New Zealand Society For Earthquake Engineering

Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

Section 10 Revision
Seismic Assessment of Unreinforced Masonry Buildings

Recommendations of a NZSEE Project Technical Group
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This document represents the current status of a review of the original Section 10 of the NZSEE Guidelines which was published in 2006.

Any comments will be gratefully received.

Please forward any comments to NZSEE Executive Officer at exec@nzsee.org.nz.

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Section 10 - Seismic Assessment of Unreinforced Masonry Buildings

10.1 General

10.1.1 Background

This section replaces the unreinforced masonry (URM) section in “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes”, published in 2006. It draws on key observations from the 2010/11 Canterbury earthquake sequence and on the significant quantity of research conducted in recent years at the University of Auckland, University of Canterbury and further afield. New sections include revised information on materials characterisation, a new method for diaphragm assessment, a new approach to the treatment of in-plane pier capacity based on failure modes, and the introduction of spandrel models.

URM construction can be vulnerable to earthquake shaking because of its high mass, lack of integrity between components and lack of deformation capability. The most hazardous features of URM buildings are inadequately restrained elements at height (such as façades, chimneys, parapets and gable-end walls), face-loaded walls, and their connections to diaphragms and return walls. These can present a significant risk to occupants as well as people within a relatively wide zone from the building.

Assessing the performance of these buildings can be complex as potential failure mechanisms are different from those occurring in other building types. Performance tends to be limited to out-of-plane wall behaviour, relative movement of different elements attached to flexible diaphragms, and tying of parts. This conflicts with the more typical idealisation of a building acting as one unified mass, but is essential to understand in order to assess these structures reliably.

The seismic capacity of URM bearing wall buildings is also difficult to quantify and may result in margins against collapse that are small for the following reasons:

- URM walls and piers may have limited non-linear deformation capability depending on their configuration, material characteristics, vertical stresses and potential failure modes.
- They rely on friction and overburden from supported loads and wall weights.
- They often have highly variable material properties.
- Their strength and stiffness degrade with each additional cycle of greater displacement of inelastic response to shaking. Therefore, they are vulnerable to incremental damage, especially in larger-magnitude, longer-duration earthquakes with multiple aftershocks.

Unlike other construction materials covered by these guidelines URM has not been permitted to contribute to the building lateral load resisting system in new buildings since 1964. Therefore, there is no standard for new URM buildings which could be used to compare to the standard achieved for an existing building. New building standard (*NBS*) and *%NBS* as it relates to URM buildings is therefore assumed to be defined by the requirements set out in this section.

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If buildings have undergone damage in an earthquake, much of the cyclic capacity may have already been used by the main event. Assessment of these buildings after an earthquake should consider this damaged state. As a result, their seismic capacity could be significantly lower than in their undamaged or repaired state. This is the important rationale for interim shoring for URM buildings (Figure 10.1) to mitigate further damage as an important part of building conservation. These techniques typically provide tying (rather than strengthening) to prevent further dilation of rocking or sliding planes, and to relieve stresses at areas of high concentration.

Note:

We recommend that you consider selective strengthening of URM buildings as a first step before proceeding to a detailed assessment, particularly in high seismicity areas. Improvement of diaphragm to wall connections, for example, will almost certainly be required to provide the building with any meaningful capacity as the as-built details will provide almost no support.

Using sound engineering judgement when assessing URM buildings is also important or you may end up with an economically non-viable solution, with the result that demolition may appear to be the only option.

Figure 10.1: Temporary securing of a mildly damaged solid masonry URM building (Dunning Thornton/Heartwood Community)

10.1.2 Scope

This section sets out guidelines for assessing:

- **unreinforced solid clay brick masonry buildings;** constructed of rectangular units in mortar, laid in single or multi-wythe walls, and in forms of bond such as common bond, English bond, running bond and Flemish bond.

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These guidelines are valid for:

- **walls in good condition**; with negligible mortar joint cracking, brick splitting, settlement or similar factors.
- **walls under face load attached to rigid or flexible diaphragms**
- **brick veneers under face loading**
- **stone masonry where the stones are layered.**

They can also be applied, with some additional requirements, to:

- unreinforced stone masonry that is well coursed and laid in running bond
- hollow clay brick and concrete block masonry (refer to Section 9 for assessment of brick or block infill masonry walls in framed construction)
- rubble stone masonry: the failure modes of these structures may be other than those covered here, including the possibility of delamination
- cobble stone masonry: assessment of face-loaded capacity is not covered by these guidelines.

Not in scope

This section does not cover:

- earthquake-damaged masonry buildings
- reinforced partially filled and fully filled block masonry

Note:

Although the strengthening of URM buildings is outside the scope of this section, brief comments on this topic have been included in Section 10.12.

10.1.3 Basis of this section

This section is largely based on experimental and analytic studies undertaken at the University of Auckland, University of Canterbury and in Australia, and on the research undertaken by Magenes et al. (1997) and Blaikie (1999, 2002). It also draws on ASCE, 2013.

Most of the default stress values have been adopted from tests undertaken at the University of Auckland (Lumantarna et al., 2014a; Lumantarna et al., 2014b) and from other sources including FEMA, 1998; ASCE, 2013; Kitching, 1999; and Foss, 2001.

Procedures for assessing face-loaded walls spanning vertically in one direction are based on displacement response that includes strongly non-linear effects. These procedures have been verified by research (Blaikie, 2001, 2002) using numerical integration time history analyses and by laboratory testing that included testing on shake tables. This research extended the preliminary conclusions reached in Blaikie and Spurr (1993). Other research has been conducted elsewhere, some of which is listed in studies including Yokel, 1971; Fattal, 1976; Hendry, 1973, 1981; Haseltine, 1977; West, 1977; Sinha, 1978; ABK Consultants, 1981; Kariotis, 1986; Drysdale, 1988; Lam, 1995; and Mendola, 1995. More recent research has been conducted by Derakhshan et al., 2014.

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Other useful information on materials, inspection and assessments is contained in FEMA (1998) and ASCE, 2013.

10.1.4 How to use this section

This section is set out as follows.

Understanding URM buildings (Sections 10.2 to 10.4)

These sections provide important context on the characteristics of URM buildings, typical building practices in New Zealand, and observed behaviour in earthquakes. As URM is a non-engineered construction, and given the recent learnings about its seismic performance, it is suggested you review this information carefully before proceeding to the assessment.

Assessing URM buildings (Sections 10.5 to 10.11)

These sections explain how to approach your assessment depending on what you are being asked and the type of building you are assessing. Given the nature of URM construction and the number of previous strengthening techniques used on these buildings, on-site investigation is particularly important. We provide a checklist of what to look for on-site as well as probable material properties, before setting out the detailed assessment methods.

Improving URM buildings (Section 10.12)

Although formally outside the scope of the section, we have included some brief comments on improving seismic performance of existing URM buildings. This is an introduction only to a broad field of techniques which is under continual development and research.

10.1.5 Notation

Symbol	Meaning	Comments
A	Angular deflection (rotation) of the top and bottom parts of a wall panel relative to a line through the top and bottom restraints, radian	The angle is in radians. It is measured as if there were no inter-storey deflection. Eqs 10B.5, 10B.13, 10B.14
A_{gross}	Gross plan area of diaphragm	Eq 10.6
$\ddot{A}_{max}$	Max acceleration	Eqs 10B.19, 10B.26
A_n	Area of net mortared/grouted section of the wall web, mm^2	Eq 10.30
A_n	Net plan are of wall, mm^2	Eq 10.51
A_n	Net plan area of masonry, mm^2	Eq 10.9
A_{net}	Net plan area of diaphragm excluding any penetration, m^2	Eq 10.6
a	Parameter given by equation	Eqs 10.13, 10.14, Table 10.12, Eqs 10.28, 10.31, 10.32, 10B.2, 10B.23, 10B.27
B	Depth of diaphragm, m	Eqs 10.8, 10.54, 10.55
b	Parameter given by equation	Section 10.8.5.1, Eqs 10.12, 10.21, Table 10.12, Eqs 10B.3, 10B.28
BCA	Building Consenting Authority	

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Symbol	Meaning	Comments
b_{sp}	Width of spandrel	Eqs 10.35, 10.37, 10.39, 10.40, 10.41, 10.42, 10.43, 10.46, 10.47, 10.48, 10.49
c	Masonry bed-joint cohesion, N/mm^2	The ability of the mortar to work in conjunction with the bricks. This is related to moisture absorption in the bricks. It depends less on the absorption qualities of individual brick types and is not greatly influenced by keying of the brick surface (e.g. holes, lattices or patterning). Cohesion is relevant to the primary decision of whether to use cracked or un-cracked masonry properties for the analyses. Eqs 10.3, 10.33, 10.39, 10.47 10.36
c	Probable cohesion, MPa	Table 10.4
$C(0)$		Section 10.10.5.1, Figure 10.78
$C(T_1)$	Elastic site hazard spectrum for horizontal loading	Eq 10.53
$C(T_d)$	Seismic coefficient at required height at period T_d	Eq 10.54
$C_h(0)$	Spectral shape factor for relevant soil determined from Clause 3.1.1, NZS 1170.5, g	Appendix 10C
$C_h(T_1)$	Spectral shape factor for relevant site subsoil type and period T_1 as determined from Section 3, NZS 1170.5, g	Eq 10.53
$C_{hc}(T_p)$	Spectral shape factor for site subsoil type *C* and period T_p as determined from Section 3, NZS 1170.5, g	Eq 10.16
C_{Hi}	Floor height coefficient for level i as defined in NZS 1170.5	Appendix 10C
$C_i(T_p)$		Eq 10.16, Section 10.10.3
C_m	Value of the seismic coefficient that would cause a mechanism to just form, g	Uniform acceleration to the entire panel is assumed in finding C_m Eq 10.21, Section 10.8.5.2 Step 15 Note, Table 10.12, 10.27, 10B.20
$C_p(0.75)$	Seismic coefficient for parts at 0.75 sec. Value of the seismic coefficient that would cause a mechanism to just form, g	Section 10.8.5.2 Step 13 & Step 15 Note
$C_p(T_p)$	Design response coefficient for parts as defined by Section 8, NZS 1170.5, g	Section 18.8.5.2 Step 8, Eqs 10.18, 10.19, Section 10.8.5.2 Step 13
CSW	Critical structural weakness	
D		Table 10.14
D_{ph}	Displacement response (demand) for a wall panel subject to an earthquake shaking as specified by Equation 10.18, mm	Eqs 10.18, 10.20
e		Table 10.14

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Symbol	Meaning	Comments
E_m	Young's modulus of masonry, MPa, kN/m^2	Eqs 10.4, 10.8, 10.51, 10.52
e_b	Eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b, mm	Eq 10.12, Table 10.12, 10.15, Section 10.8.5.2 Step 4, Figure 10B.1
e_o	Eccentricity of the mid-height pivot measured from the centroid of W_b, mm	Eq 10.12, 10.15, Section 10.8.5.2 Step 4, Figure 10B.1
e_p	Eccentricity of P measured from the centroid of W_t, mm	Eq 10.12, 10.15, Table 10.12, Section 10.8.5.2 Step 4, Figure 10B.1
e_t	Eccentricity of the mid-height pivot measured from the centroid of W_t, mm	Eqs 10.12, 10.15, Section 10.8.5.2 Step 4, Figure 10B.1
F	Applied load on timber lintel	Eq 10.42
F_i	Equivalent static horizontal force at the level of the diaphragm (level i)	Section 10.10.5.1, Figure 10.78
f'_b	Compressive strength of bricks, N/mm^2	Measured on the flat side Section 10A.3.2
f'_b	Probable brick compressive strength, MPa	Table 10.3, 10.5, Eqs 10.1, 10.2
f'_j	Normalised mortar compressive strength, N/mm^2	Eq 10A.1
f'_j	Probable mortar compressive strength, MPa	Table 10.4, Eq 10.2, Table 10.5
f'_{ji}	Measured irregular mortar compressive strength, MPa	Eq 10A.1
f'_m	Compressive strength of masonry, MPa	Eq 10.31
f'_m	Probable masonry compressive strength, MPa	Eq 10.2, Table 10.5, Eq 10.4
f'_r	Modulus of rupture of bricks, MPa	Eq 10.1, Section 10.8.5.1
f'_t	Equivalent tensile strength of masonry spandrel, MPa	Eqs 10.9, 10.35, 10.36
f_a	Axial compression stress on masonry due to gravity load, MPa	Eqs 10.3, 10.30, 10.31
f_{bt}	Probable brick tensile strength, MPa	May be taken as 85% of the stress derived from splitting tests or as 50% of stress derived from bending tests Table 10.3
f_{dt}	Diagonal tensile strength of masonry, MPa	Eqs 10.3, 10.30, 10.38, 10.40, 10.48
f_{hm}	Compression strength of the masonry in the horizontal direction ($0.5f'_m$), MPa	Eqs 10.37, 10.46
g	Acceleration due to gravity, m/sec^2	Eqs 10.15, 10.17, 10.18, 10.29
G'_d	Reduced diaphragm shear stiffness, kN/m	Eqs 10.6, 10.7, 10.8
$G'_{d,eff}$	Effective diaphragm shear stiffness, kN/m	Eqs 10.7, 10.54
G_d	Shear stiffness of straight sheathed diaphragm, kN/m	Table 10.8, Eq 10.6
G_m	Shear modulus of masonry, MPa	Eqs 10.5, 10.51, 10.52

