



# Shear Wall Design Manual

**Eurocode 2-2004**

with

**Eurocode 8-2004**

**For ETABS® 2016**

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# Chapter 1

## Introduction

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The manual describes the details of the structural steel design and stress check algorithms used by the program when the user selects the Eurocode 2-2004 design code. The various notations used in this series are described herein.

The design is based on loading combinations specified by the user. To facilitate the design process, the program provides a set of default load combinations that should satisfy requirements for the design of most building type structures. See *Design Load Combinations* in Chapter 2.

The program also performs the following design, check, or analysis procedures in accordance with Eurocode 2-2004 requirements:

- Design and check of concrete wall piers for flexural and axial loads (Chapter 2).
- Design of concrete wall piers for shear (Chapter 2).
- Design of concrete wall piers for boundary zone (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, Uniformly reinforced pier section *design*, Uniformly reinforced pier section *check*, Section Designer pier section *design*, Section Designer pier section *check*, and Spandrel design.

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

## 1.1 Recommended Reading/Practice

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

## 1.2 Notation

Following is the notation used in this manual.

$A_c$	Gross area of a wall pier, mm <sup>2</sup> .
$A_{cv}$	Net area of the wall pier, mm <sup>2</sup> .
$A_{h-min}$	Minimum required area of distributed horizontal reinforcing steel required for shear in a wall spandrel, mm <sup>2</sup> / mm.
$A_s$	Area of reinforcing steel, mm <sup>2</sup> .
$A'_s$	Area of compression reinforcing steel in a spandrel, mm <sup>2</sup> .
$A_{sc}$	Area of reinforcing steel required for compression in a pier edge member, or the required area of tension steel required to balance the compression steel force in a wall spandrel, mm <sup>2</sup> .
$A_{sc-max}$	Maximum area of compression reinforcing steel in a wall pier edge member, mm <sup>2</sup> .



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$A_{sf}$	The required area of tension reinforcing steel for balancing the concrete compression force in the extruding portion of the concrete flange of a T-beam, mm <sup>2</sup> .
$A_{st}$	Area of reinforcing steel required for tension in a pier edge member, mm <sup>2</sup> .
$A_{st-max}$	Maximum area of tension reinforcing steel in a wall pier edge member, mm <sup>2</sup> .
$A_{sv}$	Area of the vertical reinforcement for wall piers
$A_{sw}$	The required area of tension reinforcing steel for balancing the concrete compression force in a rectangular concrete beam, or for balancing the concrete compression force in the concrete web of a T-beam, mm <sup>2</sup> .
$A_{swd}$	Area of diagonal shear reinforcement in a coupling beam, mm <sup>2</sup> .
$A_{sw-min}$	Minimum required area of distributed vertical reinforcing steel required for shear in a wall spandrel, mm <sup>2</sup> / mm.
$B_1, B_2...$	Length of a concrete edge member in a wall with uniform thickness, mm.
$C_c$	Concrete compression force in a wall pier or spandrel, N.
$C_f$	Concrete compression force in the extruding portion of a T-beam flange, N.
$C_s$	Compression force in wall pier or spandrel reinforcing steel, N.
$C_w$	Concrete compression force in the web of a T-beam, N.
D/C	Demand/capacity ratio as measured on an interaction curve for a wall pier, unitless.
DB1	Length of a user defined wall pier edge member, mm. This can be different on the left and right sides of the pier, and it also can be different at the top and the bottom of the pier..

DB2	Width of a user defined wall pier edge member, mm. This can be different on the left and right sides of the pier, and it also can be different at the top and the bottom of the pier..
$E$	The earthquake load on a structure resulting from the combination of the horizontal component and the vertical component.
$E_c$	Modulus of elasticity of concrete, MPa.
$E_s$	Modulus of elasticity of reinforcing steel, assumed as 200,000 MPa.
IP-max	The maximum ratio of reinforcing considered in the design of a pier with a Section Designer section, unitless.
IP-min	The minimum ratio of reinforcing considered in the design of a pier with a Section Designer section, unitless.
$L_{BZ}$	Horizontal length of the boundary zone at each end of a wall pier, mm.
LL	Live load
$L_p$	Horizontal length of wall pier, mm. This can be different at the top and the bottom of the pier.
$L_s$	Horizontal length of wall spandrel, mm.
$M_{Ed}$	Factored bending moment at a design section, N-mm.
$M_{Edc}$	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression and the tension steel, N-mm.
$M_{Edf}$	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the extruding portion of the flange and the tension steel, N-mm.

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$M_{Eds}$	In a wall spandrel with compression reinforcing, the factored bending moment at a design section resisted by the couple between the compression steel and the tension steel, N-mm.
$M_{Edw}$	In a wall spandrel with a T-beam section and compression reinforcing, the factored bending moment at a design section resisted by the couple between the concrete in compression in the web and the tension steel, N-mm.
$N_b$	The factored axial force resistance in a wall pier at a balanced strain condition, N.
$N_{c,max}$	Maximum ratio of compression steel in an edge member of a wall pier, unitless.
$N_{Ed}$	Factored axial force at a design section, N.
$N_{left}$	Equivalent axial force in the left edge member of a wall pier used for design, N. This may be different at the top and the bottom of the wall pier.
$N_O$	Axial load capacity at zero eccentricity, N.
$N_{oc}$	The maximum compression force a wall pier can carry with strength reduction factors set equal to one, N.
$N_{ot}$	The maximum tension force a wall pier can carry with strength reduction factors set equal to one, N.
$N_r$	Factored axial resistance of a design section, N.
$N_{r,max}$	Maximum factored axial resistance of a design section, N.
$N_{right}$	Equivalent axial force in the right edge member of a wall pier used for design, N. This may be different at the top and the bottom of the wall pier.
$N_{t,max}$	Maximum ratio of tension steel in an edge member of a wall pier, unitless.

OC	On a wall pier interaction curve the "distance" from the origin to the capacity associated with the point considered.
OL	On a wall pier interaction curve the "distance" from the origin to the point considered.
$P_{\max}$ Factor	Factor used to reduce the allowable maximum compressive design strength, unitless. This is taken as 0.8 by default. This factor can be revised in the preferences.
$R$	Force modification factor for the seismic component of loads.
RLL	Reduced live load.
$T_s$	Tension force in wall pier reinforcing steel, N.
$V_{Ed}$	Factored shear force at a design section, N.
$V_r$	Nominal shear strength, N.
$V_{Rd,c}$	The portion of the shear force carried by the concrete, N.
$V_s$	The portion of the shear force in a spandrel carried by the shear reinforcing steel, N.
WL	Wind load.
$a$	Depth of the wall pier or spandrel compression block, mm.
$a_1$	Depth of the compression block in the web of a T-beam, mm.
$a_b, a_{limiting}$	Depth of the compression block in a wall spandrel for balanced strain conditions, mm.
$b_s$	Width of the compression flange in a T-beam, mm. This can be different on the left and right ends of the T-beam.
$d_{r-bot}$	Distance from the bottom of the spandrel beam to the centroid of the bottom reinforcing steel, mm. This can be different on the left and right ends of the beam.

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$d_{r-top}$	Distance from the top of the spandrel beam to the centroid of the top reinforcing steel, mm. This can be different on the left and right ends of the beam.
$d_s$	Depth of the compression flange in a T-beam, inches. This can be different on the left and right ends of the T-beam.
$d_{spandrel}$	Depth of spandrel beam minus cover to centroid of reinforcing, mm.
$f_{cd}$	Design concrete compressive strength (EC 3.1.6), MPa
$f_{cd,s}$	Concrete compressive strength, MPa. This value is used for shear design calculations.
$f'_{sd}$	Stress in compression steel of a wall spandrel, MPa.
$f_{yd}$	Design yield strength of reinforcement (EC2 3.2), MPa. This value is used for flexural and axial design calculations.
$f_{ywd}$	Design yield strength of steel reinforcing, MPa. This value is used for shear design calculations.
$h_s$	Height of a wall spandrel, mm. This can be different on the left and right ends of the spandrel.
$h_w$	Height of the wall pier, mm.
$l_c$	Computed length of the boundary element, m.
$p_{max}$	Maximum ratio of reinforcing steel in a wall pier with a Section Designer section that is designed (not checked), unitless.
$p_{min}$	Minimum ratio of reinforcing steel in a wall pier with a Section Designer section that is designed (not checked), unitless.
$t_p$	Thickness of a wall pier, mm. This can be different at the top and bottom of the pier.
$t_s$	Thickness of a wall spandrel, mm. This can be different on the left and right ends of the spandrel.

$v_d$	Calculated maximum normalized compressive stress at the extreme fiber of the concrete pier for the specified load combination
$x$	Distance from the extreme compression fiber of the wall pier or spandrel to the neutral axis, mm.
$x_b$	Distance from the extreme compression fiber of a spandrel to the neutral axis for balanced strain conditions, mm.
$x_u$	Neutral axis depth in wall piers
$\Sigma DL$	The sum of all dead load cases.
$\Sigma LL$	The sum of all live load cases.
$\Sigma RLL$	The sum of all reduced live load cases.
$\alpha$	The angle between the diagonal reinforcing and the longitudinal axis of a coupling beam.
$\alpha_{cc}$	Material coefficient taking account of long-term effects on the compressive strength (EC2 3.1.6)
$\alpha_{lcc}$	Light-weight material coefficient taking account of long-term effects on the compressive strength (EC2 11.3.5)
$\alpha_{ct}$	Material coefficient taking account of long-term effects on the tensile strength (EC2 3.1.6)
$\alpha_{lct}$	Light-weight material coefficient taking account of long-term effects on the tensile strength (EC2 3.1.6)
$\varepsilon$	Reinforcing steel strain, unitless.
$\varepsilon_c$	Limiting strain in compression, unitless. It is taken as 0.0035.
$\varepsilon_{cu2}$	Ultimate strain allowed in extreme concrete fiber (0.0035 mm/mm)
$\varepsilon_s$	Reinforcing steel strain in a wall pier, unitless.
$\varepsilon'_s$	Compression steel strain in a wall spandrel, unitless.

$\gamma_c$	Material partial factor for concrete (EC2 2.4.2.4)
$\gamma_s$	Material partial factor for steel (EC2 2.4.2.4)
$\eta$	Factor defining the effective strength of the concrete stress block (EC2 3.1.7)
$\lambda$	Factor defining the effective height of the concrete stress block (EC2 3.1.7)
$\theta$	Angle of inclination of diagonal compressive stresses with the longitudinal axis of beam or column.
$\theta_0$	Basic inclination for geometric imperfections (EC2 5.2), ratio
$\theta_i$	Inclination due to geometric imperfections (EC2 5.2), ratio

### 1.3 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.**

### 1.4 Design Load Combinations

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

program default design load combinations. An unlimited number of design load combinations can be specified.

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

As an example, if a structure is subjected to dead load, D, and live load, L, only, the EC2 design check may need only one design load combination, namely, 1.35G + 1.5Q. However, if the structure is subjected to wind, earthquake or other loads, numerous additional design load combinations may be required.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

The design load combinations are the various combinations of the prescribed load cases for which the structure is to be checked. For this code, if a structure is subjected to dead load (D), live load (L), wind (W), and earthquake (E) loads, and considering that wind and earthquake forces are reversible, Eurocode 0-2002 allows load combinations to be defined based on EC0 Eq. 6.10 or the less favorable EC0 Eqs. 6.10a and 6.10b [NDP].

