V_u \le \phi\left(V_c + 8\sqrt{f_c'}\, t_s d_{\text{spandrel}}\right), \qquad \text{(ACI 22.5.1.2)}$$

otherwise a failure condition is declared.

- When $\dfrac{L_s}{d_{\text{spandrel}}} > 4$ and $\dfrac{V_u}{\phi} > 0.5V_c$ (ACI 9.6.3), the minimum areas of vertical and horizontal shear reinforcing in the spandrel are as follows:

$$\frac{A_{v,\min}}{s} = 0.75\sqrt{f_c'}\,\frac{t_s}{f_{ys}} \ge \frac{50t_s}{f_{ys}} \qquad \text{(ACI 9.6.3.3, Table 9.6.3.3)}$$

$$\frac{A_{h\text{-min}}}{s} = 0. \qquad \text{(ACI 9.6.3.1)}$$

- When $\dfrac{L_s}{d_{\text{spandrel}}} > 4$ and $\dfrac{V_u}{\phi} \le 0.5V_c$, the minimum areas of vertical and horizontal shear reinforcing in the spandrel are as follows (ACI 9.6.3.1):

$$\frac{A_{v\text{-min}}}{s} = \frac{A_{h\text{-min}}}{s} = 0. \qquad \text{(ACI 9.6.3.1)}$$

- When $\dfrac{L_s}{d_{\text{spandrel}}} \le 4$, the program verifies:

$$V_u \le \phi 10\sqrt{f_c'}\, t_s d_{\text{spandrel}} \qquad \text{(ACI 9.9.2.1)}$$

otherwise a failure condition is declared.

For this condition, the minimum areas of horizontal and vertical shear reinforcing in the spandrel are:

$$A_{v\text{-min}} = 0.0025 t_s \qquad \text{(ACI 9.9.3.1(a))}$$

$$A_{h\text{-min}} = 0.0025 t_s . \qquad \text{(ACI 9.9.3.1(b))}$$

When calculating the $L_s / d_{\text{spandrel}}$ term, the program always uses the smallest value of d_{spandrel} that is applicable to the spandrel.

3.2.2.2 Seismic Spandrels Only

For seismic spandrels only, in addition to the requirements of the previous subsection, an area of diagonal shear reinforcement in coupling beams is also calculated when $\dfrac{L_s}{d_{\text{spandrel}}} \leq 4$ using the following equation (ACI 18.10.7.2, 18.10.7.3).

$$A_{vd} = \frac{V_u}{2\,\phi_s\, f_{ys} \sin\alpha}, \qquad \text{(ACI 18.10.7.4)}$$

where ϕ_s = 0.75 (ACI 21.2-1, Table 21.2.1), and

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

where h_s is the height of the spandrel and L_s is the length of the spandrel.

In the output, the program reports the diagonal shear reinforcing as required or not required (i.e., optional). The diagonal shear reinforcing is reported as required when $V_u > 4\lambda\sqrt{f_c'}\,d b_{\text{spandrel}}$ and $L_s / d_{\text{spandrel}} \leq 2$ (ACI 18.10.7.2, 18.10.7.3).

For nonseismic spandrels, A_{vd} is reported as zero.

Appendix A
Shear Wall Design Preferences

The shear wall design preferences are basic properties that apply to all wall pier and spandrel elements. Table A-1 identifies shear wall design preferences for ACI 318-14. Default values are provided for all shear wall design preference items. Thus, it is not required that preferences be specified. However, at least review the default values for the preference items to make sure they are acceptable. Refer to the program Help for an explanation of how to change a preference.

Table A-1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
Design Code	Any code in the program		Design code used for design of concrete shear wall elements (wall piers and spandrels)
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.

Table A-1 Shear Wall Preferences

Item	Possible Values	Default Value	Description
Phi (Shear Seismic)	> 0	0.6	The strength reduction factor for shear in a wall pier or spandrel for a seismic condition.
Pmax Factor	> 0	0.8	A factor used to reduce the allowable maximum compressive design strength.
Number of Curves	≥ 4	24	Number of equally spaced interaction curves used to create a full 360-degree interaction surface (this item should be a multiple of four). We recommend that you use 24 for this item.
Number of Points	≥ 11	11	Number of points used for defining a single curve in a wall pier interaction surface (this item should be odd).
Edge Design PT-max	> 0	0.06	Maximum ratio of tension reinforcing allowed in edge members, PT_{max}.
Edge Design PC-max	> 0	0.04	Maximum ratio of compression reinforcing allowed in edge members, PC_{max}.
Section Design IP-Max	≥ Section Design IP-Min	0.02	The maximum ratio of reinforcing considered in the design of a pier with a Section Designer section.
Section Design IP-Min	> 0	0.0025	The minimum ratio of reinforcing considered in the design of a pier with a Section Designer section.
Utilization Factor Limit	> 0	0.95	Stress ratios that are less than or equal to this value are considered acceptable.

Appendix B
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. Tables B-1 and B-2 identify the shear wall overwrites for piers and spandrels, respectively, for ACI 318-14. Note that the available overwrites change depending on the pier section type (Uniform Reinforcing, General Reinforcing, or Simplified T and C).

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

Table B-1: Pier Design Overwrites

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

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
LL Reduction Factor	Program calculated, > 0	Program calculated	A reducible live load is multiplied by this factor to obtain the reduced live load. Entering 0 for this item means that it is program calculated. See the subsection entitled "LL Reduction Factor" for more information.
Design is Special 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.

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
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.

Overwrites Applicable to Simplified T and C Pier Sections

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
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, $DB2_{left}$.

Table B-1: Pier Design Overwrites

Pier Overwrite Item	Possible Values	Default Value	Pier Overwrite Description
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, $DB2_{right}$.
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}.

Table B-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
Design this Spandrel	Yes or No	Yes	Toggle for design of the spandrel when the **Design menu > Shear Wall Design > Start Design/Check** command is clicked.
LL Reduction Factor	Program calculated, > 0	Program calculated	A reducible live load is multiplied by this factor to obtain the reduced live load. Entering 0 for this item means that it is program calculated. See the subsection entitled "LL Reduction Factor" later in this Appendix for more information.
Design is Special 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.

Table B-2 Spandrel Design Overwrites

Spandrel Overwrite Item	Possible Values	Default Value	Spandrel Overwrite Description
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, $d_{r\text{-bot left}}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
CoverTopLeft	Program calculated, or > 0	Program calculated	Distance from top of spandrel to centroid of top reinforcing, $d_{r\text{-top left}}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$.
SlabWidthLeft	≥ 0	0	Slab width for T-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, $d_{r\text{-bot right}}$ 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, $d_{r\text{-top right}}$ 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.

Table B-2 Spandrel Design Overwrites

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

B.1 LL Reduction Factor

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

Important Note: The LL reduction factor is ***not*** applied to any load combination that is included in a design load combination (combo or combos). 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.

B.2 User-Defined Edge Members

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

Appendix C
Analysis Sections and Design Sections

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

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

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

  For shear design and boundary zone checks, the program automatically (and internally) breaks 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 are defined by the user in the Section Designer utility. The pier defined in Section Designer may be planar or it may be three-dimensional.

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