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Symbol	Meaning	Comments
h	Free height of a cantilever wall from its point of restraint or height of wall in between restraints in case of simply-supported face-loaded wall	The clear height can be taken at the centre-to-centre height between lines of horizontal restraint. In the case of concrete floors, the clear distance between floors will apply. Eqs 10.13, 10.17
h_i	Average of the heights of point of support	Section 10.8.5.2 Step 8
h_i	Height of attachment of the part	Figure 10.78
H_l	Height of wall below diaphragm, m	Eq 10.8
h_{eff}	Height of wall or pier between resultant forces	Table 10.13, Eqs 10.31, 10.32, Figures 10.65, 10.74, Table 10.14, Eq 10.51
h_i	Average of the heights of points of support	Section 10.8.5.2 Step 8
h_n		Figure 10.78
h_{sp}	Height of spandrel excluding depth of timber lintel if present	Eqs 10.35, 10.37, 10.38, 10.39, 10.40, 10.41, 10.42, 10.43, 10.46, 10.47, 10.48, 10.49
h_{tot}		Eq 10.46, Figure 10.69
H_u	Heigh of wlall above diaphragm, m	Eq 10.8
I_g	Moment of inertia for the gross section representing uncracked behaviour	Eq 10.51
I_{xx}	Mass moment of inertia about x-x axis, kgm^2	Eq 10B.9
I_{yy}	Mass moment of inertia about y-y axis, kgm^2	Eq 10B.10
J	Rotational inertia of the wall panel and attached masses, kgm^2	Eqs 10.14, 10.15, 10.17, Table 10.12, Eqs 10.29, 10B.8, 10B.30
J_{anc}	Rotational inertia of ancillary masses, kgm^2	Eqs 10.15, 10B.8
J_{bo}	Rotational inertia of the bottom part of the panel about its centroid, kgm^2	Eqs 10.15, 10B.11
J_{bo}	Polar moment of inertia about centroid, kgm^2	Eqs 10B.8, 10B.11
J_{to}	Rotational inertia of the top part of the panel about its centroid	Eqs 10.15, 10B.8
k	In-plane stiffness of walls and piers, N/mm	Eqs 10.51, 10.52
K_A		Section 10.9.2
K_R	Seismic force reduction factor for in-plane seismic force	Coefficient proposed in lieu of S_p *and* K_μ Eq 10.53, Table 10.15
L	Span of diaphragm, m	Eq 10.8, 10.54, 10.55
l	Length of header	Section 10.8.5.1
l_{sp}	Clear length of spandrel between adjacent wall piers	Eqs 10.35, 10.37, 10.38, 10.40, 10.41, 10.42, 10.43, 10.44, 10.46, 10.48

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Symbol	Meaning	Comments
L_w	Length of wall	Eqs 10.31, 10.33
M	Moment capacity of the panel	Eq 10.9
$M.F$		Eq 10A.4
M_1, M_i, M_n	Moment imposed on wall/pier elements	Figure 10.72
m	Mass, kg	Eq 10B.11
m_i	Seismic mass at the level of the diaphragm (level i)	Section 10.10.5.1, Figure 10.78
$N(T_1,D)$	Near fault factor determined from Clause 3.1.6, NZS 1170.5	Eq 10.53
n	Number of recesses	Eq 10.10
N_1, N_i, N_n	Axial loads on pier elements	Figure 10.72
P	Superimposed and dead load at top of wall/pier	Eqs 10.31, 10.32, 10.33, 10.34
P	Load applied to the top of panel	P is assumed to act through the pivot at the top of the wall Section 10.8.5.1, 10.8.5.2 Step 2, 3 & 4, Eqs 10.9, 10.12, 10.13, 10.15, 10.28
p	Depth of mortar recess, mm	Eq 10.10, Figure 10.62
P-Δ	P- delta	Section 10.8
p_p	Mean axial stress due to superimposed and dead load in the adjacent wall piers	Eq 10.36
p_{sp}	Axial stress in the spandrel	Eqs 10.35, 10.37, 10.38, 10.39, 10.40, 10.41, 10.42, 10.43, 10.46, 10.47, 10.48, 10.49
P_w	Self-weight of wall and pier	Eqs 10.31, 10.32, 10.33, Figure 10.65, EQ 10.34
Q	Live load	Section 10.10.5.2
Q_1, Q_i, Q_n	Shear in pier element	Figure 10.72
R	Return period factor, R_u determined from Clause 3.1.5, NZS 1170.5	Eq 10.16
r_a		Eq 10.44, Figure 10.69
r_i		Eqs 10.44, 10.45, Figure 10.69
r_o		Eq 10.45, Figure 10.69
R_P	Risk factor for parts as defined in NZS 1170.5	Eq 10.18, Section 10.8.5.2 Step 13
R_u	Return period factor for ultimate limit state as defined in NZS 1170.5	Eq 10.53
S_i	Sway potential index	Eq 10.50
S_p	Structural performance factor in accordance with NZS 1170.5	Section 10.8.5.2, 10.10.2.1, 10.10.5.1, 10.10.8
SW	Structural weakness	

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Symbol	Meaning	Comments
t	Depth of header	Section 10.8.5.1
t	Effective thickness, mm	Varies with position Section 10.8.5.2 Step 2, 3 & 4, Eq 10B.22
T_1	Fundamental period of the building, sec.	Eq 10.53
T_d	Fundamental period of diaphragm, sec.	Eqs 10.54, 10.55
t_{gross}	Overall thickness of wall, mm	Varies with position Eq 10.10
t_l	Effective thickness of walls below the diaphragm, m	Eq 10.8, Figure 10.59
t_{nom}	Nominal thickness of wall excluding pointing, mm	Varies with position Eqs 10.9, 10.10, Section 10.8.5.2 Step 2 & 3, Eqs 10.33, 10B.22
T_p	Effective period of parts, sec.	Eqs 10.14, 10.16, 10.18, 10.19, 10.23, 10.24, 10B.15, 10B.16, 10B.17, 10B.18, 10B.24, 10B.25, 10B.31
t_u	Effective thickness of walls above the diaphragm, m	Eq 10.8, Figure 10.59
V	Probable shear strength capacity	
V_{dpc}	Capacity of a slip plane for no slip	Eq 10.34
V_{dt}	In-plane diagonal tensile strength capacity of pier and wall	Eq 10.30
V_{fl}	Peak flexural capacity of spandrel	Figure 10.68, Eqs 10.35, 10.43
$V_{fl,r}$	Residual flexural strength capacity	Figure 10.68, Eqs 10.37, 10.46
$(V_{prob})_{global,\ base}$		Figure 10.75
$(V_{prob})_{line,\ i}$		Section 10.9.2
$(V_{prob})_{wall1,wall2}$		Figure 10.77
V_r	In-plane rocking strength capacity of pier and wall	Eq 10.32, Figure 10.66
V_s	In-plane bed-joint shear strength capacity of pier and wall	Eq 10.33, Figures 10.66, 10.68
V_{s1}		Eqs 10.39, 10.47
V_{s2}		Eqs 10.40, 10.48
$V_{s,r}$	Residual spandrel shear strength capacity or residual wall sliding shear strength capacity	Eq 10.33, Figure 10.68, Eqs 10.41, 10.49
V_{tc}	In-plane toe-crushing strength capacity of pier and wall	Eq 10.31
$V_{tc,r}$		Figure 10.66
W	Weight of the wall and pier	Section 10.8.5.2 Step 3, Eqs 10.28, 10B.11
W_b	Weight of the bottom part of the panel	Section 10.8.5.1, 10.8.5.2 Step 2 & 14, Eqs 10.12, 10.13, 10.15, 10.17, 10.21

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Symbol	Meaning	Comments
W_t	Weight of the top part of the panel	Section 10.8.5.1, 10.8.5.2 Step 2 & 14, Eqs 10.12, 10.13, 10.15, 10.17, 10.21
W_{trib}	Uniformly distributed tributary weight	Eqs 10.54, 10.55
y_b	Height of the centroid of W_b from the pivot at the bottom of the panel	Eqs 10.12, 10.13, 10.15, 10.17, 10.21, Sections 10B.2.6, 10B.2.7, 10B.2.8, 10B.3.2, 10B.3.3
y_t	Height from the centroid of W_t to the pivot at the top of the panel	Eqs 10.12, 10.13, 10.15, 10.17, 10.21
Z	Hazard factor as defined in NZS 1170.5	Eq 10.53
α_a	Arch half angle of embrace	Eqs 10.43, 10.44, 10.45, 10.47, 10.48, 10.49
α_{ht}	*t/l* ratio correction factor	Eqs 10A.1, 10A.3
α_{tl}	*t/l* ratio correction factor	Eqs 10A.1, 10A.2
α_w	Diaphragm stiffness modification factor taking into account boundary walls	Eqs 10.7, 10.8
β	Factor to correct nonlinear stress distribution	Eq 10.30, Table 10.13
β_1		Section 10.9.2
β_s	Spandrel aspect ratio	Eq 10.38
γ	Participation factor for rocking system	This factor relates the deflection at the mid-height hinge to that obtained from the spectrum for a simple oscillator of the same effective period and damping Eqs 10.17, 10.18, 10.25, 10B.21, 10B.32
Δ	Horizontal displacement, mm	Eq 10B.16
Δ_d	Horizontal displacement of diaphragm	Eq 10.54
Δ_i	Deflection that would cause instability of a face-loaded wall	W_b, W_t and P are the only forces applying for this calculation Eqs 10.11, Table 10.12, Eqs 10B.6, 10B.16, 10B.30, Section 18.8.5.2 Step 6, Eq 10.20
Δ_m	An assumed maximum useful deflection = $0.6\ \Delta_i$ and $0.3\Delta_i$ for simply-supported and cantilever walls respectively	Used for calculating deflection response capacity Section 18.8.5.2 Step 6, Eqs 10.20, 10B.6
Δ_t	An assumed maximum useful deflection = $0.6\Delta_m$ and $0.8\Delta_m$ for simply-supported and cantilever walls respectively	Used for calculating fundamental period of face-loaded rocking wall Eq 10B.16
$\Delta_{tc.r}$	Deformation at the onset of toe crushing	Section 10.8.6.2, Figure 10.66
Δ_y	Yield displacement	Section 10.8.6.2, Figure 10.66
θ_y	Yield rotation fo the spandrel	Figure 10.68, Table 10.14
μ	Structural ductility factor in accordance with NZS 1170.5	Sections 10.8.5.2, 10.10.2.1, 10.10.5.1, 10.10.8
μ_{dpc}	DPC coefficient of friction	Eq 10.34

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Symbol	Meaning	Comments
μ_f	Masonry coefficient of friction	Eqs 10.3, 10.33, 10.36, 10.39, 10.47, Section 10A.2.4
μ_f	Probable coefficient of friction	Table 10.4
μ_p	Ductility of part (wall)	Section 10.8.5.2 Step 13
ρ	Density (mass per unit volume)	Eqs 10B.9, 10B.10
ξ_ε	Equivalent viscous damping	Section 10.10.2.1
$\Sigma V^*_{u,Pier}$	Sum of the 100%*NBS* shear force demands on the piers above and below the joint calculated using K_R = 1.0	
$\Sigma V_{n,Pier}$	Sum of the piers' capacities above and below the joint	
$\Sigma V^*_{u,Spandrel}$	Sum of the 100%*NBS* shear force demands on the spandrels to the left and right of the joint calculated using K_R = 1.0	
$\Sigma V_{n,Pier}$	Sum of the spandrel capacities to the left and right of the joint	
ϕ	Strength reduction factor	Section 10.8.2
ϕ	Capacity reduction factor…	Table 10.6, Table 10.7
ψ	Inter-storey slope, radian	Inter-storey deflection divided by the storey height Eq 10.12

10.1.6 Definitions

Action	Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action). The term 'load' is also often used to describe direct actions.
Adhesion	Bond between masonry unit and mortar.
Beam	An element subjected primarily to loads producing flexure and shear.
Bearing wall	A wall that carries (vertical) gravity loads due to floor and roof weight.
Bed joint	The horizontal layer of mortar on which a brick or stone is laid.
Bond	A bond is the pattern in which masonry units are laid.
Brittle	A brittle material or structure is one that fails or breaks suddenly when subjected to bending, swaying or deformation. A brittle structure has very little tendency to deform before it fails and it very quickly loses lateral load carrying capacity once failure is initiated.
Cavity wall	A cavity wall consists of two 'skins' separated by a hollow space (cavity). The skins are commonly both masonry, such as brick or concrete block, or one could be concrete. The cavity is constructed to provide ventilation and moisture control in the wall.
Cohesion	Bond between mortar and brick.
Collar joint	A collar joint is a vertical longitudinal space between wythes of masonry or between an outer masonry wythe and another backup system. This space is often specified to be filled solid with mortar or grout, but sometimes collar-joint treatment is left unspecified.