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (\text{EC0 Eq. 6.10})$$

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \Psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (\text{EC0 Eq. 6.10a})$$

$$\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (\text{EC0 Eq. 6.10b})$$

Load combinations considering seismic loading are automatically generated based on EC0 Eq. 6.12b.



$$\sum_{j \geq 1} G_{k,j} + P + A_{Ed} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \quad (\text{EC0 Eq. 6.12b})$$

For this code, if a structure is subjected to dead (D), live (L), wind (W), and earthquake (E) loads, and considering that wind and earthquake forces are reversible, the following load combinations need to be considered if equation 6.10 is specified for generation of the load combinations (EC0 6.4.3):

$$\gamma_{Gj,\text{sup}} D \quad (\text{EC0 Eq. 6.10})$$

$$\gamma_{Gj,\text{sup}} D + \gamma_{Q,1} L \quad (\text{EC0 Eq. 6.10})$$

$$\gamma_{Gj,\text{inf}} D \pm \gamma_{Q,1} W \quad (\text{EC0 Eq. 6.10})$$

$$\gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} W \quad (\text{EC0 Eq. 6.10})$$

$$\gamma_{Gj,\text{sup}} D + \gamma_{Q,1} L \pm \gamma_{Q,i} \psi_{0,i} W \quad (\text{EC0 Eq. 6.10})$$

$$\gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} W + \gamma_{Q,i} \psi_{0,i} L \quad (\text{EC0 Eq. 6.10})$$

$$D \pm 1.0E \quad (\text{EC0 Eq. 6.12b})$$

$$D \pm 1.0E + \psi_{2,i} L$$

If the load combinations are specified to be generated from the maximum of EC0 Eqs. 6.10a and 6.10b, the following load combinations from both equations are considered in the program.

$$\gamma_{Gj,\text{sup}} D \quad (\text{EC0 Eq. 6.10a})$$

$$\xi \gamma_{Gj,\text{sup}} D \quad (\text{EC0 Eq. 6.10b})$$

$$\gamma_{Gj,\text{sup}} D + \gamma_{Q,1} \psi_{0,1} L \quad (\text{EC0 Eq. 6.10a})$$

$$\xi \gamma_{Gj,\text{sup}} D + \gamma_{Q,1} L \quad (\text{EC0 Eq. 6.10b})$$

$$\gamma_{Gj,\text{inf}} D \pm \gamma_{Q,1} \psi_{0,1} W \quad (\text{EC0 Eq. 6.10a})$$

$$\gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} \psi_{0,1} W \quad (\text{EC0 Eq. 6.10a})$$

$$\gamma_{Gj,\text{inf}} D \pm \gamma_{Q,1} W \quad (\text{EC0 Eq. 6.10b})$$

$$\xi \gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} W \quad (\text{EC0 Eq. 6.10b})$$

$$\gamma_{Gj,\text{sup}} D + \gamma_{Q,1} \psi_{0,1} L \pm \gamma_{Q,i} \psi_{0,i} W \quad (\text{EC0 Eq. 6.10a})$$

$$\gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} \psi_{0,1} W + \gamma_{Q,i} \psi_{0,i} L \quad (\text{EC0 Eq. 6.10a})$$

$$\xi \gamma_{Gj,\text{sup}} D + \gamma_{Q,1} L \pm \gamma_{Q,i} \psi_{0,i} W \quad (\text{EC0 Eq. 6.10b})$$

$$\gamma_{Gj,\text{sup}} D \pm \gamma_{Q,1} W + \gamma_{Q,i} \psi_{0,i} L \quad (\text{EC0 Eq. 6.10b})$$

$$D \pm 1.0E \quad (\text{EC0 Eq. 6.12b})$$

$$D \pm 1.0E + \psi_{2,i} L$$

For both sets of load combinations, the variable values for the CEN Default version of the load combinations are defined in the list that follows. Values for other countries, as determined from their National Annex, are included in Appendix C.

$$\gamma_{Gj,\text{sup}} = 1.35 \quad (\text{EC0 Table A1.2(B)})$$

$$\gamma_{Gj,\text{inf}} = 1.00 \quad (\text{EC0 Table A1.2(B)})$$

$$\gamma_{Q,1} = 1.5 \quad (\text{EC0 Table A1.2(B)})$$

$$\psi_{0,i} = 0.7 \text{ (live load, assumed not to be storage)} \quad (\text{EC0 Table A1.1})$$

$$\psi_{0,i} = 0.6 \text{ (wind load)} \quad (\text{EC0 Table A1.1})$$

$$\xi = 0.85 \quad (\text{EC0 Table A1.2(B)})$$

$$\psi_{2,i} = 0.3 \text{ (assumed to be office/residential space)} \quad (\text{EC0 Table A1.1})$$

These are also the default design load combinations in the program whenever the Eurocode 2-2004 code is used. In generating the preceding default loading combinations, the importance factor is taken as 1. The user should use other appropriate design load combinations if roof live load is separately treated, or if other types of loads are present. PLL is the live load multiplied by the Pattern Live Load Factor. The Pattern Live Load Factor can be specified in the Preferences.

When using the Eurocode 2-2004 code, the program design assumes that a P-delta analysis has been performed.

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.

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

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

### 1.4.1 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 section for additional information.

### 1.4.2 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.

### 1.4.3 Wind Load Component

The wind load component of the default design load combinations consists of the contribution from a single wind load case. Thus, if multiple wind load cases are defined in the model, the program will contribute multiple design load combinations, one for each wind load case that is defined.

### 1.4.4 Earthquake Load Component

The earthquake load component of the default design load combinations consists of the contribution from a single earthquake load case. Thus, if multiple earthquake load cases are defined in the program model, the program 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.

### 1.4.5 Combinations That Include a Response Spectrum

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

The output from a response spectrum is all positive. *Any shear wall design load combination that includes a response spectrum load case is checked in the program 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  $P$ ,  $M_2$  and  $M_3$  are considered for three-dimensional wall piers.

### 1.4.6 Combinations that Include Time History Results

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

When a design load combination includes time history results, you can either design for the envelope of those results or you can do a design for each step of the time history. The type of time history design is specified in the shear wall

design preferences. Please refer to the on-line Help for more information about preferences.

When the design uses envelopes, the design is for the maximum of each response quantity (axial load, moment, etc.) as if those 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 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.

### 1.4.7 Combinations that Include Static Nonlinear Results

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

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

## 1.5 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 Eurocode 2-2004. 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.

## 1.6 Shear Wall Design Overwrites

The shear wall design overwrites are basic assignments that apply only to those piers or spandrels to which they are assigned. The overwrites for piers and spandrels are separate. Appendix B identifies the shear wall overwrites for Eurocode 2-2004. 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.

## 1.7 Choice of Units

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.

English as well as SI and MKS metric units can be used for input. But the codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. For example, the Eurocode 2-2004 code is published in Millimeter-Newton-Second units. By default, all equations and descriptions presented in this manual correspond to Millimeter-Newton-Second units. However, any system of units can be used to define and design the structure in the program.

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## Chapter 2 Pier Design

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This chapter describes how the program designs and checks concrete wall piers for flexural and axial loads using Eurocode 2-2004 which was reaffirmed in 2010. First we describe how the program *designs* piers that are specified by a Simplified Section. Next we describe how the program *checks* piers that are specified by a Uniform Pier Reinforcing Section or General Section (i.e., Designer Section). Then we describe how the program *designs* piers that are specified by a Uniform Pier Reinforcing Section or General (Section Designer) Section.

This chapter also describes how the program designs each leg of concrete wall piers for shear using Eurocode 2-2004. Note that in this program it is not possible to specify shear reinforcing and then have the program check it. The program only designs the pier for shear and reports how much shear reinforcing is required. The shear design is performed at stations at the top and bottom of the pier.

This chapter also describes the design of boundary zone elements for each pier in accordance with EC8 Section 5.4.3.4.2 and 5.5.3.4 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 Determine Factored Moments and Forces

The loads for a particular design load combination are obtained by applying the corresponding factors to all of the load cases, giving  $N_{Ed}$ ,  $M_{Ed2}$ , and  $M_{Ed3}$ . These first order factored moments are further increased to account for geometric imperfections (EC2 5.2) for Uniform and general Reinforcing options. The design forces are not amplified for Simplified C&T Design. The eccentricity to account for geometric imperfections,  $e_i$ , is defined as:

$$e_i = \theta_i l_0 / 2 \quad (\text{EC2 Eq. 5.2})$$

where  $l_0$  is the effective length of the member and  $\theta_i$  is an inclination, defined as a ratio as:

$$\theta_i = \theta_0 \alpha_h \alpha_m \quad (\text{EC2 Eq. 5.1})$$

where  $\alpha_m$  is a reduction factor for the number of members, taken as 1 in the program for isolated members,  $\alpha_h$  is a reduction factor for length, taken as  $2/\sqrt{l}$ , and  $\theta_0$  [NDP] is the basic inclination, defined as a ratio, and can be overwritten in the Preferences. The resulting geometric imperfection moments, are calculated as:

$$M_{i2} = e_{i2} N_{Ed}$$

$$M_{i3} = e_{i3} N_{Ed}$$

where  $e_i$  shall be taken greater than or equal to the code specified minimum eccentricity  $e_{\min}$  (EC2 6.2). The minimum eccentricity,  $e_{\min}$ , is defined as:

$$e_{\min} = h/30 \geq 20 \text{ mm} \quad (\text{EC2 6.1})$$

The moment generated because of the geometric imperfection eccentricity, or the min eccentricity if greater, is considered only in a single direction at a time.

### 2.1.1.1 Second Order Moments

The design algorithm assumes that the moments  $M_{Ed2}$  and  $M_{Ed3}$  are obtained from a second order elastic ( $P$ - $\Delta$ ) analysis or by applying fictitious, magnified horizontal forces following the recommendations of EC2 Annex H.

The computed moments are further increased for individual pier stability effects,  $P$ - $\delta$  (EC2 5.8.5), by computing a moment magnification factor based on the Nominal Stiffness method (EC2 5.8.7) or computing a nominal second order moment based on the Nominal Curvature method (EC2 5.8.8).

#### 2.1.1.1.1 Nominal Stiffness Method

The overall design moment,  $M_{Ed}$ , based on the Nominal Stiffness method is computed as:

$$M_{Ed} = M_{0Ed} (\text{factor}) \quad (\text{EC2 Eq. 5.31})$$

where  $M_{0Ed}$  is defined as:

$$M_{0Ed} = M_{0e} + M_i$$

$$M_{0e} = 0.6 M_{02} + 0.4 M_{01} \geq 0.4 M_{02} \quad (\text{EC2 Eq. 5.32})$$

$M_{02}$  and  $M_{01}$  are the moments at the ends of the pier, and  $M_{02}$  is numerically larger than  $M_{01}$ .  $M_{01}/M_{02}$  is positive for single curvature bending and negative for double curvature bending. The preceding expression of  $M_{0e}$  is valid if there is no transverse load applied between the supports.

The moment magnification factor associated with the major or minor direction of the pier is defined as:

$$\text{factor} = 1 + \beta / \left[ (N_B / N_{Ed}) - 1 \right] \quad (\text{EC2 Eq. 5.28})$$

The factor  $\beta$  depends on the distribution of the first and second order moments and is defined as:

$$\beta = \pi^2 / c_0, \quad (\text{EC2 Eq. 5.29})$$

where  $c_0$  is a coefficient that depends on the distribution of the first order moment and is taken equal to 8, which is consistent with a constant first order moment. The term  $N_B$  is the buckling load and is computed as:

$$N_B = \frac{\pi^2 EI}{(\beta_l l_u)^2}$$

where  $\beta_l$  is conservatively taken as 1. The unsupported length of the pier for the direction of bending considered is defined as  $l_u$ . The two unsupported lengths are  $l_{22}$  and  $l_{33}$ , corresponding to instability in the minor and major directions of the object, respectively. These are the lengths between the support points of the object in the corresponding directions.

When using the stiffness method, the  $EI$  associated with a particular column direction is considered in the design as:

$$EI = 0.3EI_g$$

This value neglects creep effects and may be unconservative if significant creep effects exist.

The additional moment from the Nominal Stiffness method must be a positive number. Therefore,  $N_{Ed}$  must be greater than  $N_B$ . If  $N_{Ed}$  is found to be less than or equal to  $N_B$ , a failure condition is declared.

#### 2.1.1.1.2 Nominal Curvature Method

The overall design moment,  $M_{Ed}$ , based on the Nominal Curvature method is computed as:

$$M_{Ed} = M_{0Ed} + M_2 \quad (\text{EC2 Eq. 5.31})$$

where  $M_{0Ed}$  is defined as:

$$M_{0Ed} = M_{0e} + M_i$$

$$M_{0e} = 0.6 M_{02} + 0.4 M_{01} \geq 0.4 M_{02} \quad (\text{EC2 Eq. 5.32})$$

$M_{02}$  and  $M_{01}$  are the moments at the ends of the column, and  $M_{02}$  is numerically larger than  $M_{01}$ .  $M_{01}/M_{02}$  is positive for single curvature bending and negative

for double curvature bending. The preceding expression of  $M_{0e}$  is valid if no transverse load is applied between the supports.

The additional second order moment associated with the major or minor direction of the pier is defined as:

$$M_2 = N_{Ed} e_2 \quad (\text{EC2 Eq. 5.33})$$

where  $N_{Ed}$  is the design axial force, and  $e_2$ , the deflection due to the curvature, is defined as:

$$e_2 = (1/r) l_o^2 / c \quad (\text{EC2 5.8.8.2(3)})$$

The effective length,  $l_o$ , is taken equal to  $\beta_1 l_u$ . The factor  $c$  depends on the curvature distribution and is taken equal to 8, corresponding to a constant first order moment. The term  $1/r$  is the curvature and is defined as:

$$1/r = K_r K_\phi 1/r_0 \quad (\text{EC2 Eq. 5.34})$$

The correction factor,  $K_r$ , depends on the axial load and is taken as 1 by default. The factor  $K_\phi$  is also taken as 1, which represents the situation of negligible creep. Both of these factors can be overwritten on a member-by-member basis. The term  $1/r_0$  is defined as:

$$1/r_0 = \varepsilon_{yd} / (0.45d) \quad (\text{EC2 5.8.8.3(1)})$$

The preceding second order moment calculations are performed for major and minor directions separately.

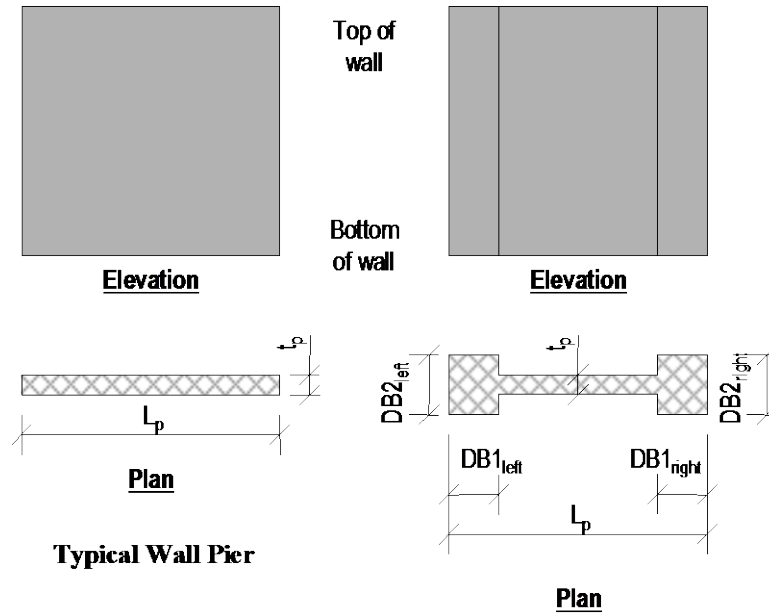
The additional second order moment,  $M_2$  is ignored when the slenderness,  $\lambda$  is less than the limiting slenderness specified by the code.

$$\lambda_{\text{lim}} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}} \quad (\text{EC2 5.8.3.1(1)})$$

## 2.1.2 Designing a Simplified Pier Section

This section describes how the program designs a pier that is assigned a simplified section. The geometry associated with the simplified section is

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



**Note:** The dimensions shown may be different at the bottom and the top of the wall.

**Typical Wall Pier with Edge Members**

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

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

- The length of the wall pier is designated  $L_p$ . This is the horizontal length of the wall pier in plan.
- The thickness of the wall pier is designated  $t_p$ . The thickness specified for left and right edge members ( $DB2_{left}$  and  $DB2_{right}$ ) may be different from this wall thickness.
- $DB1$  represents the horizontal length of the pier edge member.  $DB1$  can be different at the left and right sides of the pier.

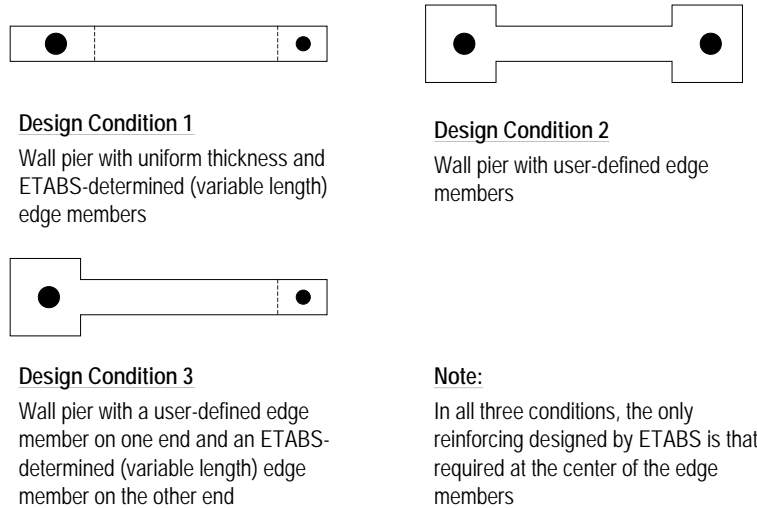
- DB2 represents the horizontal width (or thickness) of the pier edge member. DB2 can be different at the left and right sides of the pier.

The dimensions illustrated are specified in the shear wall overwrites (Appendix 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 is the same width as the wall, and the program determines the required length of the edge member. In all cases, whether the edge member size is user specified or program determined, the program reports the required area of reinforcing steel at the center of the edge member. This section describes how the program determined length of the edge member is determined and how the program calculates the required reinforcing at the center of the edge member.

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

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



*Figure 2-2 Design Conditions for Simplified Wall Piers*

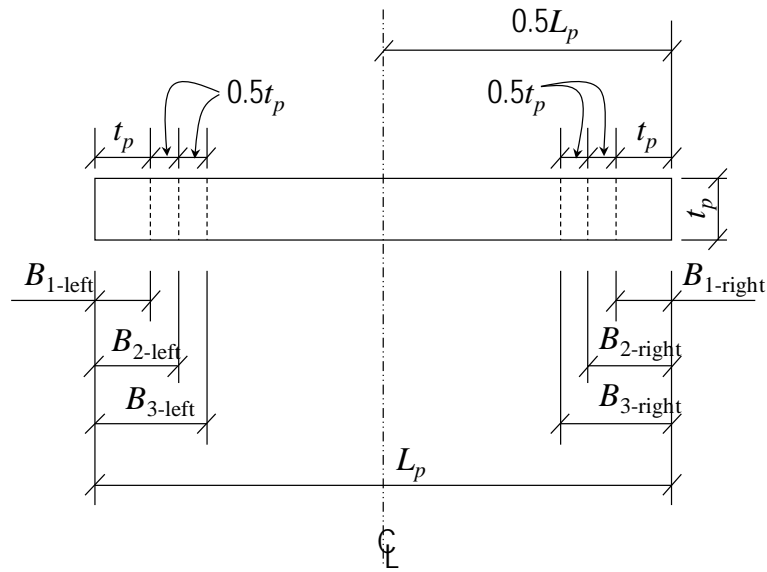
### 2.1.2.1 Design Condition 1

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

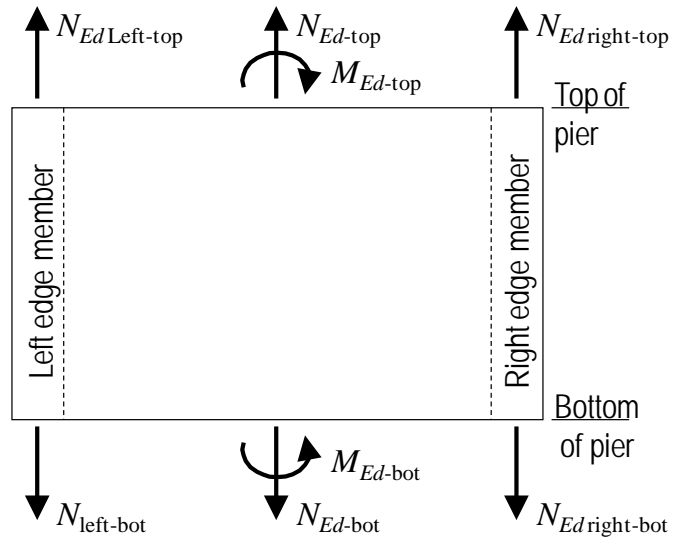
Consider the wall pier shown in Figure 2-3. For a given design section, say the top of the wall pier, the wall pier for a given design load combination is designed for a factored axial force  $N_{Ed-top}$  and a factored moment  $M_{Ed-top}$ .

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

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



**Wall Pier Plan**



**Wall Pier Elevation**

Figure 2-3 Wall Pier for Design Condition 1



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

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

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

Note that for dynamic loads,  $N_{Ed\text{ left-top}}$  and  $N_{Ed\text{ right-top}}$ , 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  $N_{Ed\text{ left-top}}$  and  $N_{Ed\text{ right-top}}$  forces are obtained first for each load case before the combinations are made.

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

$$A_{st} = \frac{N_{Ed}}{f_{yd}}$$

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

$$Abs(N_{Ed}) = (P_{\text{max Factor}}) [\eta f_{cd} (A_c - A_{sc}) + f_{yd} A_{sc}]$$

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

$$A_{sc} = \frac{\frac{Abs(N_{Ed})}{(P_{\text{max Factor}})} - \eta f_{cd}}{f_{yd} - \eta f_{cd}}$$

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

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

$$A_{st-\max} = N_{Ed} T_{\max} t_p B_1.$$

Similarly, the compression reinforcing is limited by:

$$A_{sc-\max} = N_{Ed} C_{\max} t_p B_1.$$

If  $A_{st}$  is less than or equal to  $A_{st-\max}$  and  $A_{sc}$  is less than or equal to  $A_{sc-\max}$ , the program will proceed to check the next loading combination; otherwise the program will increment the appropriate  $B_l$  dimension (left, right or both, depending on which edge member is inadequate) by one-half of the wall thickness to  $B_2$  (i.e.,  $1.5t_p$ ) and calculate new values for  $N_{Ed \text{ left-top}}$  and  $N_{Ed \text{ right-top}}$  resulting in new values of  $A_{st}$  and  $A_{sc}$ . This iterative procedure continues until  $A_{st}$  and  $A_{sc}$  are within the allowed steel ratios for all design load combinations.

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

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

### 2.1.2.2 Design Condition 2

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

### 2.1.2.3 Design Condition 3

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

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

### 2.1.3 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.

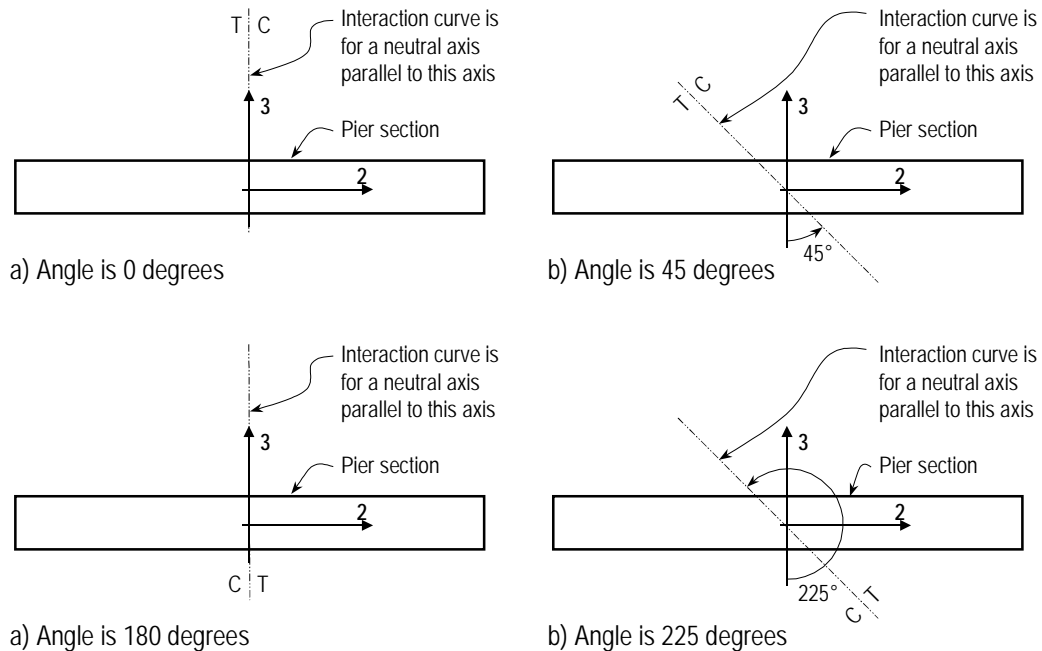
**Note:** In this program, the interaction surface is defined by a series of PMM interaction curves that are equally spaced around a 360-degree circle.

#### 2.1.3.1 Interaction Surface

In this program, a three-dimensional interaction surface is defined with reference to the  $P$ ,  $M_2$  and  $M_3$  axes. The surface is developed using a series of interaction curves that are created by rotating the direction of the pier neutral axis in equally spaced increments around a 360-degree circle. For example, if 24 PMM curves are specified (the default), there is one curve every 15 degrees ( $360^\circ/24 \text{ curves} = 15^\circ$ ). Figure 2-4 illustrates the assumed orientation of the pier neutral axis and the associated sides of the neutral axis where the section is in tension (designated T in the figure) or compression (designated C in the figure) for various angles.

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

Each PMM interaction curve that makes up the interaction surface is numerically described by a series of discrete points connected by straight lines. The coordinates of these points are determined by rotating a plane of linear strain about the neutral axis on the section of the pier. Details of this process are described later in the section entitled *Details of the Strain Compatibility Analysis*.