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Course	A course refers to a row of units stacked on top of one another.
Cross wall	An interior wall that extends from the floor to the underside of the floor above or to the ceiling, securely fastened to each and capable of resisting lateral forces.
Dead load	The weight of the building materials that make up a building, including its structure, enclosure and architectural finishes. The dead load is supported by the structure (walls, floors and roof).
Diaphragm	A horizontal structural element (usually suspended floor or ceiling or a braced roof structure) that is strongly connected to the walls around it and distributes earthquake lateral forces to vertical elements, such as walls, of the lateral force resisting system. Diaphragms can be classified as flexible or rigid.
Dimension	When used alone to describe masonry units, means nominal dimension.
Ductility	The ability of a structure to sustain its load-carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.
Earthquake-Prone Building (EQP)	A legally defined category which describes a building that has been assessed as likely to have its ultimate limit state capacity exceeded in moderate earthquake shaking (which is defined in the regulations as being one third of the size of the shaking that a new building would be designed for on that site). A building having seismic capacity less than 34*%NBS*.
Earthquake Risk Building (ERB)	A building having seismic capacity less than 67*%NBS*.
Face-loaded walls	Walls subjected to out-of-plane shaking. Also see Out-of-plane load.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, steel, or precast concrete without reinforced concrete topping fall in this category.
Gravity load	The load applied in a vertical direction, including the weight of building materials (dead load), environmental loads such as snow, and moveable building contents (live load).
Gross area	The total cross-sectional area of a section through an element bounded by its external perimeter faces without reduction for the area of cells and re-entrant spaces.
In-plane load	Seismic load acting along the wall length.
In-plane walls	Walls loaded along its length. Also referred as in-plane loaded wall.
Irregular building	A building that has a sudden change in the shape of plan is considered to have a horizontal irregularity. A building that changes shape up its height (such as setbacks or overhangs) or is missing significant load-bearing walls is considered to have a vertical irregularity. In general, irregular buildings do not perform as well as regular buildings perform in earthquakes.
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects.
Leaf	See Wythe.
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil.
Load	See Action.
Low-strength masonry	Masonry laid in weak mortar; such as weak cement/sand or lime/sand mortar.
Masonry unit	A preformed component intended for use in masonry construction.
Masonry	Any construction in units of clay, stone or concrete laid to a bond and joined together with mortar.

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Mortar	The cement/lime/sand mix in which masonry units are bedded.
Mullion	A vertical member, as of stone or wood, between the lights of a window, the panels in wainscoting, or the like.
Net area	The gross cross-sectional area of the wall less the area of un-grouted areas or penetrations.
Out-of-plane load	Seismic load (earthquake shaking) acting normally (perpendicular) or at right angles to the wall surface. Walls subjected to out-of-plane shaking are also known as face-loaded walls. Walls are weaker and less stable under out-of-plane than under in-plane seismic loads.
Partition	A non-load-bearing wall which is separated so as not to be part of the seismic resisting structure.
Party wall	A party wall (occasionally party-wall or parting wall) is a dividing partition between two adjoining buildings or units that is shared by the tenants of each residence or business.
Pier	A portion of wall between doors, windows or similar structures.
Pointing (masonry)	Troweling mortar into a masonry joint after the masonry units have been laid. Higher quality mortar is used than for the brickwork.
Primary element	An element which is relied on as part of the seismic force resisting system.
Regular building	see Irregular building.
Return wall	A short wall usually perpendicular to, and at the end of, a freestanding wall to increase its structural stability.
Rigid diaphragm	A suspended floor, roof or ceiling structure that is able to transfer lateral loads to the walls with negligible horizontal deformation of the diaphragm. Floors or roofs made from reinforced concrete, such as reinforced concrete slabs, fall into this category.
Running or stretcher bond	The unit set out when the units of each course overlap the units in the preceding course by between 25% and 75% of the length of the units.
Seismic hazard	The potential for damage caused by earthquakes. The level of hazard depends on the magnitude of probable earthquakes, the type of fault, the distance from faults associated with those earthquakes, and the type of soil at the site.
Seismic system	That portion of the structure which is considered to provide the earthquake resistance to the entire structure.
Shear wall	A wall which is subjected to lateral loads due to wind or earthquakes acting parallel to the direction of an earthquake load being considered (also known as an in-plane wall). Walls are stronger and stiffer in plane than out of plane.
Special study	A procedure for justifying a departure from these guidelines or determining information not covered by them. Special studies are outside the scope of these guidelines.
Stack bond	The unit set out when the units of each course do not overlap the units of the preceding course by the amount specified for running or stretcher bond.
Strength, design	The nominal strength multiplied by the appropriate strength reduction factor.
Strength, probable	The theoretical strength of a component section, calculated using the section dimensions as detailed and the theoretical characteristic material strengths as defined in these guidelines.
Strength, required	The strength of a component section required to resist combinations of actions for ultimate limit states as specified in AS/NZS 1170 Part 0.
Structural element	Component of a building that provides gravity and lateral load resistance and is part of a continuous load path. Walls are key structural elements in all masonry buildings.

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Through stone	A long stone (header unit) that connects two wythes together in a stone masonry wall. It is also known as bond stone. Contrary to its name, a through stone can also be a concrete block, a wood element, or steel bars with hooked ends embedded in concrete that perform the same function.
Transom	See Mullion.
Transverse wall	See Cross wall.
Unreinforced masonry (URM) wall	A masonry wall containing no steel, timber, cane or other reinforcement. An unreinforced wall resists gravity and lateral loads solely through the strength of the masonry materials.
Veneer	See Wythe.
Wall tie	The tie in a cavity wall used to tie the internal and external walls (or wythes) constructed of wires, steel bars or straps.
Wall	A vertical element which because of its position and shape contributes to the rigidity and strength of a structure.
Wythe	A continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, the adjoining wythe(s). A single wythe is also referred to as a veneer or leaf.

10.2 Typical URM Building Practices in New Zealand

10.2.1 General

Most of New Zealand’s URM buildings were built during a relatively narrow window of time; between the late 1870s and 1940 (Russell & Ingham, 2010). As a result, construction methods are relatively uniform with only a few variations reflecting the origins of the stonemasons and the customary stones (“hard rock” or “soft rock”) they used for laying. However, these buildings vary substantially in their structural configuration and layout.

10.2.2 Building forms

The range of typical URM buildings is set out in Table 10.1 together with some common characteristics for each type. Note that:

- Most of the smaller buildings are cellular in nature, combining internal masonry or timber walls with the perimeter masonry façade to provide an overall rigid unit.
- Many smaller commercial URM buildings have fairly open street façades at ground level and high bottom storeys.
- Larger buildings tend to have punched wall frames (Figure 10.2) and open plan areas where floors and roofs are supported by timber, cast iron or steel posts.
- Large, complex buildings such as churches are particularly vulnerable to earthquake shaking as they tend to have irregular plans, tall storey heights, offset roofs, few partitions and many windows.

In these guidelines smaller buildings (ie less than two storeys), including small churches and halls are categorised as *basic buildings* to distinguish them from more complex buildings. Simplified approaches, particularly associated with determining material property and analysis, are possible when assessing basic buildings. These are covered in the appropriate sections below.

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The interaction of buildings constructed with common boundary or party walls is discussed in more detail in Section 10.5.4.

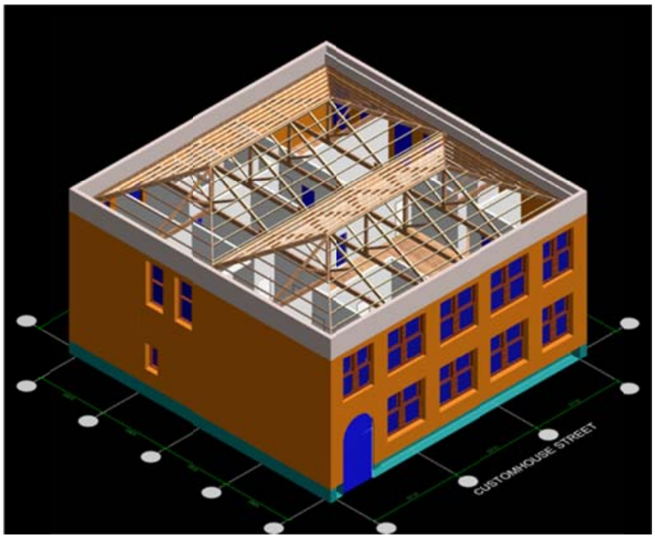
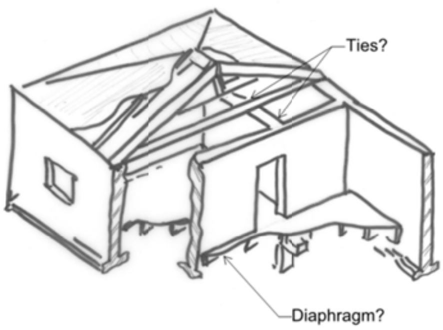
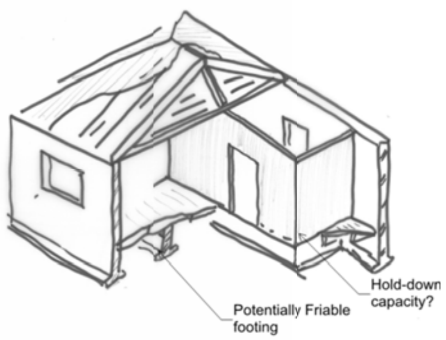


Figure 10.2: URM building with punched wall

Table 10.1: Building forms

Form	Illustration	Particular issues
1 Storey cellular: **Masonry Internal Walls** *Bracing predominantly from in-plane walls cantilevering from ground level*		• Bonding at wall intersections • Plan regularity – diaphragm demand if irregular • Relative stiffness/strength from varying wall lengths • Subfloor height and level of fixity • Ground floor diaphragm/bracing
1 Storey cellular: **Timber Internal Walls** *Bracing predominantly from walls loaded in-plane cantilevering from ground level*		• Connection to masonry at intersections • Stiffness compatibility with masonry – wall geometry • Stiffness compatibility with masonry – materiality (plaster/lath, fibrous plaster) • Flexibility of strapping/lining with respect to masonry • Timber wall foundation bracing capacity

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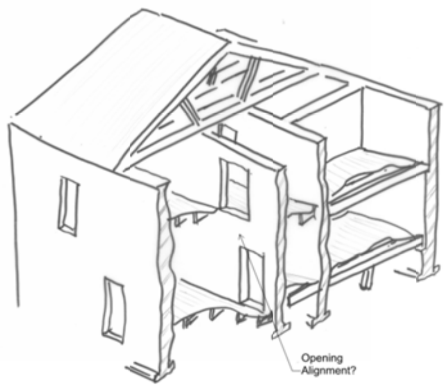
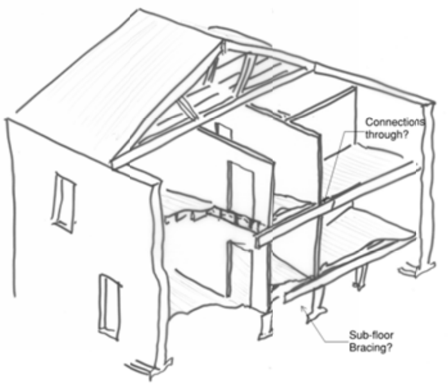
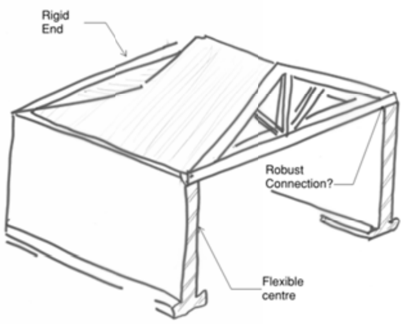
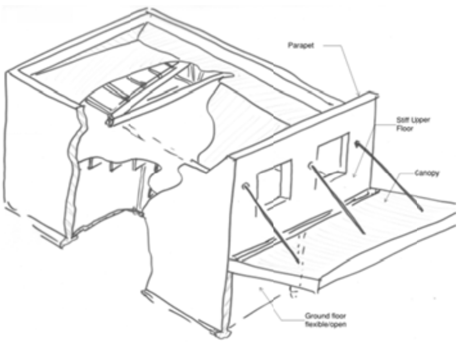
Form	Illustration	Particular issues
>1 Storey Cellular: **Masonry Internal Walls** *Bracing predominantly from walls loaded in-plane with interaction over doorways and between floors*	 	As 1 Storey plus: • Wall coupling over doorways • Change in wall thickness at first floor
>1 Storey Cellular: **Timber Internal Walls** *Bracing predominantly from walls loaded in-plane with interaction over doorways and between floors*	 	As 1 Storey plus: • Hold-down of upper walls to lower walls • Hold-down and bracing of lower walls to piles
1 Storey Open: *Bracing predominantly from walls loaded out-of-plane cantilevering from ground level*	 	• End walls and differential stiffness • Ground conditions and foundations critical • Wall connection with ground floor slab if present
>1 Storey Open: *Bracing predominantly from walls loaded out-of-plane cantilevering from ground level, with contributions from end walls* Most common town centre commercial structures		• Diaphragm stiffness • Diaphragm strength • Ancillary structures forming bracing • Contribution of shop front beams/frame • Plan regularity