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

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

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

### 2.1.3.2 Formulation of the Interaction Surface

The formulation of the interaction surface in this program is based consistently on the basic principles of ultimate strength design given in Sections EC2 6.1. The program uses the requirements of force equilibrium and strain compatibility to determine the axial load and moment resistance ( $N_r$ ,  $M_{2r}$ ,  $M_{3r}$ ) of the wall pier. For

the pier to be deemed adequate, the required strength ( $N_{Ed}, M_{Ed2}, M_{Ed3}$ ) must be less than or equal to the design strength.

$$(N_{Ed}, M_{Ed2}, M_{Ed3}) \leq (N_r, M_{2r}, M_{3r})$$

The concrete compression stress block is assumed to be rectangular, with an effective strength of  $\eta f_{cd}$  (EC2 3.1.7) and effective height of  $\lambda x$ , as shown in Figure 3-1 in Chapter 3, where  $\eta$  is taken as:

$$\eta = 1.0 \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (\text{EC2 Eq. 3.21})$$

$$\eta = 1.0 - [(f_{ck} - 50)/200] \text{ for } 50 < f_{ck} \leq 90 \text{ MPa} \quad (\text{EC2 Eq. 3.22})$$

and  $\lambda$  is taken as:

$$\lambda = 0.8 \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (\text{EC2 Eq. 3.19})$$

$$\lambda = 0.8 - [(f_{ck} - 50)/400] \text{ for } 50 < f_{ck} \leq 90 \text{ MPa} \quad (\text{EC2 Eq. 3.20})$$

The effect of the material partial factors,  $\gamma_c$  and  $\gamma_s$  [NDPs], and the material coefficients,  $\alpha_{cc}$ ,  $\alpha_{ct}$ ,  $\alpha_{lcc}$ , and  $\alpha_{lct}$  [NDPs], are included in the generation of the interaction surface (EC2 3.1.6).

Default values for  $\gamma_c$ ,  $\gamma_s$ ,  $\alpha_{cc}$ ,  $\alpha_{ct}$ ,  $\alpha_{lcc}$ , and  $\alpha_{lct}$  are provided by the program but can be overwritten using the Preferences.

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

$$N_{r,max} = 0.8[\eta f_{cd} (A_c - A_s) + f_{yd} A_s].$$

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

$$N_{t,max} = f_{yd} A_s$$

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

In addition to  $N_{r,max}$  and  $N_{t,max}$ , the axial load at the balanced strain condition, i.e.,  $N_b$ , is also determined. In this condition, the tension reinforcing reaches the

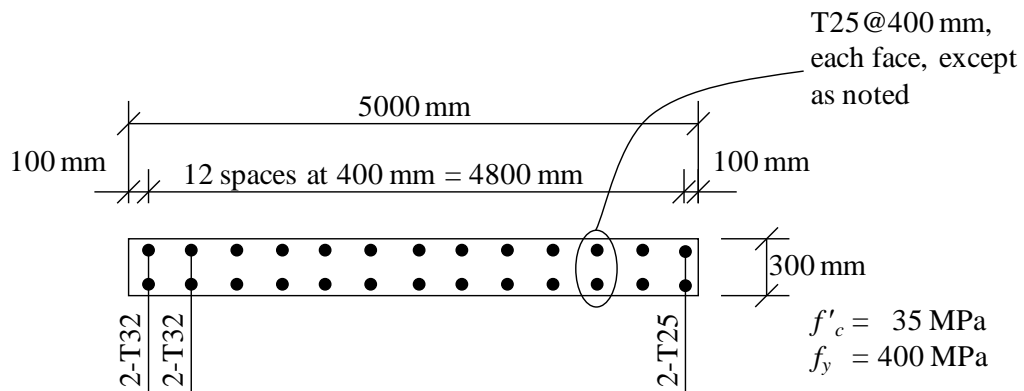
strain corresponding to its specified factored yield strength,  $f_{yd}$ , just as the concrete reaches its assumed ultimate strain of  $\epsilon_{cu3}$  (EC2 Table 3.1).

Note that  $N_{r,max}$  is reduced not only by the material safety factors but also by an additional factor of 0.80. In the preferences, this factor is called the  $P_{max}$  Factor and it can be changed if desired. In all Eurocode 2-2004 code designs, it is prudent to consider this factor to be 0.80, as required by the code.

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

As previously mentioned, by default, 11 points are used to define a single interaction curve. When creating a single interaction curve, the program includes the points at  $N_b$ ,  $N_{r,max}$  and  $N_{t,max}$  on the interaction curve. Half of the remaining number of specified points on the interaction curve occur between  $N_b$  and  $N_{r,max}$  at approximately equal spacing along the  $N_r$  axis. The other half of the remaining number of specified points on the interaction curve occur between  $N_b$  and  $N_{r,max}$  at approximately equal spacing along the  $N_t$  axis.

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



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

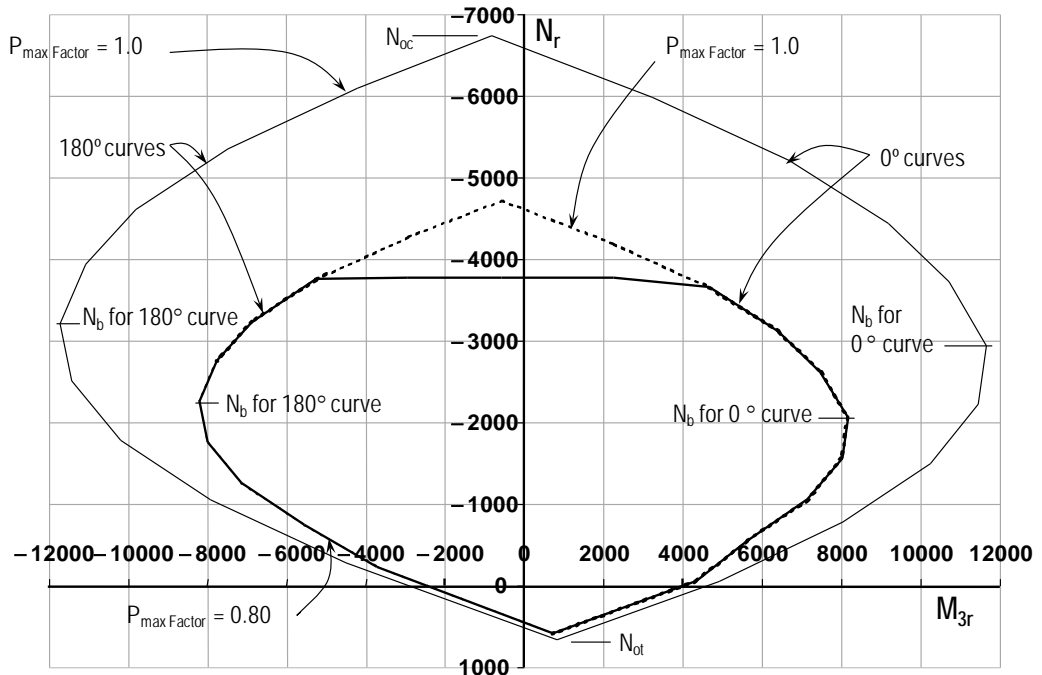
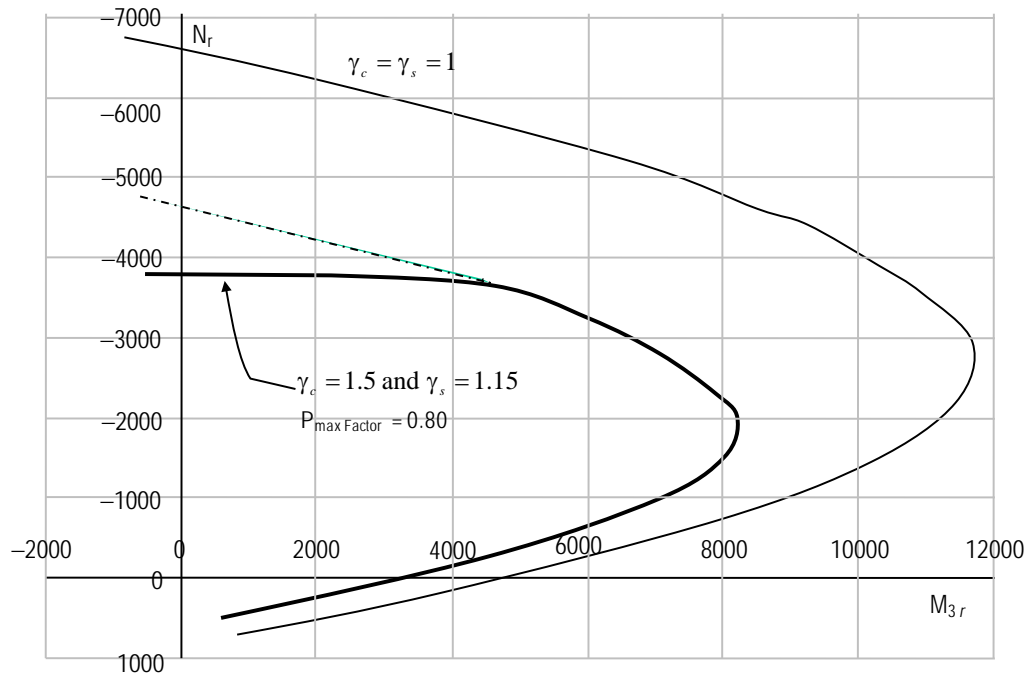


Figure 2-6 Interaction Curves for Example Wall Pier Shown in Figure 2-5

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 M<sub>3</sub> moments are considered because this is a two-dimensional example.
- In this program, compression is negative and tension is positive.
- The 0° and 180° interaction curves are not symmetric because the wall pier reinforcing is not symmetric.
- The smaller interaction surface (drawn with a heavier line) has both the strength reduction factors and the  $P_{\max}$  Factor, as specified by Eurocode 2-2004.

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.



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

- The dashed line shows the effect of setting the  $P_{\max \text{ Factor}}$  to 1.0.
- The larger interaction surface has both the strength reduction factor and the  $P_{\max \text{ Factor}}$  set to 1.0.
- The interaction surfaces shown are created using the default value of 11 points for each interaction curve.

The smaller, heavier curve in Figure 2-7 has the material reduction factor and the  $P_{\max \text{ Factor}}$  as specified in Eurocode 2-2004. The other curve, which is plotted with  $\gamma$  factors as 1.0, has an  $P_{\max \text{ Factor}}$  of 1.0. The purpose of showing these interaction curves is to explain how the program creates the interaction curve. Recall that the material strength reduction factors 1.5 and 1.15 are actually  $\gamma_c$  and  $\gamma_s$ , and that their values can be revised in the overwrites as required.



### 2.1.3.3 Details of the Strain Compatibility Analysis

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

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

In these planes, the maximum concrete strain is always taken as  $-0.0035$  and the maximum steel strain is varied from  $-0.0035$  to plus infinity. (Recall that in this program compression is negative and tension is positive.) When the steel strain is  $-0.0035$ , the maximum compressive force in the wall pier,  $N_{oc}$ , is obtained from the strain compatibility analysis. When the steel strain is plus infinity, the maximum tensile force in the wall pier,  $N_{ot}$ , is obtained. When the maximum steel strain is equal to the yield strain for the reinforcing,  $N_b$  is obtained.

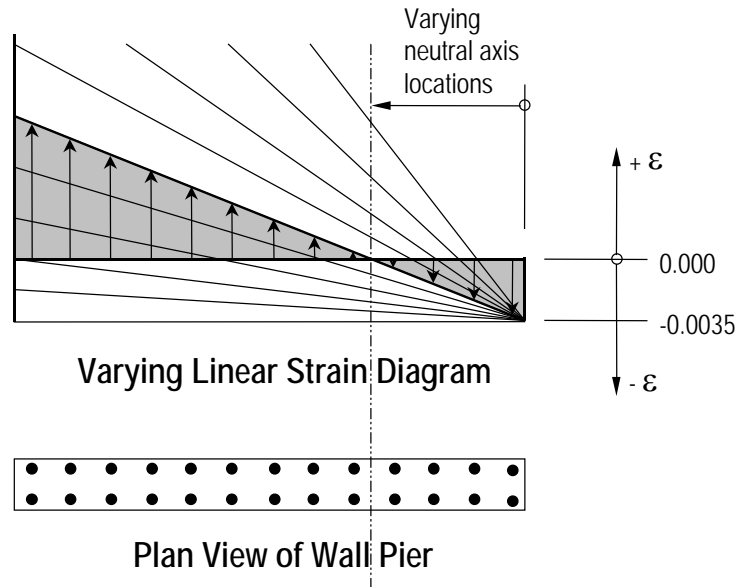
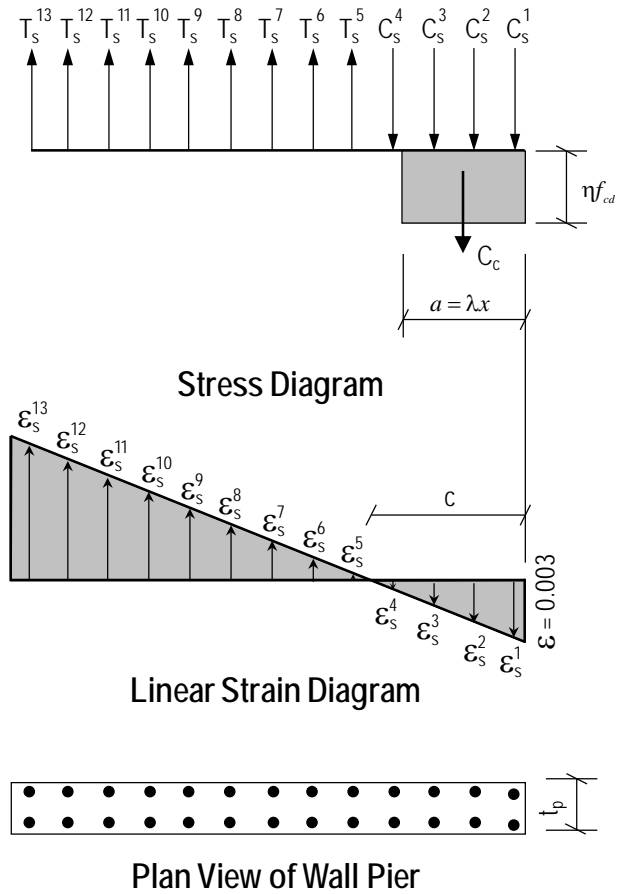


Figure 2- 8 Varying Planes of Linear Strain

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

$$C_c = (\eta f_{cd}) \lambda x t_p \quad (\text{EC2 3.1.7})$$



**Figure 2- 9 Wall Pier Stress-Strain Relationship**

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  $\varepsilon_s$  is the strain,  $E_s$  is the modulus of elasticity,  $\sigma_s$  is the stress, and  $f_{yd}$  is the yield stress of the reinforcing steel.

$$\sigma_s = \varepsilon_s E_s \leq f_{yd} \quad (\text{EC2 32.7})$$

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

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

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

$$N_r = (\sum T_s - C_c - \sum C_s) \leq N_{\max}$$

$$N_r \leq N_{o,\max} \text{ (if compression)}$$

$$N_r \leq N_{ot,\max} \text{ (if tension)}$$

In the preceding equation, the tensile force  $T_s$  and the compressive forces  $C_c$  and  $C_s$  are all positive. If  $N_r$  is positive, it is tension, and if it is negative, it is compression. The terms  $N_{oc,\max}$  and  $N_{ot,\max}$  are calculated according to EC2. The appropriate expression of these two terms was provided previously.

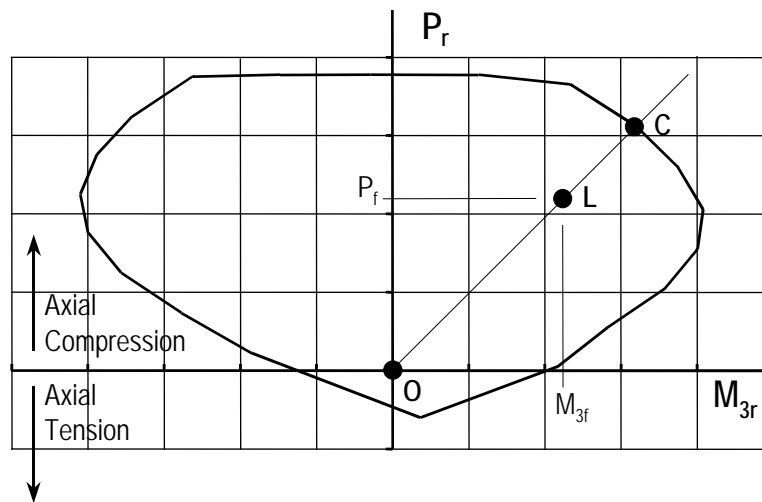
The value of  $M_2$  is calculated by summing the moments resulting from all of the forces about the pier local 2-axis. Similarly, the value of  $M_3$  is calculated by summing the moments resulting from all of the forces about the pier local 3-axis. The forces whose moments are summed to determine 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

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

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

### 2.1.4 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  $N_{Ed}$  and  $M_{3Ed}$ . The point L, defined by  $(N_{Ed}, M_{3Ed})$ , is placed on the interaction diagram, as shown in the figure. If the point lies within the interaction curve, the wall pier capacity is adequate. If the point lies outside of the interaction curve, the wall pier is overstressed.



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

As a measure of the stress condition in the wall pier, the program calculates a stress ratio. The ratio is achieved by plotting the point L and determining the location of point C. 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  $(N_{Ed}, M_{3Ed})$  lies on the interaction curve and the wall pier is stressed to capacity.
- If  $OL < OC$  (or  $D/C < 1$ ), the point  $(N_{Ed}, M_{3Ed})$  lies within the interaction curve and the wall pier capacity is adequate.

- If  $OL > OC$  (or  $D/C > 1$ ), the point  $(N_{Ed}, M_{3Ed})$  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.

### 2.1.5 Designing a General Reinforcing or Uniform Reinforcing Pier Section

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

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

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

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

The eight reinforcing ratios used are the maximum and the minimum ratios plus six more ratios. The spacing between the reinforcing ratios is calculated as an increasing arithmetic series in which the space between the first two ratios is equal to one-third of the space between the last two ratios. Table 2-1 illustrates the spacing, both in general terms and for a specific example, when the minimum reinforcing ratio, IPmin, is 0.0025 and the maximum, IPmax, is 0.02.

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

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

After the eight reinforcing ratios have been determined, the program develops interaction surfaces for all eight of the ratios using the process described earlier in the section entitled *Checking a General or Uniform Reinforcing Pier Section*.

Next, for a given design load combination, the program generates a demand/capacity ratio associated with each of the eight interaction surfaces. The program then uses linear interpolation between the eight interaction surfaces to determine the reinforcing ratio that gives 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

This section describes how the program designs concrete wall piers for shear using Eurocode 2-2004. 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.

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

1. Determine the factored forces  $N_{Ed}$ ,  $M_{Ed}$  and  $V_{Ed}$  that are acting on the leg of the wall pier section.
2. Determine the factored shear resistance,  $V_{Rd,c}$ , that can be carried by the concrete. Note that  $N_{Ed}$  and  $M_{Ed}$  are required for the calculation of  $V_{Rd,c}$ .
3. Determine the required shear reinforcing to carry the balance of the shear force.

### 2.2.1 Determine Factored Forces

The factored forces  $N_{Ed}$ ,  $M_{Ed}$  and  $V_{Ed}$  that are acting on the individual legs of the pier section are determined from the basic forces for each load case and the load combination factors. Then the leg of the wall pier is designed for the factored forces.

For Ductility Class High (DCH) Shear Walls, in addition to designing for factored shear force, each leg of the shear wall is designed for enhanced factored seismic loading. In the enhanced factored forces, the seismic load factors are multiplied by a force modification factor,  $\varepsilon$ . The force modification factor  $\varepsilon$  reflects the capacity of a structure to dissipate energy through inelastic behavior. As given in Eurocode 8, the value of  $\varepsilon$  is taken as follows (EC8, 5.5.2.4(7))

$$\varepsilon = q \sqrt{\left(\frac{\gamma_{Rd} M_{Rd}}{q M_{Ed}}\right)^2 + 0.1 \left(\frac{S_e(T_c)}{S_e(T_1)}\right)^2} \leq q \quad (\text{EC8 Eqn. 5.25})$$

$\varepsilon$  is the magnification factor (as input by user in wall preferences called Force Modification Factor), calculated from EC8 Eqn. 5.25, but  $\varepsilon > 1.5$ .

$q$  is the behavior factor used in the design.

$M_{Ed}$  is the design bending moment at the base of the wall.

$M_{Rd}$  is the design flexural resistance at the base of wall.

$\gamma_{Rd}$  is the factor account for overstrength due to steel hardening;  $\gamma_{Rd}$  may be taken equal to 1.2.

$T_1$  is the fundamental time period of vibration of the building in the direction of shear forces,  $V_{Ed}$ .

$T_c$  is the upper limit period of the constant acceleration region of spectrum.

## 2.2.2 Determine the Concrete Shear Capacity

Given the design force set  $N_{Ed}$  and  $V_{Ed}$ , the shear force that can be carried without requiring design shear reinforcement,  $V_{Rd,c}$ , is calculated as:

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_l \sigma_{cp}] t_p d \quad (\text{EC2 Eq. 6.2.a})$$

with a minimum of:

$$V_{Rd,c} = (v_{\min} + k_l \sigma_{cp}) t_p d \quad (\text{EC2 Eq. 6.2.b})$$

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

$$d = 0.8 L_p$$

where  $f_{ck}$  is in MPa, and  $k$ ,  $\rho_l$ , and  $\sigma_{cp}$  are calculated as:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (d \text{ is in mm}) \quad (\text{EC2 6.2.2(1)})$$

$$\rho_l = \frac{A_s}{t_p d} \leq 0.02 \quad \text{where } A_s \text{ is half of the pier longitudinal rebar. } (\text{EC2 6.2.2(1)})$$

$$\sigma_{cp} = N_{Ed} / A_g < 0.2 f_{cd} \quad (\text{in MPa}) \quad (\text{EC2 6.2.2(1)})$$

The factor  $k_l = 0.15$  [NDP] and the values of  $C_{Rd,c}$  [NDP] and  $v_{\min}$  [NDP] are determined as:

$$C_{Rd,c} = 0.18 / \gamma_c \quad (\text{EC2 6.2.2(1)})$$



$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2} \quad (\text{EC2 Eq. 6.3N})$$

$t_p$  is the thickness of the wall pier resisting the shear perpendicular to the shear force direction. For columns with rectangular cross-sections,  $b_w$  is taken as the width of the section perpendicular to the shear direction. For columns with circular cross-sections,  $b_w$  is taken as the average width appropriate for shear direction.

### 2.2.3 Determine the Required Shear Reinforcing

Given  $V_{Ed}$  and  $V_{Rd,c}$ , the following procedure provides the required shear reinforcing in area per unit length.

- The shear force,  $V_{Ed}$  is limited to a maximum limit,  $V_{r,\max}$ , given by

$$V_{Rd,\max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot\theta + \tan\theta) \quad (\text{EC2 Eq. 6.9})$$

The coefficient  $\alpha_{cw}$  [NDP] takes account of the state of stress in the compression chord and is taken equal to 1, which is recommended for non-prestressed structures. The strength reduction factor for concrete cracked in shear,  $v_1$  [NDP] is defined as:

$$v_1 = 0.6 [1 - f_{ck}/250] \quad (f_{ck} \text{ is in MPa}) \quad (\text{EC2 Eq. 6.6N})$$

The inner lever arm distance,  $z$ , is approximated as  $0.9d$ .

If the load combination includes seismic, the value of  $\theta$  is taken as  $45^\circ$ . However, for other cases  $\theta$  is optimized using the following relationship:

$$(\cot\theta + \tan\theta) = 0.9 \alpha_{cw} v_1 f_{cd} / V_{Ed}$$

where

$$21.8^\circ \leq \theta \leq 45^\circ$$

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

If  $V_{Ed} \leq V_{Rd,c}$ ,

$$\frac{A_{sw}}{s} = 0$$

else if  $V_{Rd,c} < V_{Ed} \leq V_{Rd,max}$

$$\frac{A_{sw}}{s} = \frac{V_{Ed} \tan \theta}{f_{yd} z} \quad (\text{EC2 Eq. 6.8})$$

else if  $V_{Ed} > V_{Rd,max}$

a failure condition is declared.

- Where the minimum shear reinforcement is required by section EC2 9.6.3, or by calculations, the minimum area of shear reinforcement per unit spacing is taken as:

$$\frac{A_{sw}}{s} \geq \begin{cases} 0.001A_g, \\ 0.25A_{sv}. \end{cases} \quad (\text{EC2 9.6.3.1})$$

- In the preceding, the term  $\theta$  is used. Here  $\theta$  is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the member. The  $\theta$  value is normally between 21.8 to 44 degrees. It is determined according to Section 11.3.6 of the Code.

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

## 2.3 Wall Pier Boundary Elements

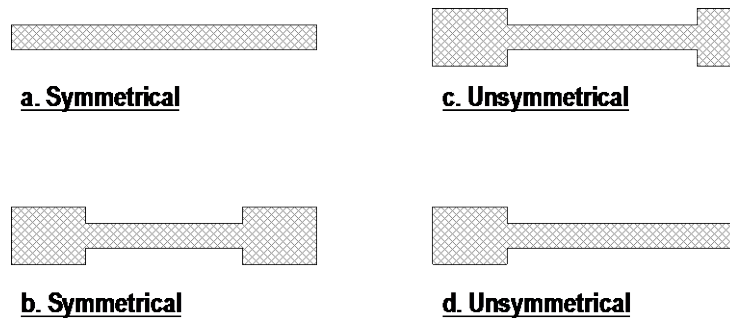
This section describes how the program considers the boundary element requirements for each leg of concrete wall piers using Eurocode 8-2004 when the DCH or Ductility Class Medium (DCM) option is chosen. The program uses an approach based on the requirements of Sections 5.4.3.4 and 5.5.3.4 of Eurocode 8-2004. The program does not compute boundary zone requirement when maximum the normalized extreme fiber compressive stress is less than 0.15 (EC8 5.4.3.4.2(12)a).