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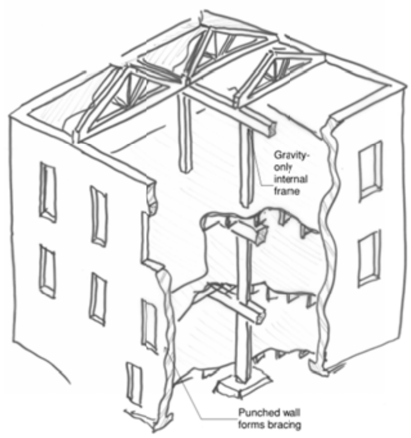
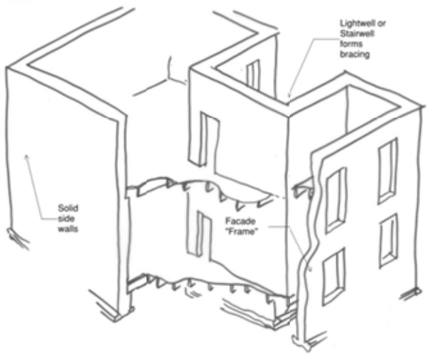
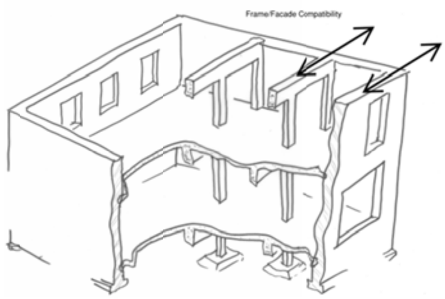
<table>
<tr><th>Form</th><th>Illustration</th><th>Particular issues</th></tr>
<tr><td>Multi-storey Open
Bracing predominantly from perimeter walls loaded in-plane</td><td>
</td><td>Wall-to-diaphragm connection demands high for out-of-plane wall loads
Diaphragm stiffness important for out-of-plane wall analysis
Diaphragm strength demands often high
Holes in diaphragms
Punched walls in-plane analysis can be complex</td></tr>
<tr><td>Multi-storey with Internal Structures
Bracing from combination of internal walls and perimeter walls loaded in-plane</td><td>
</td><td>Wall-to-diaphragm connection demands high
Compatibility between flexible internal and stiff external structures
Punched walls in-plane analysis can be complex</td></tr>
<tr><td>Multi-storey Frame/Wall
Bracing from combination of internal walls and perimeter walls loaded in-plane</td><td>
</td><td>Often heavyweight floors: stiff but strength difficult to ascertain
Internal frame stiffness vs. perimeter punched wall stiffness
High shear demands on in-plane connection to perimeter elements</td></tr>
<tr><td>Monumental – Single Form
Bracing predominantly from cantilever action, single degree of freedom

Statues, towers, chimneys and the like</td><td>
</td><td>Often rocking governed – can be beneficial
Foundation stability critical
Combination of materials forming masonry unit
Damping</td></tr>
</table>

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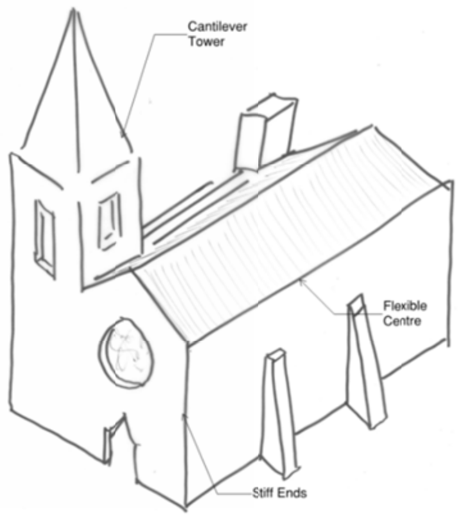
Form	Illustration	Particular issues
Monumental – Multiple Forms *Multiple degrees of freedom with different stiffnesses/periods* Most churches and larger civic structures	 	• Highly complex interaction between elements • Special study • Peer review recommended

10.2.3 Foundations

Foundations for URM buildings were typically shallow strip footings (Figure 10.3(a)), including under openings in punched walls or facades. Bricks were typically placed transverse to the wall to give a half-to-one brick-thickening, although larger multi-stepped thickenings were used in large structures. The bricks were typically protected from direct contact with the ground with a layer of concrete. In smaller buildings, this was often thin and unreinforced.

Deeper concrete strips (Figure 10.3(b)) for larger buildings were often nominally reinforced with plain reinforcing bars, flats, or train/tram rails. In extremely poor ground or where the foundation formed a sea wall or wharf, these reinforced concrete strips generally spanned between driven timber or sometimes between steel or precast piles. The design was often rudimentary, with the depth of the concrete at least half that of the span regardless of reinforcement.

As the widening of the foundation was often nominal, some settlement was common in poorer ground either during or after construction. Settlement during construction could often be "built in" so would not be visible.

Larger industrial buildings with timber, steel or cast iron posts were often founded on large, isolated pads. As these were sized for the "live" actions, they are often lightly loaded so are an excellent indicator of settlement.

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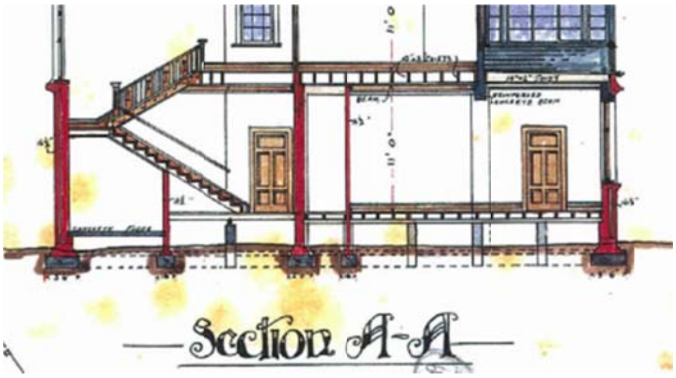
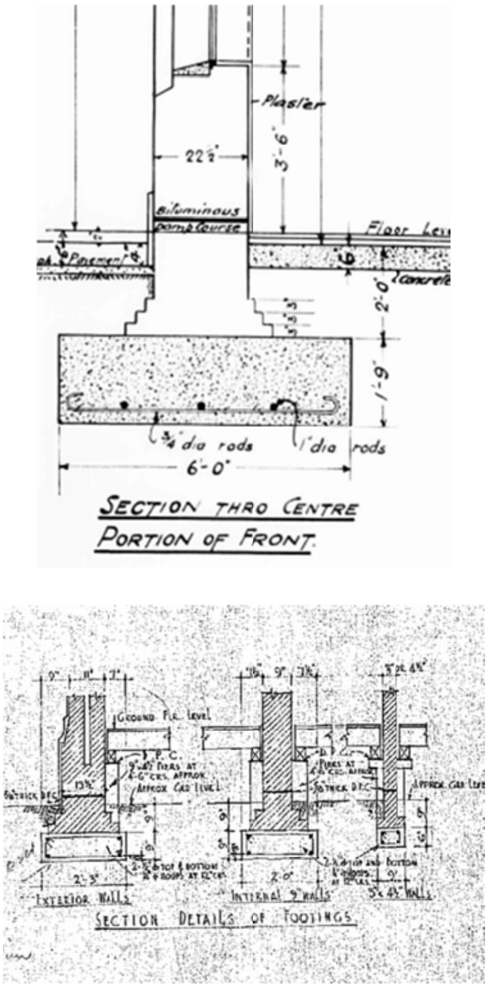


(a) Typical foundation details



(b) A cross section of URM building foundation

Figure 10.3: URM building foundations

10.2.4 Wall construction

Solid and cavity walls were common types of construction:

- Solid walls were generally used for industrial buildings and buildings on the outskirts of town, and for party walls and walls either not visible or in lower storeys.
- Cavity walls were used in buildings to control moisture ingress. They also allow the use of higher quality bricks where a better architectural finish was required on the exterior.

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In cavity walls, the exterior masonry wythes act as an architectural finish (which can give a misleading impression of these walls' structural thickness). It was also common to provide an outer wythe that was continuous over the full height of the wall plus an inner one-brick-thick wythe for the top storey and two or more wythes for lower storeys (Figure 10.4). Construction quality was usually better for visible walls and veneers than in hidden areas or at the rear of buildings.

Figure 10.4: Change in cross-section of brick wall (Holmes Consulting Group)

Often a cavity wall, which was originally on the exterior of the building, has become an interior wall following subsequent alteration. This will be recognisable by a wall thickness that is not a wythe multiple.

10.2.4.1 Wall thickness

The commonly used nominal thicknesses of brick walls in New Zealand are 230 mm (9", two wythes), 350 mm (14", three wythes) and 450 mm (18", four wythes). This is in addition to any outer veneer of 110 mm (4½", one wythe).

10.2.4.2 Cavity ties

In cavity walls, outer wythes were usually tied to the inner wythe or main structural wall with #8 ties, sometimes with a kink in the middle, or with flat pieces of tin generally at spacings of 900 mm horizontally and every fifth or sixth course vertically (Figure 10.5). Cast steel, wrought steel or mild steel toggles were sometimes used at similar spacings.

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(a) Common wire ties

(b) Double hook ties

(c) Butterfly ties

(d) V-drip flat fishtailed wall ties

Figure 10.5: Commonly observed wall ties (Dymtro Dizhur)

10.2.4.3 Masonry bond and cross sections

A number of different bond patterns have been used for URM buildings, as described below. The bond pattern is an important feature of URM buildings: it determines how the masonry units in a wall are connected and has a significant effect on both the wall strength and how its components act together as a complete structural element.

Stretcher units, or stretchers, are bricks laid in the plane of the wall. Header units, or headers are bricks laid across the wall joining the masonry wythes together.

In cross section, a wall three units thick is a three wythe wall. To act as one, each wythe should be adequately connected to the adjoining wythe with headers at appropriate intervals.

Note that sometimes fake headers are incorporated into a wythe that do not cover two adjoining wythes. These can disguise the presence of a cavity wall where there is a cavity void between the inner and outer wythes.

Clay brick masonry

Most New Zealand URM buildings were constructed with either common bond, which is the most frequently occurring bond pattern, or English bond, which is often found on the bottom (ground) storey.

Common bond is sometimes referred to as American bond or English garden wall bond. It has layers of stretchers, and headers every three to six courses (Figure 10.6(a)). These headers can be at different levels in different buildings, and sometimes even within the same building. For example, the headers may be every second course at the bottom of the ground storey but every fourth course near the top of the third storey. Header courses may

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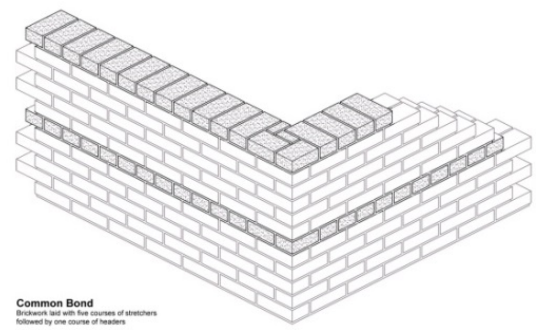
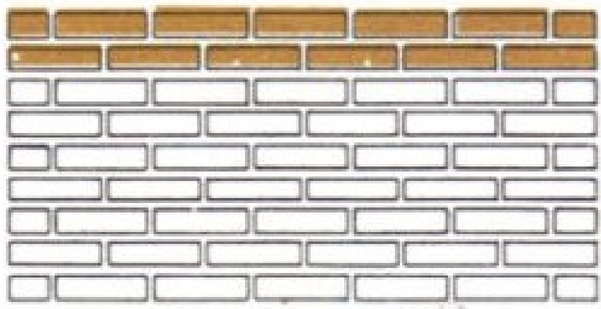
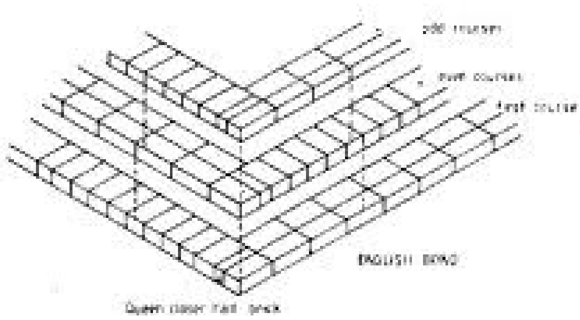
be irregular and made to fit in at ends of walls and around drainpipes with half widths and other cut bricks.

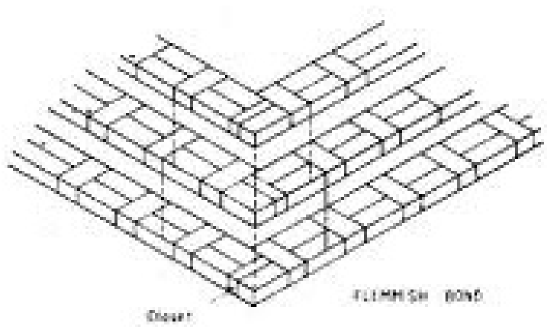
English bond has alternating header and stretcher courses (Figure 10.6(c)).





(a) Common bond **(b) Running bond**





(c) English bond **(d) Flemish bond**

Figure 10.6: Different types of brick masonry bonds

Other bond patterns used in New Zealand include Running bond (Figure 10.6(b)) and Flemish bond (Figure 10.6(d)). Running bond (stretcher courses only) often indicates the presence of a cavity wall. Flemish bond (alternating headers and stretchers in every course) is the least common bond pattern and is generally found between openings on an upper storey; for example, on piers between windows.

Stone masonry

Stone masonry buildings in New Zealand are mainly built with igneous rocks such as basalt and scoria, or sedimentary rocks such as limestone. Greywacke, which is closely related to schist, is also used in some parts of the country. Trachyte, dolerite, and combinations of these are also used.

Wall texture

Wall texture describes the disposition of the stone courses and vertical joints. There are three different categories (Figure 10.7): ashlar (squared stone); rubble (broken stone); and cobble stones (field stone), which is less common.