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

### 2.3.1 Details of Check for Boundary Element Requirements

The following information is available for the boundary element check:

- The design forces  $N_{Ed}$ ,  $V_{Ed}$ , and  $M_{Ed}$  for the pier section.
- The height of the entire wall,  $h_w$ , length of the wall pier,  $L_p$ , the gross area of the pier,  $A_c$ , 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 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,  $f_{cd}$  and  $f_{yd}$ .
- The symmetry of the wall pier (i.e., is the left side of the pier the same as the right side of the pier). Only the geometry of the pier is considered, not the 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 normalized compressive stress,  $v_d$ , at the extreme fiber of the concrete pier for the specified load combination (EC8 5.4.3.4.2(5)a).

$$v_d = N_{Ed} / l_w t_p f_{cd}$$

When the extreme normalized fiber compressive stress exceeds 0.15, boundary elements are required.

The neutral axis depth,  $x_u$ , is estimated based on axial force,  $N_{Ed}$ , and the total area of the vertical reinforcement,  $A_{sv}$ , using EC8 Eq. 5.21 (EC8 5.4.3.5.2(5)a).

$$\omega_v = \frac{A_{sv} \cdot f_{yd}}{l_w t_p f_{cd}}$$

$$x_u = (v_d + \omega_v) \frac{l_w t_p}{(t_p - 2\text{Cover})} \quad (\text{EC8 Eqn. 5.21})$$

And the length of boundary element,  $l_c$ , is computed using the following expression (EC8 5.4.3.4.2(6)):

$$l_c = x_u (1 - \varepsilon_{cu2} / \varepsilon_{cu2,c})$$

where,

$$\varepsilon_{cu2} = 0.0035 \quad (\text{EC8 5.4.3.4.2(6)})$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.1\alpha\omega_{\omega d} \quad (\text{EC8 5.4.3.4.2(6)})$$

$$\alpha\omega_{\omega d} = 30\mu_{\varphi} (v_d + \omega_v) \varepsilon_{sy,d} \frac{t_p}{(t_p - 2\text{Cover})} - 0.035 \quad (\text{EC8 5.4.3.4.2(6)})$$

$$\varepsilon_{sy,d} = f_{yd} / E_s$$

$\mu_{\varphi}$  is the required value of the curvature ductility factor as input by the user in the wall preferences (EC8 5.4.3.4.2(2)).

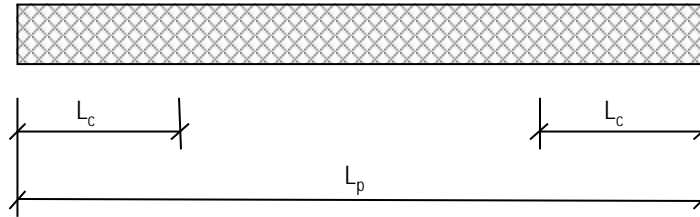
$$\mu_{\varphi} = 2q_o - 1 \quad \text{when } T_l \geq T_c \quad (\text{EC8 Eqn. 5.4})$$

$$\mu_{\varphi} = 1 + 2(q_o - 1)T_c / T_l \quad \text{when } T_l < T_c \quad (\text{EC8 Eqn. 5.5})$$

If boundary elements are required, the program calculates the minimum required length of the boundary zone at each end of the wall,  $L_{BZ}$ , in accordance with the requirements of Section 5.4.3.4.2(6) in Eurocode 8-2004.

The computed boundary length,  $l_c$ , must be greater than  $0.15l_w$  or  $1.50t_p$  (EC8 5.4.3.4.2(6)). Figure 2-12 illustrates the boundary zone length  $l_c$ .

$$l_c = \max \{ 0.15l_w, 1.5t_p \} \quad (\text{EC8 5.4.3.4.2(6)})$$



*Figure 2- 12 Illustration of Boundary Zone Length,  $L_c$*

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## Chapter 3 Spandrel Design

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This chapter describes how the program designs concrete shear wall spandrels for flexure and shear when Eurocode 2-2004 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  $M_{Ed}$ .
- Determine the required flexural reinforcing.

These steps are described in the following sections.

### 3.1.1 Determine the Maximum Factored Moments

In the design of flexural reinforcing for spandrels, the factored moments for each design load combination at a particular beam station are first obtained.

The beam section is then designed for the maximum positive and the maximum negative factored moments obtained from all of the design load combinations.

### 3.1.2 Determine the Required Flexural Reinforcing

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

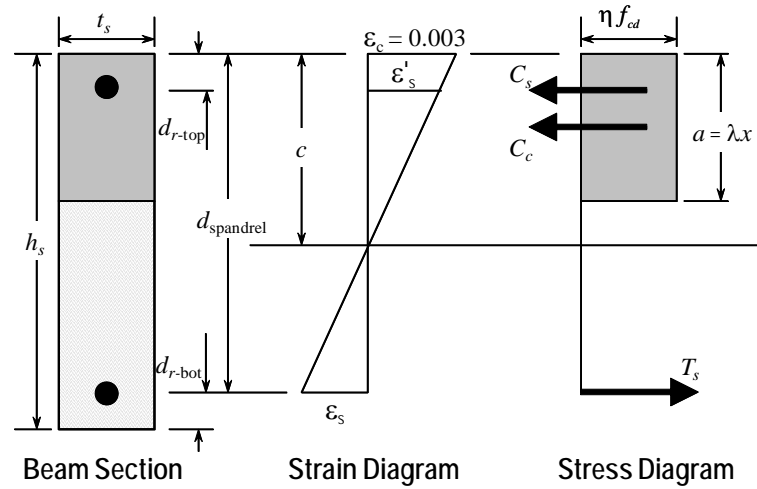
In this program, positive beam moments produce bottom steel. In such cases, the beam may be designed as a rectangular section, or as a T-beam section. Indicate that a spandrel is to be designed as a T-beam by specifying the appropriate slab width and depth dimensions in the spandrel design overwrites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

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

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

#### 3.1.2.1 Rectangular Beam Flexural Reinforcing

Refer to Figure 3-1. For a rectangular beam, the factored moment,  $M_{Ed}$ , is resisted by a couple between the concrete in compression and the tension in reinforcing steel. This is expressed as given below.



**Figure 3-1 Rectangular Spandrel Beam Design, Positive Moment**

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

where  $C_c = \eta f_{cd} a t_s$  and  $d_{\text{spandrel}}$  is equal to  $h_s - d_{r\text{-bot}}$  for positive bending and  $h_s - d_{r\text{-top}}$  for negative bending.

In designing for a factored negative or positive moment,  $M_{Ed}$  (i.e., designing top or bottom steel), the effective strength and depth of the compression block are given by  $\eta f_{cd}$  and  $\lambda x$  (see Figure 3-1) respectively, where:

$$\lambda = 0.8 \text{ for } f_{ck} \leq 50 \text{ MPa,} \quad (\text{EC2 Eq. 3.19})$$

$$\lambda = 0.8 - \left[ (f_{ck} - 50) / 400 \right] \text{ for } 50 < f_{ck} \leq 90 \text{ MPa,} \quad (\text{EC2 Eq. 3.20})$$

$$\eta = 1.0 \text{ for } f_{ck} \leq 50 \text{ MPa,} \quad (\text{EC2 Eq. 3.21})$$

$$\eta = 1.0 - \left[ (f_{ck} - 50) / 200 \right] \text{ for } 50 < f_{ck} \leq 90 \text{ MPa,} \quad (\text{EC2 Eq. 3.22})$$

where  $x$  is the depth of the neutral axis,  $\lambda$  is a factor defining the effective height of the compression zone, and  $\eta$  is a factor defining the effective strength.



In designing for a factored negative or positive moment,  $M_{Ed}$  (i.e., designing top or bottom steel), the depth of the compression block is given by  $a$ .

$$a = d_{\text{spandrel}} - \sqrt{d_{\text{spandrel}}^2 - \frac{2M_{Ed}}{\eta f_{cd} t_s}}$$

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

The depth of the compression block,  $a$ , is compared with  $\lambda x_{\text{limiting}}$  (EC2 5.5). The limiting value of the ratio of the neutral axis depth at the ultimate limit state to the effective depth,  $(x/d)_{\text{lim}}$ , is expressed as a function of the ratio of the redistributed moment to the moment before redistribution,  $\delta$ , as follows:

$$(x/d)_{\text{lim}} = (\delta - k_1)/k_2 \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (\text{EC2 Eq. 5.10a})$$

or, 
$$x_{\text{lim}} = d(\delta - k_1)/k_2 \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (\text{EC2 Eq. 5.10a})$$

or, 
$$x_{\text{lim}} = d(\delta - k_3)/k_4 \text{ for } f_{ck} > 50 \text{ MPa} \quad (\text{EC2 Eq. 5.10b})$$

No redistribution is assumed, such that  $\delta$  is assumed to be 1. The four factors,  $k_1$ ,  $k_2$ ,  $k_3$ , and  $k_4$  [NDPs], are defined as:

$$k_1 = 0.44 \quad (\text{EC2 5.5(4)})$$

$$k_2 = 1.25(0.6 + 0.0014/\varepsilon_{cu2}) \quad (\text{EC2 5.5(4)})$$

$$k_3 = 0.54 \quad (\text{EC2 5.5(4)})$$

$$k_4 = 1.25(0.6 + 0.0014/\varepsilon_{cu2}) \quad (\text{EC2 5.5(4)})$$

where the ultimate strain,  $\varepsilon_{cu2}$  [NDP], is determined from EC2 Table 3.1 as:

$$\varepsilon_{cu2} = 0.0035 \text{ for } f_{ck} < 50 \text{ MPa} \quad (\text{EC2 Table 3.1})$$

$$\varepsilon_{cu2} = 2.6 + 35 \left[ \frac{(90 - f_{ck})}{100} \right]^4 \text{ for } f_{ck} \geq 50 \text{ MPa} \quad (\text{EC2 Table 3.1})$$

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

### 3.1.2.1.1 Tension Reinforcing Only Required

- If  $a \leq \lambda x_{\text{limiting}}$  (EC2 5.5), the area of tensile steel reinforcement is then given by:

$$A_s = \frac{M_{Ed}}{f_{yd} \left( d_{\text{spandrel}} - \frac{a}{2} \right)}$$

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  $a > \lambda x_{\text{limiting}}$  (EC2 5.5), compression reinforcement is required and is calculated as follows.

The depth of the concrete compression block,  $a$ , is set equal to  $a_b = \lambda x_{\text{limiting}}$ . The compressive force developed in the concrete alone is given by

$$C_c = \eta f_{cd} t_s a_b$$

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

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

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

$$M_{Eds} = M_{Ed} - M_{Edc}$$

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

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

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

$$\varepsilon'_s = \frac{0.0035(x - d_r)}{x}$$

The stress in the compression steel,  $f'_{sd}$ , is given by

$$f'_{sd} = E_s \varepsilon'_s = \frac{0.0035 E_s (x - d_r)}{x} \leq f_{yd}$$

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  $\lambda x_{\text{limiting}}$ .

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

$$A'_s = \frac{C_s}{(f'_{sd} - \eta f_{cd})}$$

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

$$A_{sw} = \frac{M_{Edc}}{f_{yd} \left( d_{\text{spandrel}} - \frac{a_b}{2} \right)}$$

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

$$A_{sc} = \frac{M_{Eds}}{f_{yd} (d_{\text{spandrel}} - d_r)}$$

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

The total tension reinforcement  $A_s$  is given by:

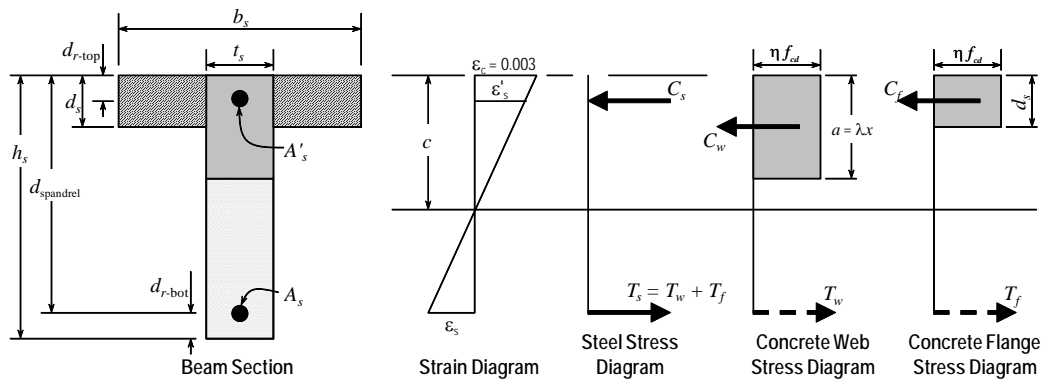
$$A_s = A_{sw} + A_{sc}$$

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

### 3.1.2.2 T-Beam Flexural Reinforcing

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

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



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

- If  $a \leq d_s$ , the subsequent calculations for the reinforcing steel are exactly the same as previously defined for rectangular section design. However, in that case, the width of the compression block is taken to be equal to the width of the compression flange,  $b_s$ . Compression reinforcement is provided when the dimension "a" exceeds  $\lambda x_{\text{limiting}}$ , where  $\lambda$  and  $x_{\text{limiting}}$  are given by previous equations for rectangular beams.