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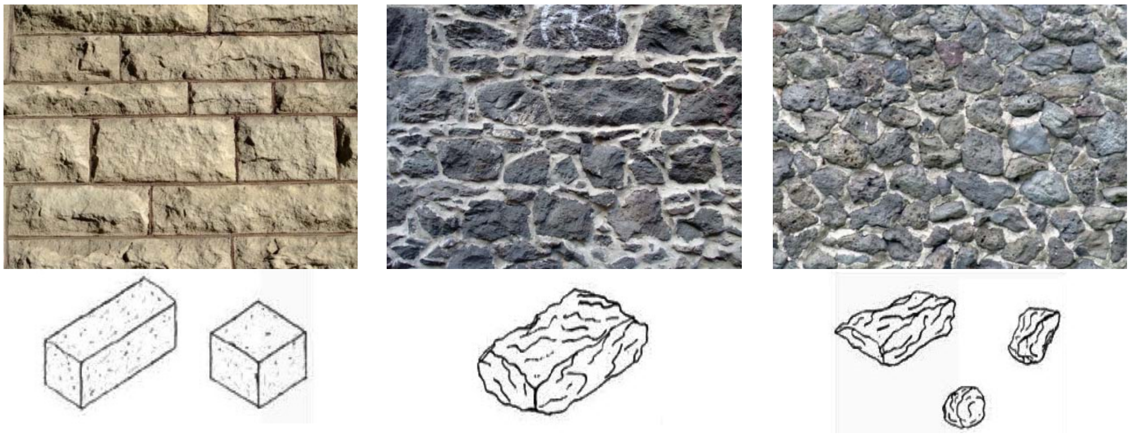
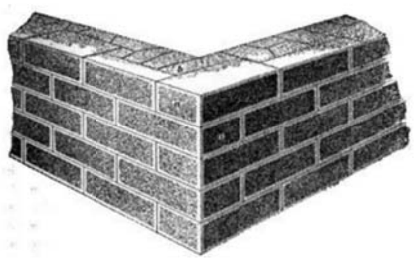


(a) Ashlar (squared stone) **(b) Rubble (broken stone)** **(c) Cobble stones (field stone)**

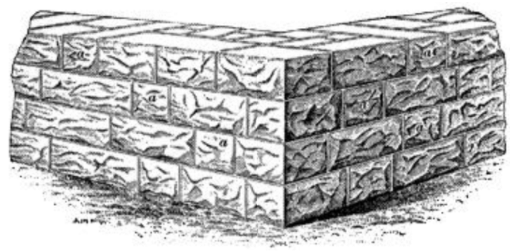
Figure 10.7: Classification of stone units (Marta Giaretton)

Ashlar (dressed or undressed) is stonework cut on four sides so that the adjoining sides will be at right angles to each other (Figure 10.7(a)). Ashlar is usually laid as either coursed ashlar, which is in regular courses with continuous joints (Figure 10.8(a)), or block-in-course ashlar (Figure 10.8(b)). It may also appear as broken courses (which describes the broken continuity of the bed and head joints) of either random-course ashlar (Figure 10.8(c)), or broken ashlar (Figure 10.8(d)).

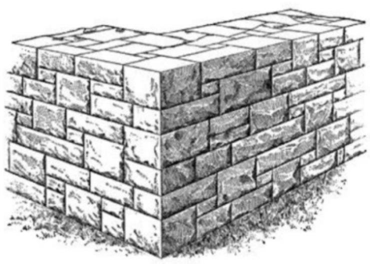
All ashlar should have straight and horizontal bed joints, and the vertical joints should be kept plumb. This type of stone can also be found in coursed rubble; in which case it may be considered as a hybrid between rubble and ashlar stonework.



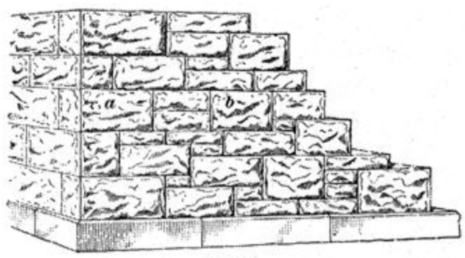
(a) Coursed ashlar



(b) Block-in-course ashlar



(c) Random-course ashlar



(d) Broken ashlar

Figure 10.8: Schematic of different forms of Ashlar bond (Lowndes 1994)

Rubble stonework consists of stones in which the adjoining sides are not required to be at right angles (Figure 10.7(b)). This form of masonry is often used for rough masonry such as foundations and backing, and frequently consists of common, roughly dressed field stone.

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Wall cross section

It is usually not possible to establish the cross section characteristics of a stone masonry wall from the bond pattern. More detailed inspection is required to identify any connections between the wythes; determine what material the core is composed of; and locate any voids, a cavity, or the presence of other elements such as steel ties. All of these contribute to determining the wall's structural properties.

(a) Dressed stone in outer leaves and "rubble" fill **(b) Stone facing and brickwork backing** **(c) Stone facing and concrete core**

Figure 10.9: Stone masonry cross sections in New Zealand. Representative cases observed in Christchurch after the Canterbury earthquakes (Marta Giaretton)

Concrete block masonry

Although solid concrete masonry was used in New Zealand from the 1880s, hollow concrete block masonry was not used widely until the late 1950s. Masonry was usually constructed in running bond, but stacked bond was sometimes used for architectural effect.

From the 1960s onwards, masonry was usually constructed with one wythe 190 mm thick, although this was sometimes 140 mm thick. Cavity construction, involving two wythes with a cavity between, was mostly used for residential or commercial office construction but occasionally for industrial buildings. The external wythe was usually 90 mm thick and the interior wythe was either 90 mm or 140 mm. Cavity construction was often used for infills, with a bounding frame of either concrete or encased steelwork.

To begin with, reinforcement in concrete masonry was usually quite sparse, with vertical bars tending to be placed at window and door openings and wall ends, corners and intersections, and horizontal bars at sill and heads and the tops of walls or at floor levels. Early on, it was common to fill just the reinforced cells. Later, when the depressed web open-ended bond beam blocks became more available, more closely spaced vertical reinforcement became more practicable. When the depressed web open-ended bond beam blocks (style 20.16) became available without excessive distortion from drying shrinkage, these tended to replace the standard hollow blocks for construction of the whole wall (with specials at ends, lintels and the like).

Wire reinforcement formed into a ladder structure ("Bloklok" or a similar proprietary product) was common in cavity construction. Two wires ran in the mortar in bed joints, joined across the cavity by another wire at regular centres and acting as cavity ties.

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10.2.5 Constituent materials

10.2.5.1 Bricks

New Zealand brick sizes are based on imperial size. The most common nominal size of clay bricks used in masonry buildings is 230 mm x 110 mm x 70mm (9"x 4½"x 3½').

10.2.5.2 Mortar

Mortar is usually soft due to factors including inferior initial construction, ageing, weathering and leaching (Figure 10.10). Both the type and proportions of mortar constituents varied significantly throughout the country. Until early last century, lime-sand mortar was common but cement-lime-sand mortar and cement-sand mortar were also used.

Note:

While the lime in lime mortars will continue to absorb moisture and "reset", over time it will leach leading to deterioration of the mortar.

Figure 10.10: Soft mortar. Note the delaminated mortar from bricks in the background (Ingham and Griffith, 2011)

10.2.5.3 Timber

Totara, rimu, matai (black pine) and kahikatea (white pine) were the most commonly used timber species in URM buildings.

10.2.5.4 Concrete block

From the beginning, hollow concrete blocks were manufactured by the Besser process, where lean mix concrete was compacted into moulds using vibration. Concrete strength was usually 30 MPa or greater.

10.2.6 Floor/roof diaphragms

Floors of URM buildings were usually made from timber and sometimes from reinforced concrete slabs.

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10.2.6.1 Timber floors

Timber floor diaphragms are usually constructed of 19-25 mm thick tongue and groove (T&G) membrane nailed to timber joists that are supported by timber or steel beams. Matai, rimu and oregon were commonly used for the floor diaphragm membrane. These timbers may have hardened from a century of drying and be "locked up" from long use. The diaphragm may also have been damaged by insect infestation or decay from moisture ingress. As well as the timber characteristics, the response of these diaphragms during an earthquake is dictated by the behaviour of the nail joints. It should be recognised that the nails in use a century ago were much softer than those used today. Resistance comes primarily from friction between the boards, complemented by "vierendeel" action from the pairs of nails in a board. A further complication is that the response of timber diaphragms is different for each direction, recognising that joists and boards span in different directions. Hence diaphragm in-plane stiffness and strength should be assessed for earthquake loading oriented both parallel and perpendicular to the orientation of the joists.

10.2.6.2 Reinforced concrete slabs

Reinforced concrete slabs were usually monolithic to brick walls and form a rigid diaphragm. While they may have been reinforced with bars, as is commonly the case for modern construction, these bars were often round or of a roughness pattern that provides significantly less bond than expected today. As a result, the presence of termination details (such as hooks, thickenings, threads/nuts) will have a marked effect on the load carrying capacity. Other types of reinforcement included expanded metal lath (Figure 10.11) and even train rails.

Figure 10.11: Concrete slab with expanded metal lath reinforcement. Corrosion of the lath from carbonation of the concrete over time has caused the concrete to spall.

Portland cement gradually became available throughout New Zealand from the 1890s to the late 1920s, which was the time of much URM construction. Non-Portland cement concretes (often called "Clinker" concretes as they were produced from only a single firing of lime products) are significantly weaker and should be assessed with caution. Similarly, as concrete was a relatively expensive material during these times, voids or ribs were often formed in slabs using hollow ceramic tiles.

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Note:

Take care when making assumptions relating to the concrete strength. Intrusive investigation is essential to understand the makeup of the original slab construction and its constituents properly if forces greater than nominal are to be transferred.

10.2.6.3 Roofs

The roof structure is usually provided with straight sarking (Figure 10.12) or diagonal sarking (Figure 10.13) nailed to purlins supported by timber trusses. Straight sarking has similar action to flooring, but boards are often square edges so do not have the stiffness and strength of the high-friction tongue and groove connection. Diagonal sarking is naturally stiffer and stronger than rectangular sarking because the boards provide the diagonal "truss" members between the rafters and purlins. However, its ductility and displacement capacity will be less than for rectangular sarking as movements will cause direct shearing of the fixings along the lines of the boards.

Note:

Refer to Section 10.8.3 for the capacities of these types of systems. This is also covered in more detail in Section 11.

(a) Typical horizontal roof sarking

(b) Roof diaphragm with vertical sarking

Figure 10.12: Typical timber diaphragms – straight sarking

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Figure 10.13: Typical timber diaphragms - diagonal sarking

The strength of both floor and roof diaphragms is complemented by the ceiling sheathing material. Common types of ceilings that provide structural capacity are timber lath-and-plaster, fibrous plaster, steel lath-and-plaster, and pressed metal. More modern additions of plywood boards and plasterboard may have also occurred over time.

10.2.7 Diaphragm seating and connections

URM buildings are characterised by absent or weak connections between various structural components.

Often, walls parallel to the joists and rafters are not tied to the floors and roof respectively (Figure 10.14), except in a few cases depending on the design architect. Wall-diaphragm anchor plates, sometimes referred to as rosettes or washers, have been used to secure diaphragms to walls since the late 19th century (Figure 10.15). If these are present in a building, they may have been installed during the original construction or at any time since as a remediation measure.

Figure 10.14: A lack of connection of the walls parallel to joist and rafters with diaphragms and return walls leading to collapse of wall under face load

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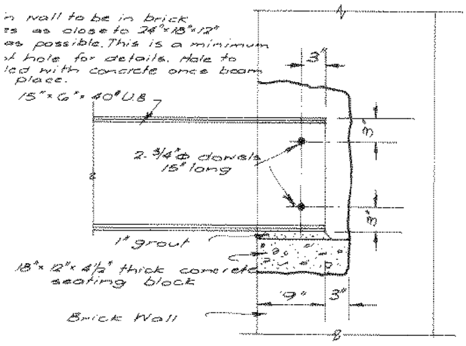
Figure 10.15: 1896 image showing anchor plate connections installed in early URM construction (National Library of New Zealand)

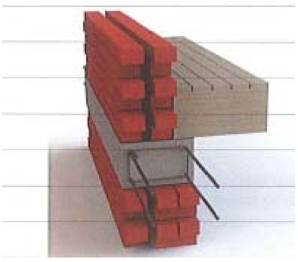
Even where walls are carrying beams, joists or rafters, they are not always secured to these elements. Connections made of steel straps tying the beams, joists or rafters to walls have been observed (Figures 10.16 and 10.17), sometimes with a fish-tail cast into concrete pockets.

Another common feature is a gap on either side of the timber joists and beams to avoid moisture transfer from brickwork to timber. With such connections, horizontal shear cannot be transferred from walls to joists. However, if the joists are set tightly in the pocket they can be effective in horizontal shear transfer between the wall and floor structure.

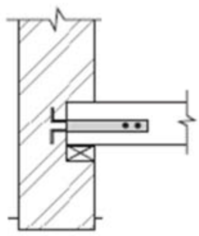


(a) Steel beam to wall pocketed connection

(b) Floor joist to wall connection. Note presence of steel strap (Matt Williams)



(c) Floor seating arrangement



(d) Fish-tail connection between wall and joist

Figure 10.16: Typical connection between masonry walls and joist

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(a) Wall to roof truss connection (Miyamoto International)

(b) Roof seating arrangement and parapet wall (Dymtro Dizhur)

(c) Wall to roof truss connection. Note truss is seated on a concrete pad stone (Miyamoto International)

Figure 10.17: Typical wall to roof connections

10.2.8 Wall to wall connections

In most cases, there are no mechanical connections provided to tie orthogonal walls together. Concrete bands may be provided but may not be tied together at corners as it is possible that they were built by different teams at different stages. If they are jointed, it may just be with intermittent steel ties, or bricks pocketed into the abutting walls which have very little tie or shear capacity.

10.2.9 Damp-proof course (DPC)

Most traditional buildings incorporate a damp-proof course (DPC) in the masonry between foundations and ground floor level. This can be made from galvanised metal, lead, slate, thick bitumen or bitumen fabric.

The DPC layer usually forms a slip plane (Figure 10.18(a)) which is weaker than the surrounding masonry for sliding. It also forms a horizontal discontinuity which can affect bond for face loading or hold-down of walls for in-plane loading. Sliding on the DPC layer has been recorded, as shown in Figure 10.18(b).

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Consideration of the DPC layer is an important part of establishing the capacity of the wall: refer to Section 10.8.6 for details.

(a) DPC below timber – Chest Hospital, Wellington

(b) Bitumen DPC and sliding evident after Cook Strait earthquakes

Figure 10.18: Common DPC materials

10.2.10 Built-in timber

Most traditional URM buildings incorporate built-in timbers (Figure 10.19) for:

- fixing of linings, skirting, cornices and dado/picture rails
- plates supporting intermediate floor joists
- forming header connections between wall layers, and
- top plates for affixing rafters or trusses.

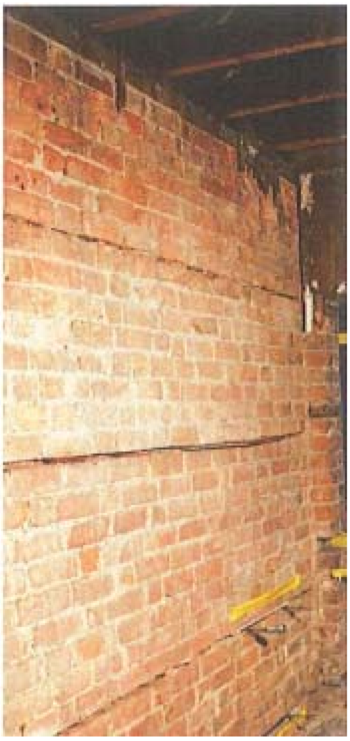

Figure 10.19: 12 mm timber built into every eighth course for fixing linings

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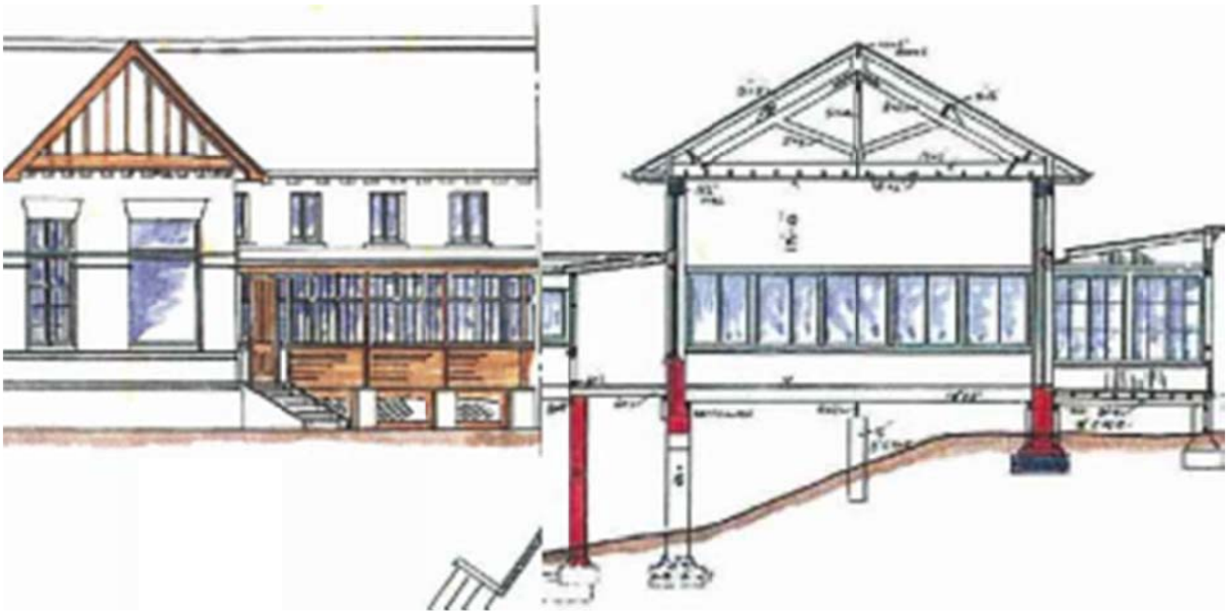
Degradation of these items is common, which causes localised stresses or bowing of walls. This will typically be more severe on the south side of buildings or nearer the ground level. Timber also shrinks, particularly perpendicular to the grain, and such timbers are often not in full contact with the surrounding masonry. In the case of continuous timber plates, engagement with the masonry is often limited to localised timber blocks notched into the walls.

10.2.11 Bond beams

Bond beams or perimeter tie beams (Figure 10.20) were typically constructed of reinforced concrete, plain concrete or timber. They can provide significant benefits to the performance of masonry buildings, including:

- providing a larger, often stronger substrate for the attachment of fixings and thereby providing better load distribution
- distributing diaphragm loads along the length of a wall
- tying wythes together in cavity construction (Figure 10.20), provided that the bond beam is laid over both wythes
- providing coupling between wall panels for in-plane loads
- providing longitudinal tying to spandrel beams, and
- providing out-of-plane stability to face-loaded walls.

Depending on the age of the structure, there may be poor/no hook or termination details in reinforced concrete bond beams, so concentrated loads near the ends of such bond beams should be avoided. Stirrup reinforcement in these beams is often nominal – if present at all – so care should be taken when shear loads are being applied to these elements.



(a) Bond beam in cavity wall also forming lintel – Chest Hospital, Wellington (Dunning Thornton)

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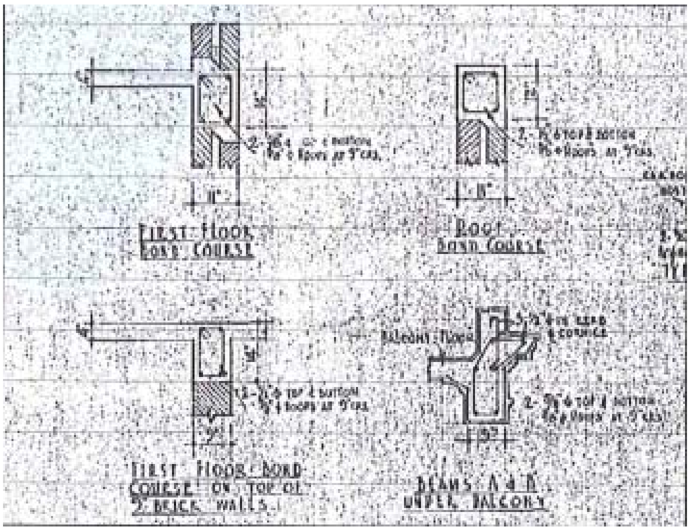

(b) Typical lintel detail (Dizhur)

Figure 10.20: Bond beams

The presence of a concrete band provides no surety that reinforcement is present. Figure 10.21 shows a concrete capping beam that is obviously not reinforced.

The reinforcement in the beam may also have degraded or may soon degrade if carbonation/chloride attack has penetrated into the concrete to the depth of reinforcing). When severe, this will split the concrete.

Figure 10.21: The wide cracks through bond beams indicate a lack of reinforcement in the beam (Dizhur)

10.2.12 Bed-joint reinforcement

Bed-joint reinforcement (course reinforcement) varies in type and application. It can include:

- single wires or pairs of wires laid in mortar courses to augment in-plane performance
- single wires or pairs of wires laid in mortar courses to act as lintels or ties to soldier courses
- prefabricated/welded lattices laid in multi-wythe walls to ensure bond

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- prefabricated/welded lattices laid across cavity walls to form cavity ties
- cast iron oversize cavity ties laid in multi-wythe walls to ensure bond, and
- chicken mesh.

Bed-joint reinforcement is often small in size relative to a fairly massive wall. It adds robustness but usually does not add significant structural strength.

This type of reinforcement is not usually apparent from a visual inspection. However, the requirement for bed joint reinforcement was often noted in the original masonry specifications.

10.2.13 Lintels

Lintels commonly comprise:

- reinforced concrete beams the full width of the wall
- reinforced concrete beams behind a decorative facing course, with this facing course supported on cavity ties or a steel angle
- steel angles
- steel flats (shorter spans)
- timber piece
- soldier course arches or flat arches, and
- stone lintels.

Arches or flat arches add a permanent outward thrust to a building which can destabilise walls in plane. This thrust along with any other forces should be resisted by ties in the building.

Reinforced concrete beams can contribute to in-plane pier/wall behaviour as they effectively reinforce the spandrel. However, they concentrate bearing loads at their supports and, if such frames dilate, can be points of overloading or destabilisation. They are also useful components for attachments for diaphragms (if the window heads are sufficiently high) as they provide a robust, blocky element to connect to.

10.2.14 Secondary components

Parapets are commonly placed on top of the perimeter walls. They are usually positioned off centre from the wall beneath, and capping stones or other ornamental features are then attached to the street side. Roof flashings are often chased into the brickwork on the external face just above roof level, creating a potential weak point in the masonry where rocking can occur.

Note:

Parapets, chimneys, pediments, cornices and signage (Figure 10.22) on street frontages present a significant hazard to the public. The Ministry of Business, Innovation and Employment has issued a determination (2012/043) clarifying that external hazards such as these must be included in the seismic assessment rating of a building.

Partition walls are other secondary elements which are usually not tied to the ceiling diaphragm and can pose a serious threat to life safety.

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Figure 10.22: Secondary elements (Miyamoto International)

10.2.15 Seismic strengthening methods used to date

Many URM buildings have been strengthened over the years either because of legislative requirements (e.g. earthquake-prone building legislation) or post-earthquake reconstruction (e.g. following the 1942 Wairarapa earthquake).

A number of strengthening techniques have been used (Ismail, 2012). The main principles were to tie unrestrained components, such as chimneys and parapets, to the main load-bearing structure and to tie various building components together so the building could act globally as a box with the intention that the available lateral capacity of the building could be fully mobilised even though it may not always have been increased.

Note:

Before 2004, seismic strengthening requirements for URM buildings were very low. In addition, in most strengthening projects the material properties were not verified by testing, anchors were mostly untested, and they were installed without documented quality assurance procedures.

Assessment of previously retrofitted buildings requires an understanding of the retrofit measures that historically have been carried out and the likely effect these would have on the seismic performance.

Techniques used historically for strengthening different structural mechanisms include:

- chimneys: internal post-tensioning and steel tube reinforcement, concrete filling, external strapping and bracing, removal and replacement
- parapets: vertical steel mullions, raking braces, steel capping, post-tensioning, internal bonded reinforcement, near surface mounted (NSM) composite strips
- face-loaded walls: vertical steel or timber mullions, horizontal transoms, post-tensioning, internal bonded reinforcement, composite fibre overlay, NSM composite strips, reinforced concrete or cementitious overlay, grout saturation/injection, horizontal and vertical reinforced concrete bands.

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- wall-diaphragm connections: steel angle or timber joist/ribbon plate with either grouted bars or bolts/external plate, blocking between joists notched into masonry, external pinning to timber beam end or to concrete beam or floor, through rods with external plates, new isolated padstones, new bond beams
- diaphragm strengthening: plywood overlay floor or roof sarking, plywood ceiling, plywood/light gauge steel composite, plasterboard ceiling, thin concrete overlay/topping, elastic cross bracing, semi-ductile cross bracing (e.g. Proving ring), replacement floor over/below with new diaphragm
- in-plane wall strengthening/ new primary strengthening elements: sprayed concrete overlay, vertical post-tensioning, internal horizontal reinforcement or external horizontal post-tensioning, bed-joint reinforcement, composite reinforced concrete boundary or local reinforcement elements, composite FRP boundary or local reinforcement elements, nominally ductile concrete walls or punched wall/frame or reinforced concrete masonry walls, nominally ductile steel concentric or cross bracing, limited ductility steel moment frame or concrete frame or concrete walls or timber walls, ductile eccentrically braced frame/K-frames, ductile concrete coupled or rocking walls, or tie to new adjacent (new) structure
- reinforcement at wall intersections in plan: removal and rebuilding of bricks with inter-bonding, bed-joint ties, drilled and grouted ties, metalwork reinforcing internal corner, grouting of crack
- foundation strengthening: mass underpinning, grout injection, concentric/balanced re-piling, eccentric re-piling with foundation beams, mini piling/ground anchors
- façade wythe ties: helical steel mechanical engagement – small diameter, steel mechanical engagement – medium diameter, epoxied steel rods/gauze sleeve, epoxied composite/non-metallic rods, brick header strengthening
- canopies: reinforce or recast existing hanger embedment, new steel/cast iron posts, new cantilevered beams, deck reinforcement to mitigate overhead hazard, conversion to accessible balcony, base isolation.