- 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:

$$C_f = \eta f_{cd} (b_s - t_s) d_s$$

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:

$$A_{sf} = \frac{C_f}{f_{yd}}$$

The portion of the total moment,  $M_{Ed}$ , that is resisted by the flange,  $M_{Edf}$ , is given by:

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

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

$$M_{Edw} = M_{Ed} - M_{Edf}$$

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:

$$a_1 = d_{\text{spandrel}} - \sqrt{d_{\text{spandrel}}^2 - \frac{2M_{Edw}}{\eta f_{cd} t_s}}$$

### 3.1.2.2.1 Tension Reinforcing Only Required

If  $a_l \leq \lambda x_{\text{limiting}}$  (EC 5.5), 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.

$$A_{sw} = \frac{M_{Edw}}{f_{yd} \left( d_{\text{spandrel}} - \frac{a_l}{2} \right)}$$

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  $a_l > \lambda x_{\text{limiting}}$  (EC 5.5), compression reinforcing is required. In that case, the required reinforcing is computed as follows.

The depth of the concrete compression block,  $a$ , is set equal to  $a_b = \lambda x_{\text{limiting}}$  (EC 5.5). The compressive force developed in the web concrete alone is given by:

$$C_w = \eta f_{cd} a t_s$$

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

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

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

$$M_{Eds} = M_{Edw} - M_{Edc}$$

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

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

The strain in the compression steel,  $\varepsilon_s'$ , is given by:

$$\varepsilon_s' = \frac{0.0035(x - d_{r\text{-top}})}{x}$$

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

$$f_{sd}' = E_s \varepsilon_s' = \frac{0.0035 E_s (x - d_{r\text{-top}})}{x} \leq f_{yd}$$

The term  $x$  is equal to  $a_{\text{limiting}}/\lambda$ .

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

$$A_s' = \frac{C_s}{f_{sd}' - \eta f_{cd}}$$

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

$$A_{sw} = \frac{M_{Edc}}{f_{yd} \left( d_{\text{spandrel}} - \frac{a}{2} \right)}$$

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

$$A_{sc} = \frac{M_{Eds}}{f_{yd} (d_{\text{spandrel}} - d_{r\text{-top}})}$$

The total tension reinforcement  $A_s$  is given by:

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

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

### 3.1.2.2.3 Minimum and Maximum Tensile Reinforcement

The minimum flexural tensile steel reinforcement,  $A_{s,\min}$  [NDP], required in a beam section is given as the maximum of the following two values:

$$A_{s,\min} = 0.26 \left( f_{ctm} / f_{yk} \right) t_s d_{\text{spandrel}} \quad (\text{EC2 Eq. 9.1N})$$

$$A_{s,\min} = 0.0013 t_s d_{\text{spandrel}}$$

where  $t_s$  is the mean width of the tension zone, equal to the web width for T-beams, and  $f_{ctm}$  is the mean value of axial tensile strength of the concrete, calculated as:

$$f_{ctm} = 0.30 f_{ck}^{(2/3)} \quad \text{for } f_{ck} \leq 50 \text{ MPa} \quad (\text{EC2 Table 3.1})$$

$$f_{ctm} = 2.12 \ln(1 + f_{ck}/10) \quad \text{for } f_{ck} > 50 \text{ MPa} \quad (\text{EC2 Table 3.1})$$

$$f_{cm} = f_{ck} + 8 \text{ Mpa.}$$

The maximum flexural steel reinforcement,  $A_{s,\max}$  [NDP], permitted as either tension or compression reinforcement is defined as:

$$A_{s,\max} = 0.04 A_c \quad (\text{EC2 9.2.1.1(3)})$$

where  $A_c$  is the gross cross-sectional area.

## 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  $V_{Ed}$ .
2. Determine the shear resistance,  $V_{Rd,c}$ , that can be carried by the concrete.
3. 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 Steps 2 and 3.

### 3.2.1 Determine Factored Forces

The factored forces  $N_{Ed}$ ,  $M_{Ed}$ , and  $V_{Ed}$  that are acting on the spandrel section are determined from the basic forces for each load case and the load combination factors. Then the spandrel is designed for the factored forces.

In addition to designing for the factored shear force, the spandrel section is designed for enhanced factored forces if the design load combination involves any seismic loading. In the enhanced factored forces, the seismic load factors are multiplied by a force modification factor. The force modification factor is a user defined parameter that should be specified in Shear Wall Design Preferences for Eurocode 2-2004 code. This modification factor reflects the capability of a structure to dissipate energy through inelastic behaviors.

### 3.2.2 Determine the Concrete Shear Capacity

Given the design force set  $M_{Ed}$  and  $V_{Ed}$ , the shear capacity provided by the concrete alone,  $V_{Rd,c}$ , is calculated as follows:

$$V_{Rd,c} = \left[ C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_1 \sigma_{cp} \right] t_s d \quad (\text{EC2 Eq. 6.2.a})$$

with a minimum of:

$$V_{Rd,c} = (v_{\min} + k_1 \sigma_{cp}) t_s d \quad (\text{EC2 Eq. 6.2.b})$$

where  $f_{ck}$  is in MPa, and  $k$ ,  $\rho_l$ , and  $\sigma_{cp}$  are calculated as:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (d \text{ is in mm}) \quad (\text{EC2 6.2.2(1)})$$

$$\rho_l = \frac{A_s}{b_w d} \leq 0.02 \quad (\text{EC2 6.2.2(1)})$$

$$\sigma_{cp} = N_{Ed}/A_c < 0.2f_{cd} \quad (\text{in MPa}) \quad (\text{EC2 6.2.2(1)})$$

The effective shear area,  $A_c$ , is taken as  $t_s d$ . The factor  $k_l = 0.15$  [NDP] and the values of  $C_{Rd,c}$  [NDP] and  $v_{\min}$  [NDP] are determined as:

$$C_{Rd,c} = 0.18/\gamma_c \quad (\text{EC2 6.2.2(1)})$$

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2} \quad (\text{EC2 Eq. 6.3N})$$

### 3.2.3 Determine Required Shear Reinforcement

To prevent crushing of the concrete compression struts, the design shear force  $V_{Ed}$  is limited by the maximum sustainable design shear force,  $V_{Rd,\max}$ . If the design shear force exceeds this limit, a failure condition occurs. The maximum sustainable shear force is defined as:

$$V_{Rd,\max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot\theta + \tan\theta). \quad (\text{EC2 Eq. 6.9})$$

The coefficient  $\alpha_{cw}$  [NDP] takes account of the state of stress in the compression chord and is taken equal to 1, which is recommended for non-prestressed structures. The strength reduction factor for concrete cracked in shear,  $v_1$  [NDP] is defined as:

$$v_1 = 0.6[1 - f_{ck}/250] \quad (f_{ck} \text{ is in MPa}). \quad (\text{EC2 Eq. 6.6N})$$

The inner lever arm distance,  $z$ , is approximated as  $0.9d$ .

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

If  $V_{Ed}$  is greater than  $V_{Rd,c}$  and less than  $V_{Rd,\max}$ , the required shear reinforcement in the form of stirrups or ties per unit spacing,  $A_{sw}/s$ , is calculated as:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z f_{ywd} \cot \theta}. \quad (\text{EC2 Eq. 6.8})$$

The computation of  $\theta$  is given in a preceding section.

The maximum of all the calculated  $A_{sw}/s$  values, obtained from each design load combination, is reported for the beam, along with the controlling combination name. The calculated shear reinforcement must be greater than the minimum reinforcement ratio of:

$$\rho_{w,\min} = (0.08 \sqrt{f_{ck}}) / f_{yk}. \quad (\text{EC2 Eq. 9.5N})$$

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

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

### 3.2.3.1 Seismic Spandrels Only

For seismic spandrels only, in addition to the requirements of the previous subsection, an area of diagonal shear reinforcement,  $A_{vd}$ , in coupling beams is also calculated when  $\frac{L_s}{d_{\text{spandrel}}} \leq 3$  or  $V_{Ed} > f_{ctd} t_s d$  using the following equation

(EC8 5.5.3.5):

$$A_{vd} = \frac{V_{Ed}}{2 f_{yd} \sin \alpha}, \quad (\text{EC8 Eqn. 5.49})$$

where

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

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

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## Appendix A

### Shear Wall Design Preferences

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The shear wall design preferences are basic properties that apply to all wall pier and spandrel elements. Table A1 identifies shear wall design preferences for Eurocode 2-2004. Default values are provided for all shear wall design preference items. Thus, it is not required that preferences be specified. However, at least review the default values for the preference items to make sure they are acceptable. Refer to the program Help for an explanation of how to change a preference.

**Table A1 Shear Wall Preferences**

<b>Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Description</b>
Design Code	Any code in the program	Eurocode 2-2004	Design code used for design of concrete shear wall elements (wall piers and spandrels)
Rebar Material		A615Gr60	Assumed self-explanatory.
Rebar Shear Material		A615Gr60	Assumed self-explanatory.

**Table A1 Shear Wall Preferences**

Item	Possible Values	Default Value	Description
Country	Denmark, Finland, Germany, Norway, Poland, Portugal, Singapore, Slovenia, Sweden United Kingdom	CEN Default	
Combination Equations	Eq. 6.10; Max of Eq. 6.10a/6.10b	Eq. 6.10	
Reliability Class	Class 1, Class 2, Class 3	Class 2	
Second Order Method	Nominal Stiffness, Nominal Curvature, None	Nominal Stiffness	
Gamma (Steel)	> 0	1.5	Material partial factor for steel (EC2 2.4.2.4).
Gamma (Concrete)	> 0	1.15	Material partial factor for concrete (EC2 2.4.2.4).
Gamma (Shear)	> 0	1.25	
Number of Curves	$\geq 4$	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 24 for this item.
Number of Points	$\geq 11$	11	Number of points used to define a single curve in a wall pier interaction surface (this item should be odd).
Edge Design PT-max	> 0	0.06	Maximum ratio of tension reinforcing allowed in edge members, $PT_{max}$ .
Edge Design PC-max	> 0	0.04	Maximum ratio of compression reinforcing allowed in edge members, $PC_{max}$ .
Section Design IP-Max	$\geq$ Section Design IP-Min	0.02	The maximum ratio of reinforcing considered in the design of a pier with a Section Designer section.

**Table A1 Shear Wall Preferences**

<b>Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Description</b>
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

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The shear wall design overwrites are basic assignments that apply only to those piers or spandrels to which they are assigned. The overwrites for piers and spandrels are separate. Tables B1 and B2 identify the shear wall overwrites for piers and spandrels, respectively, for Eurocode 2-2004. 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**

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



**Table B-1: Pier Design Overwrites**

<b>Pier Overwrite Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Pier Overwrite Description</b>
LL Reduction Factor	Program calculated, > 0	Program calculated	A reducible live load is multiplied by this factor to obtain the reduced live load. Entering 0 for this item means that it is program calculated. See the subsection entitled "LL Reduction Factor" for more information.
Design is Seismic?	Yes or No	Yes	Toggle for design as seismic or nonseismic. Additional design checks are performed for seismic elements compared to nonseismic elements. Also, in some cases, the strength reduction factors are different.
<b>Seismic Design Grade</b>			
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.
<b>Overwrites Applicable to Uniform Reinforcing Pier Sections</b>			
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.
<b>Overwrites Applicable to General Reinforcing Pier Sections</b>			
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.
<b>Overwrites Applicable to Simplified T and C Pier Sections</b>			
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	$\geq 0$	0	Length of the bottom of a user-defined edge member on the left side of a wall pier, $DB1_{left}$ .
DB2LeftBot	$\geq 0$	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	$\geq 0$	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	$\geq 0$	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	$\geq 0$	0	Length of the top of a user-defined edge member on the left side of a wall pier, $DB1_{left}$ .
DB2LeftTop	$\geq 0$	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	$\geq 0$	Same as DB1-left-bot	Length of the top of a user-defined edge member on the right side of a wall pier, DB1 <sub>right</sub> .
DB2RightTop	$\geq 0$	Same as DB2-left-bot	Width of the top of a user-defined edge member on the right side of a wall pier, DB2 <sub>right</sub> .
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 <sub>max</sub> .
Edge Design PT-max	$> 0$	Specified in Preferences	Maximum ratio of tension reinforcing allowed in edge members, PT <sub>max</sub> .