Figures 10.23 to 10.27 illustrate some of these techniques. A detailed table (Table 10.2) is included in Section 10.6.11. This table lists common strengthening techniques and particular features or issues to check for each method.

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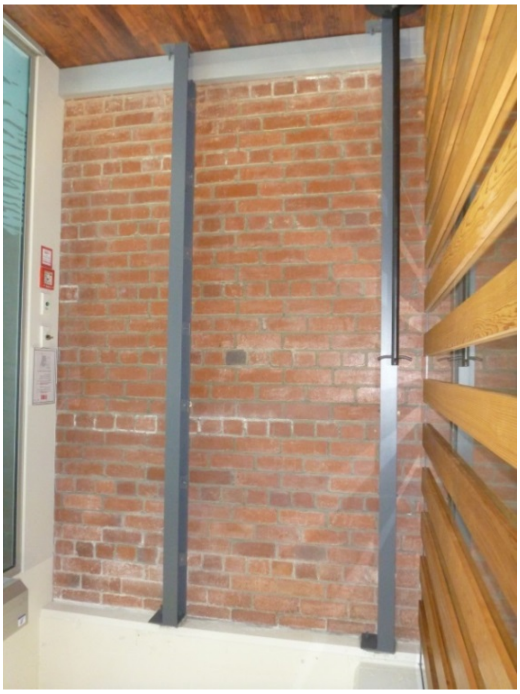
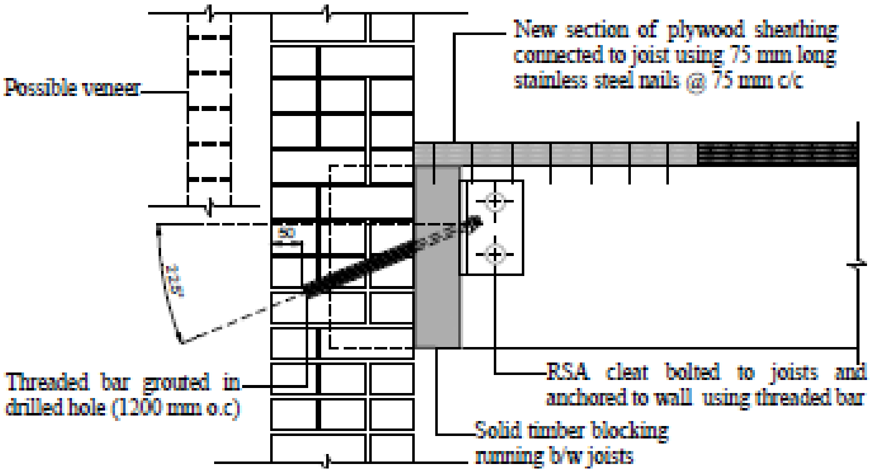
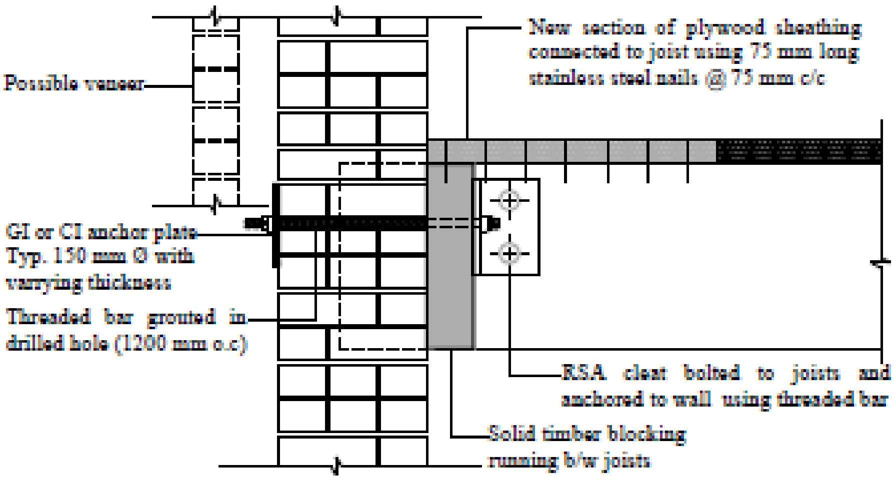


Figure 10.23: Bracing of wall against face load (Dunning Thornton)



(a) Bent adhesive anchor



(b) Through anchor with end plate (plate anchor)

Figure 10.24: Wall-diaphragm connections (Ismail, 2012)

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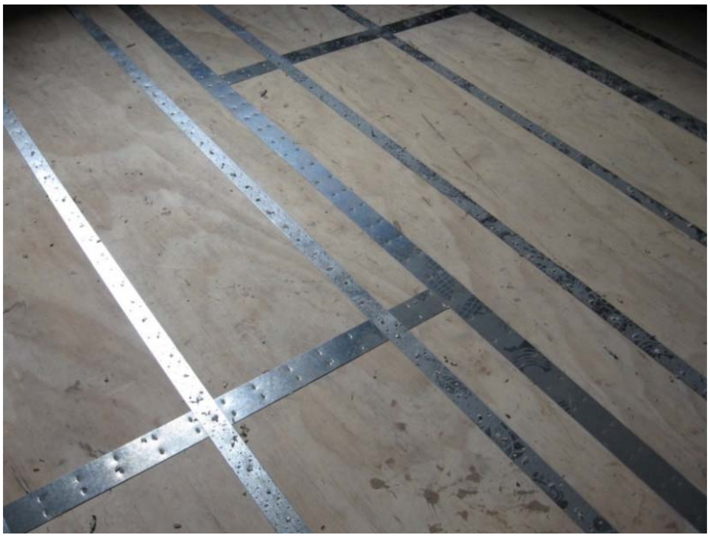
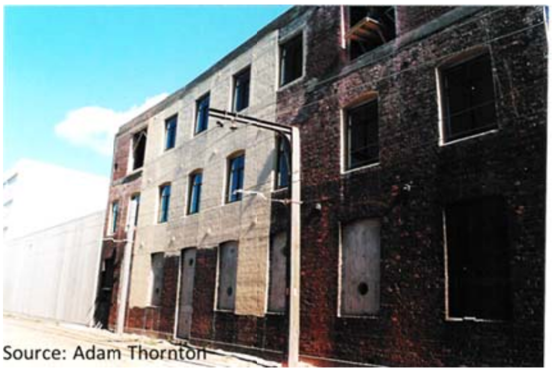


Figure 10.25: New plywood diaphragm (Holmes Consulting Group)

(a) Concentric steel frame (Beca)

(b) Steel frame (Dizhur)



(c) FRP overlay

(d) Steel frame (Dunning Thornton)

Figure 10.26: Improving in-plane capacity of URM walls

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Strengthening of parapets is often done using racking braces, with one end tied to the timber roof structure (refer to Figure 10.27). However, issues with this method include a lack of vertical tie-down to counter the vertical force component of brace and ground shaking, or the flexibility of the roof amplifying shaking of the parapet.

Note:

When strengthening parapets, it is essential to make a robust connection down to the wall below and back into the structure. The danger of non-robust strengthening is that the parapet still fails, but collapses in larger, more dangerous pieces.

Figure 10.27: Parapet bracing. Note a lack of vertical tie-up of the parapet (Dmytro Dizhur)

10.3 Observed Seismic Behaviour of URM Buildings

10.3.1 General

When assessing and retrofitting existing URM buildings it is important to understand the potential seismic deficiencies and failure hierarchy of these buildings and their components.

The most hazardous of these deficiencies are inadequately restrained elements located at height, such as street-facing façades, unrestrained parapets, chimneys, ornaments and gable end walls. These are usually the first elements to fail in an earthquake and are a risk to people in a zone extending well outside the building perimeter.

The next most critical elements are face-loaded walls and their connections to diaphragms and return walls. Even though their failure may not lead to the building's catastrophic collapse, they could pose a severe threat to life safety.

However, when building components are tied together and out-of-plane failure of walls is prevented, the building will act as a complete entity and in-plane elements will come under lateral force action.

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Failures of URM buildings (summarised in Figure 10.28) can be broadly categorised as:

- **local failures** – these include the toppling of parapets, walls not carrying joists or beams under face load, and materials falling from damaged in-plane walls. These local failures could cause significant life-safety hazards, although buildings may still survive these failures.
- **global failures** – these include failure modes leading to total collapse of a building due to such factors as loss of load path and deficient configuration.

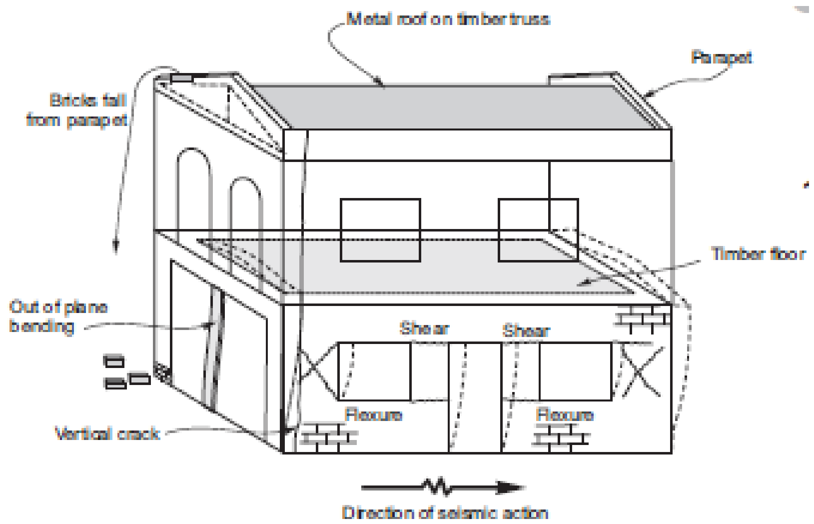

Figure 10.28: Failure modes of URM buildings

In URM buildings, in-plane demands on walls decrease up the height of the walls. In-plane capacity also decreases with height as the vertical load decreases. In contrast, out-of-plane demands are greatest at the upper level of walls (Figure 10.29), but out-of plane capacity is lowest in these areas due to a lack of vertical load on them. Hence, the toppling of walls starts from the top unless these are tied to the diaphragm.

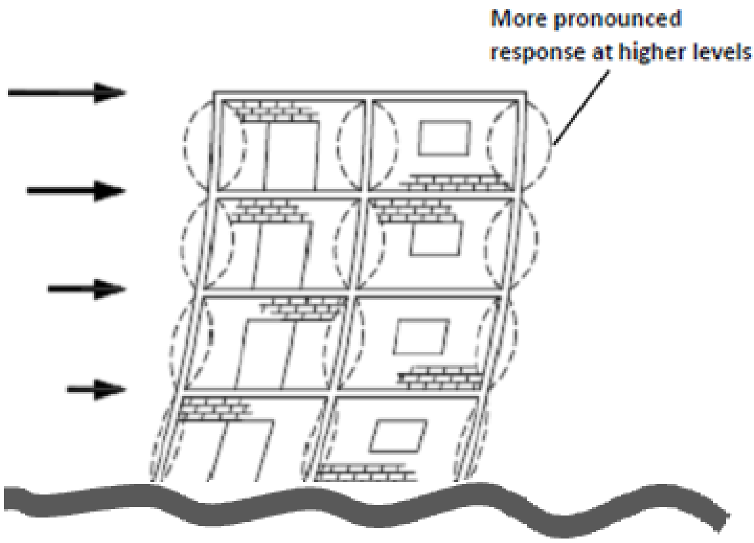

Figure 10.29: Out-of-plane vibration of masonry walls are most pronounced at the top floor level (adapted from Tomazevic, 1999)

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10.3.2 Building configuration

Building configuration tends to dictate the nature of URM failures. Cellular type buildings act as stiff structures, attracting high accelerations and therefore force-governed failure of their parts. Collapse of walls under face load as they try to span vertically and horizontally between floors and abutting walls respectively tends to be independent for each cell, depending on the angle of loading and the wall configuration.

Buildings where the span or flexibility of the diaphragm is an order of magnitude more than the walls tend to have more displacement-related failures. Walls and parapet collapse initiates from the mid-span of the diaphragm where movements are greatest (but accelerations are necessarily as high).

Taller buildings exhibit less damage at low levels than shorter buildings (Figure 10.30), as the confinement of the masonry from the weight above provides significant strength. In larger buildings, the weaker elements (usually spandrels) fail first from bottom up (Figure 10.46). This results in period lengthening of the structure and reduces the ability to transmit forces up the building.

As with all structures, the behaviour of URM buildings with a more regular configuration is generally more predictable. Buildings with irregular plan configurations, such as those on street corners (especially with an acute angle corner), suffer high displacements on their outer points. Shop fronts similarly experience high drifts, but these are often masked by “buttressing” from adjacent buildings in a “row” effect. This effect also disguises a vertical irregularity in which stiff façades tend to move as a solid element above the flexible open shop front.

Figure 10.30: Reduction of damage towards base of building as axial load increases (Dunning Thornton)

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10.3.3 Diaphragms

The timber diaphragms commonly used in URM buildings are generally flexible, which may result in large diaphragm displacements during an earthquake. These will impose large displacement demand on the adjoining face-loaded walls, which could lead them to fail (Figure 10.31).

Figure 10.31: Out-of-plane wall failure due to excessive roof diaphragm movement (Dizhur et al, 2011)

Figure 10.32 shows a photograph of delamination of plaster due to interaction between wall and ceiling due to shear transfer.