**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 <b>Design menu &gt; Shear Wall Design &gt; Start Design/Check</b> 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 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.
Seismic Design Grade			

**Table B-2 Spandrel Design Overwrites**

<b>Spandrel Overwrite Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Spandrel Overwrite Description</b>
Length	Program calculated, or $> 0$	Program calculated	Wall spandrel length, $L_s$ . Inputting 0 means the item is to be program calculated.
ThickLeft	Program calculated, or $> 0$	Program calculated	Wall spandrel thickness at left side of spandrel, $t_s$ . Inputting 0 means the item is to be program calculated.
DepthLeft	Program calculated, or $> 0$	Program calculated	Wall spandrel depth at left side of spandrel, $h_s$ . Inputting 0 means the item is to be program calculated.
Cover Bottom Left	Program calculated, or $> 0$	Program calculated	Distance from bottom of spandrel to centroid of bottom reinforcing, $d_{r-bot\ left}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$ .
Cover Top Left	Program calculated, or $> 0$	Program calculated	Distance from top of spandrel to centroid of top reinforcing, $d_{r-top\ left}$ on left side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$ .
Slab Width Left	$\geq 0$	0	Slab width for T-beam at left end of spandrel, $b_s$ .
Slab Depth Left	$\geq 0$	0	Slab depth for T-beam at left end of spandrel, $d_s$ .
Thick Right	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.
Depth Right	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.
Cover Bottom Right	Program calculated, or $> 0$	Program calculated	Distance from bottom of spandrel to centroid of bottom reinforcing, $d_{r-bot\ right}$ on right side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$ .
Cover-Top Right	Program calculated, or $> 0$	Program calculated	Distance from top of spandrel to centroid of top reinforcing, $d_{r-top\ right}$ on right side of beam. Inputting 0 means the item is to be program calculated as $0.1h_s$ .
Slab Width Right	$\geq 0$	0	Slab width for T-beam at right end of spandrel, $b_s$ .
Slab Depth Right	$\geq 0$	0	Slab depth for T-beam at right end of spandrel, $d_s$ .

**Table B-2 Spandrel Design Overwrites**

<b>Spandrel Overwrite Item</b>	<b>Possible Values</b>	<b>Default Value</b>	<b>Spandrel Overwrite Description</b>
Material	Any defined concrete material property		Material property associated with 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, the user 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.

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## Appendix C

### Nationally Determined Parameters (NDPs)

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The Comité Européen de Normalisation (CEN) version of Eurocode 2-2004 specifies a set of clauses in the design code, for which Nationally Determined Parameters (NDPs) are permitted to be adjusted by each member country within their National Annex. Variations in these parameters between countries are considered in the program by choosing the desired country in the Preferences. This appendix lists the NDPs as adopted in the program for the CEN Default version of the design code. Additional tables are provided that list the NDPs that differ from the CEN Default values for each country supported in the program.

**Table C-1 CEN Default NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
$\gamma_c$	2.4.2.4(1)	1.5
$\gamma_s$	2.4.2.4(1)	1.15
$\alpha_{cc}$	3.1.6(1)	1.0
$\alpha_{ct}$	3.1.6(2)	1.0

**Table C-1 CEN Default NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
$\max f_{yk}$	3.2.2(3)	600MPa
Load Combinations	5.1.3(1)	Combinations from Eq. 6.10
$\theta_0$	5.2(5)	0.005
$k_1$	5.5(4)	0.44
$k_2$	5.5(4)	$1.25(0.6 + 0.0014/\epsilon_{cu2})$
$k_3$	5.5(4)	0.54
$k_4$	5.5(4)	$1.25(0.6 + 0.0014/\epsilon_{cu2})$
$\lambda_{lim}$	5.8.3.1(1)	$20 \cdot A \cdot B \cdot C / \sqrt{n}$
Second Order Analysis Method	5.8.5(1)	Nominal Curvature
$C_{Rd,c}$	6.2.2(1)	$0.18/\gamma_c$
$\nu_{min}$	6.2.2(1)	$0.035k^{3/2}f_{ck}^{-1/2}$
$k_1$	6.2.2(1)	0.15
Cot $\theta$	6.2.3(2)	$1 \leq \text{Cot}\theta \leq 2.5$
$\nu_1$	6.2.3(3)	$0.6 \left[ 1 - \frac{f_{ck}}{250} \right]$
$\alpha_{cw}$	6.2.3(3)	1.0
Beam $A_{s,min}$	9.2.1.1(1)	$0.26 \frac{f_{ctm}}{f_{yk}} b_1 d \geq 0.0013 b_1 d$
Beam $A_{s,max}$	9.2.1.1(3)	0.04A <sub>c</sub>

**Table C-1 CEN Default NDPs**

NDP	Clause	Value
Beam $\rho_{w,\min}$	9.2.2(5)	$(0.08\sqrt{f_{ck}}) / f_{yk}$
Column $A_{s,\min}$	9.5.2(2)	$\frac{0.10N_{Ed}}{f_{yd}} \geq 0.002A_c$
Column $A_{s,\max}$	9.5.2(3)	$0.04A_c$
$\alpha_{cc}$	11.3.5(1)	0.85
$\alpha_{ct}$	11.3.5(2)	0.85
$C_{Ird,c}$	11.6.1(1)	$0.15/\gamma_c$
$v_{l,\min}$	11.6.1(1)	$0.30k^{3/2}f_{ck}^{1/2}$
$k_1$	11.6.1(1)	0.15
$v_1$	11.6.2(1)	$0.5\eta_1(1 - f_{ck}/250)$

**Table C-2 United Kingdom NDPs**

NDP	Clause	Value
$\psi_{0,i}$ (wind load)	EC0 Combos	0.5
$\alpha_{cc}$	3.1.6(1)	0.85
$k_1$	5.5(4)	0.4
$k_2$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$k_3$	5.5(4)	0.4



**Table C-2 United Kingdom NDPs**

NDP	Clause	Value
$k_4$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$v_{l,min}$	11.6.1(1)	$0.30k^{3/2}f_{ck}^{1/2}$

**Table C-3 Slovenia NDPs**

NDP	Clause	Value
Column $A_{s,min}$	9.5.2(2)	$\frac{0.15N_{Ed}}{f_{yd}} \geq 0.003A_c$

**Table C-4 Norway NDPs**

NDP	Clause	Value
$\alpha_{cc}$	3.1.6(1)	0.85
$\alpha_{ct}$	3.1.6(2)	0.85
$\lambda_{lim}$	5.8.3.1(1)	$13(2 - r_m)A_f$
$k_1$	6.2.2(1)	0.15 for compression 0.3 for tension
Beam $\rho_{w,min}$	9.2.2(5)	$(0.1\sqrt{f_{ck}})/f_{yk}$
Column $A_{s,min}$	9.5.2(2)	$\frac{0.5N_{Ed}}{f_{yd}} \geq \frac{0.2A_c f_{cd}}{f_{yd}}$ but not less than $0.01A_c$
Column $A_{s,max}$	9.5.2(3)	$0.08A_c$

**Table C-4 Norway NDPs**

NDP	Clause	Value
$v_{l,min}$	11.6.1(1)	$0.028k^{3/2}f_{ck}^{1/2}$
$k_1$	11.6.1(1)	0.15 for compression 0.3 for tension
$v_1$	11.6.2(1)	$0.5(1 - f_{ck}/250)$

**Table C-5 Singapore NDPs**

NDP	Clause	Value
$\alpha_{cc}$	3.1.6(1)	0.85
$k_1$	5.5(4)	0.4
$k_2$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$k_3$	5.5(4)	0.54
$k_4$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$v_{lim}$	5.8.3.1(1)	$0.30k^{3/2}f_{ck}^{1/2}$

**Table C-6 Sweden NDPs**

NDP	Clause	Value
Beam $A_{s,max}$	9.2.1.1(3)	Unlimited
Column $A_{s,min}$	9.5.2(2)	$0.002A_c$

**Table C-6 Sweden NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
Column $A_{s,max}$	9.5.2(3)	Unlimited
$\alpha_{cc}$	11.3.5(1)	1.0
$\alpha_{ct}$	11.3.5(2)	1.0

**Table C-7 Finland NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
$\alpha_{cc}$	3.1.6(1)	0.85
Max $f_{yk}$	3.2.2(3)	700MPa
Load Combinations	5.1.3(1)	Combinations from Eq. 6.10a/b
$k_2$	5.5(4)	1.10
Beam $A_{s,max}$	9.2.1.1(3)	Unlimited
Column $A_{s,max}$	9.5.2(3)	0.06A <sub>c</sub>

**Table C-8 Denmark NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
$\gamma_c$	2.4.2.4(1)	1.45
$\gamma_s$	2.4.2.4(1)	1.20
Max $f_{yk}$	3.2.2(3)	650MPa

**Table C-8 Denmark NDPs**

NDP	Clause	Value
Load Combinations	5.1.3(1)	Combinations from Eq. 6.10a/b
$\lambda_{lim}$	5.8.3.1(1)	$20 \cdot \sqrt{\frac{A_c f_{cd}}{N_{Ed}}}$
Second Order Analysis Method	5.8.5(1)	Nominal Stiffness
Beam $\rho_{w,min}$	9.2.2(5)	$(0.063\sqrt{f_{ck}}) / f_{yk}$
$\alpha_{cc}$	11.3.5(1)	1.0
$\alpha_{ct}$	11.3.5(2)	1.0
$v_{l,min}$	11.6.1(1)	$0.03k^{3/2}f_{ck}^{1/2}$

**Table C-9 Portugal NDPs**

NDP	Clause	Value
Max $f_{yk}$	3.2.2(3)	500MPa

**Table C-10 Germany NDPs**

NDP	Clause	Value
$\alpha_{cc}$	3.1.6(1)	0.85
$\alpha_{ct}$	3.1.6(2)	0.85
max $f_{yk}$	3.2.2(3)	500MPa
$\theta_0$	5.2(5)	$0 \leq \alpha_{\theta} \leq 1.0$

**Table C-10 Germany NDPs**

<b>NDP</b>	<b>Clause</b>	<b>Value</b>
$k_1$	5.5(4)	0.64
$k_2$	5.5(4)	0.8
$k_3$	5.5(4)	0.72
$k_4$	5.5(4)	0.8
$\lambda_{lim}$	5.8.3.1(1)	$25 \text{ for }  n  \geq 0.41$ $16/\sqrt{n} \text{ for }  n  < 0.41$
Second Order Analysis Method	5.8.5(1)	Nominal Stiffness
$C_{Rd,c}$	6.2.2(1)	$0.15/\gamma_c$
$\nu_{min}$	6.2.2(1)	$(0.0525/\gamma_c)k^{3/2}f_{ck}^{1/2} \text{ for } d \leq 600mm$ $(0.0375/\gamma_c)k^{3/2}f_{ck}^{1/2} \text{ for } d > 800mm$
$k_1$	6.2.2(1)	0.12
Cot $\theta$	6.2.3(2)	$1 \leq \text{Cot}\theta \leq 3.0$
$\nu_1$	6.2.3(3)	$0.75 \nu_2$ where $\nu_2 = (1.1 \cdot f_{ck}/500) \leq 1.0$
Beam $A_{s,max}$	9.2.1.1(3)	$0.08A_c$
Beam $\rho_{w,min}$	9.2.2(5)	$0.16f_{ctm}/f_{yk}$
Column $A_{s,min}$	9.5.2(2)	$0.15 N_{Ed}  / f_{yd}$
$\alpha_{cc}$	11.3.5(1)	0.75
$\nu_{l,min}$	11.6.1(1)	$(0.0525/\gamma_c)k^{3/2}f_{lck}^{1/2} \text{ for } d \leq 600mm$ $(0.0375/\gamma_c)k^{3/2}f_{lck}^{1/2} \text{ for } d > 800mm$

**Table C-10 Germany NDPs**

NDP	Clause	Value
$k_1$	11.6.1(1)	0.12
$v_1$	11.6.2(1)	$0.75\eta_1$

**Table C-11 Poland NDPs**

NDP	Clause	Value
$\gamma_c$	2.4.2.4(1)	1.4
Cot $\theta$	6.2.3(2)	$1 \leq \text{Cot}\theta \leq 2.0$
Column $A_{s,\min}$	9.5.2(2)	$0.15 N_{Ed}  / f_{yd} \geq 0.003A_c$

**Table C-12 Ireland NDPs**

NDP	Clause	Value
$\alpha_{cc}$	3.1.6(1)	0.85
$k_1$	5.5(4)	0.4
$k_2$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$k_3$	5.5(4)	0.4
$k_4$	5.5(4)	$0.6 + 0.0014/\epsilon_{cu2}$
$v_{l,\min}$	11.6.1(1)	$0.028k^{3/2}f_{ck}^{1/2}$

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