Figure 10.32: Lath and plaster ceiling. Note that stresses where shears are transmitted to the wall have caused the plaster to delaminate from the timber lath.

In some cases, diaphragm and shear-wall accelerations can increase with the flexibility of the diaphragm (Tena-Colunga & Abrams, 1996).

10.3.4 Connections

10.3.4.1 General

The following types of damage to wall-diaphragm connections have been postulated (Campbell et al, 2012) – the first four were actually observed during the 2010/11 Canterbury earthquake sequence:

- punching shear failure of masonry
- yield or rupture of connector rod
- rupture at join between connector rod and joist plate

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- splitting of joist or stringer
- failure of fixing at joist
- splitting or fracture of anchor plate
- yield or rupture at threaded nut.

10.3.4.2 Wall to wall connections

Connections between the face-loaded and return walls will open (i.e. there is return wall separation) after a few initial cycles of shaking (Figure 10.33) because of stiffness incompatibility between stiff in-plane and flexible face-loaded walls and a natural dilation of a wall and pier assembly working in plane. This leads to loss of flange effect and softening of the building, resulting in a change in dynamic characteristics of the walls and piers. The integrity of connection between wall at junctions and corners depends on bonding between orthogonal walls.

While return wall separation can cause significant damage to the building fabric it does not necessarily constitute significant structural damage. This is provided the wall elements have adequate out-of-plane capacity to span vertically and there are enough wall diaphragm ties.

(a) Vertical cracks (Dmytro Dizhur)

(b) Corner vertical splitting where walls poorly keyed in together

Figure 10.33: Damage to in-plane and face-loaded wall junctions

10.3.4.3 Wall to floor/wall to roof connections

Failure of rosettes, rupture of anchor bars and punching shear failure of the wall was commonly observed following the 2010/11 Canterbury earthquake sequence (Figure 10.34). This failure mode is characterised by failure of the mortar bed and head joints in a manner that traces a failure surface around the perimeter of the anchor plate. For

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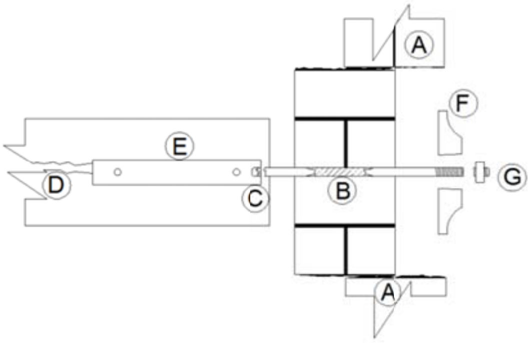
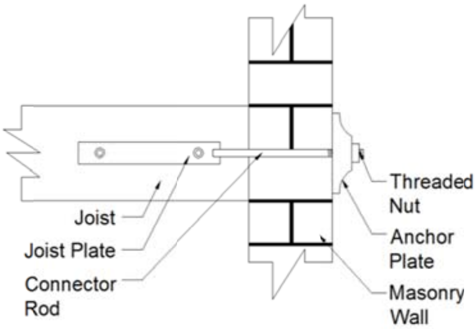
multi-wythe walls the head joints will not be in alignment and, as for a concrete punching shear failure, it is possible that the failure surface on the interior surface of the wall may cover a broader area.

Figure 10.34: Plate anchor on verge of punching shear failure (Dizhur et al, 2011)

Testing at the University of Auckland (Campbell et al., 2012) has shown that anchor plates may exhibit a variety of different failure modes (refer to Figures 10.35 and 10.36 for examples) so their condition should be considered carefully.



(a) Location of failure modes



(b) Components of the connection assembly

Figure 10.35: Wall-diaphragm anchor plate failure modes (Campbell et al., 2012)

(a) Sample 1-02: Failure where previously necked

(b) Sample 2-01: Brittle failure of anchor plate

(c) Sample 2-02: Brittle failure where connector rod was fixed to joist plate

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(d) Sample 3: Failure where previously necked **(e) Sample 4: Failure at threaded region** **(f) Sample 6: Failure at threaded region**

Figure 10.36: Observed failure modes from tensile test series (Campbell et al., 2012)

Adhesive anchorages have been a popular form of anchorage for many years. These typically involve a threaded rod being chemically set into a drilled hole using either grout or epoxy adhesive. Unfortunately, there have been numerous observations of failed adhesive anchorages following the 2010/11 Canterbury earthquake sequence (Figure 10.37). Reasons for this include:

- Their use in regions expected to be loaded in flexural tension during an earthquake (such as on the rear surface of a parapet that may topple forward onto the street) – the brick work was likely to crack in the vicinity of the anchorages and cause them to fail, even if the adhesive had been placed effectively.
- Incorrect installation – examples included cases of insufficient or absent adhesive, where the drilled hole had not been sufficiently cleared of brick dust from the drilling operation so there was inadequate bond to the brick surface, or where the inserted anchorage was of insufficient length.
- Anchors that were adequately set into a brick but the secured brick had failed in bed-joint shear around its perimeter. As a result, only the individual brick was left connected to the anchorage, while the remainder of the brickwork had failed.

Figure 10.37: Failed adhesive brick anchors (Dizhur et al., 2013)

10.3.5 Walls subjected to face loads

Out-of-plane wall collapse under face load is one of the major causes of destruction of masonry buildings, particularly when a timber floor and roof are supported by these walls. The seismic performance of the URM face-loaded walls depends on the type of diaphragm, performance of wall-diaphragm connections and the wall-wall connection. Figure 10.38

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illustrates the response of face-loaded walls to the type of diaphragm and wall-diaphragm connections.

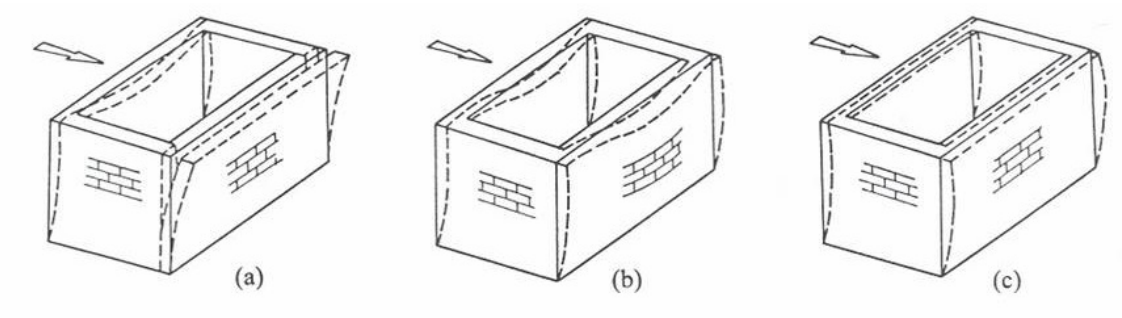

Figure 10.38: Effect of types of diaphragm on face-loaded walls – a) inferior wall-to-wall connection and no diaphragm, b) good wall-to-wall connection and ring beam with flexible diaphragm, c) good wall-to-wall connection and rigid diaphragm

Figures 10.39 and 10.40 show images of damage to masonry buildings due to collapse of walls under face load.

Figure 10.39: Out-of-plane instability of wall under face load due to a lack of ties between the face-loaded wall and rest of the structure (Richard Sharpe)

Gable end walls sit at the top of walls at the end of buildings with pitched roofs. If this triangular portion of the wall is not adequately attached to the roof or ceiling, it will rock as a free cantilever (similar to a chimney or parapet) so is vulnerable to collapse. This is one of the common types of out-of-plane failure of gable walls (Figure 10.40).

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Figure 10.40: Collapse of gable wall. Note a secured gable end that survived earthquake loading and a companion failed gable end that was not secured (Ingham & Griffith, 2011)

Cavity wall construction can be particularly vulnerable to face-loading. Severe structural damage and major collapse of URM buildings with this type of construction was observed during the 2010/2011 Canterbury earthquake inspections (Figure 10.41) and their performance was significantly worse than solid URM construction in resisting earthquake forces.

Figure 10.41: Failure of URM cavity walls (Dizhur)

The veneers of cavity wall construction also have the potential to topple during earthquake shaking (Figure 10.41). Toppling is typically attributed to the walls' high slenderness ratio, deteriorated condition of the ties, overly flexible ties, pull-out of ties from the mortar bed joints due to weak mortar, or a total absence of ties.

In multi-storey buildings the out-of-plane collapse of walls is more pronounced at the top floor level. This is due to the lack of overburden load on the walls and amplification of the earthquake shaking there (Figure 10.29).

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10.3.6 Walls subjected to in-plane loads

Damage to URM walls due to in-plane seismic effects (in the direction of the wall length) is less significant than damage due to out-of-plane seismic effects. In addition, the stocky elements in URM (walls, piers and spandrels) usually make these structures more forgiving of distress in individual elements than the skeletal structures of modern framed buildings; principally, because the spectral displacements are small compared to the member dimensions. Nevertheless, some failure modes are less acceptable than others.

In general, the preferred failure modes are rocking or sliding of walls or individual piers. These modes have the capacity to sustain high levels of resistance during large inelastic straining. For example, sliding displacements at the base of a wall can be tolerated because the wall is unlikely to become unstable due to the shear displacements.

Masonry walls are either unpenetrated or penetrated. A penetrated wall consists of piers between openings plus a portion below openings (sill masonry) and above openings (spandrel masonry). When subjected to in-plane earthquake shaking, masonry walls and piers may demonstrate diagonal tension cracking, rocking, toe crushing, sliding shear, or a combination of these. Similarly, the spandrels may demonstrate diagonal tension cracking, unit cracking or joint sliding. Figure 10.42 shows the potential failure mechanisms for unpenetrated and penetrated walls.

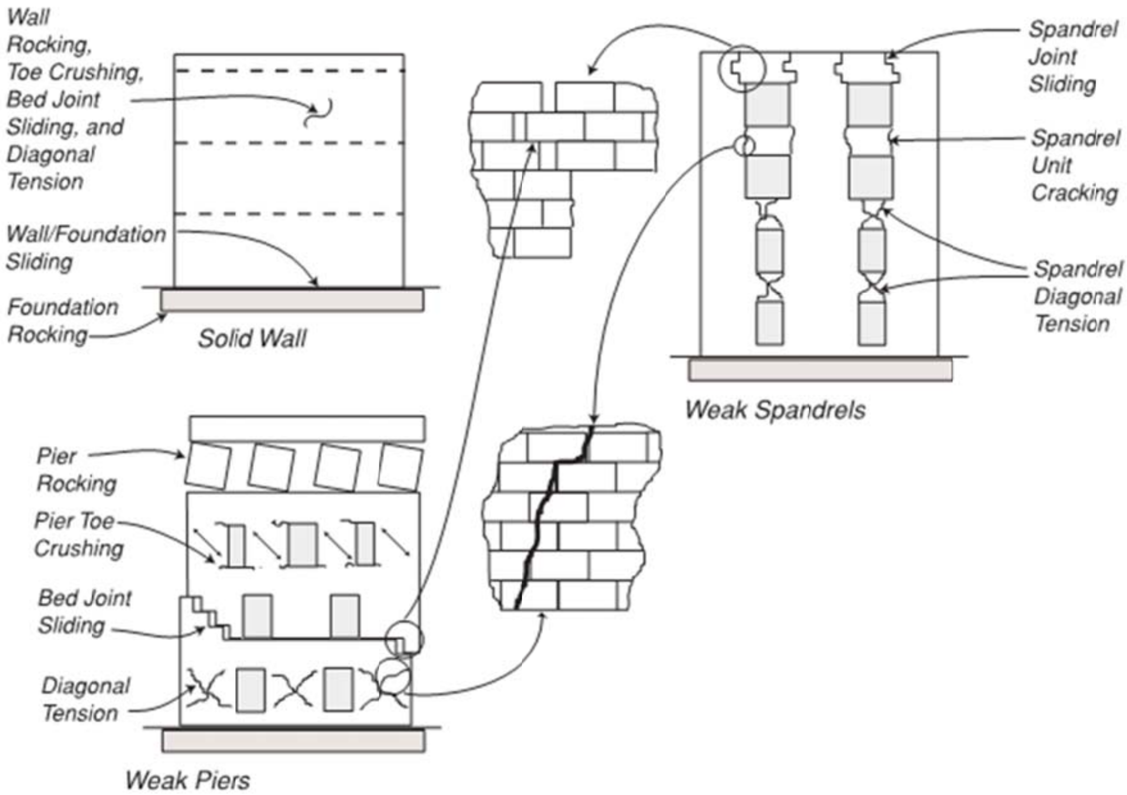

Figure 10.42: In-plane failure modes of URM wall (FEMA, 1998)

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Rocking of URM piers may result in the crushing of pier end zones and, under sustained cyclic loading, bricks could delaminate if the mortar is weak. An example of this is shown in Figure 10.43, where the damage to the building is characterised by the rotation of entire piers.

Figure 10.43: Rocking and delamination of bricks of a one-storey unreinforced brick masonry building with reinforced concrete roof slab (Bothara & Hiçyılmaz, 2008)

Sliding shear can occur along a distinctly defined mortar course (Figure 10.44(a)) or over a limited length of several adjacent courses, with the length that slides increasing with height (Figure 10.44(b)). This can often be mistaken for diagonal tension failure, which is less common in walls with moderate to low axial forces.

(a) Sliding shear failure along a defined plane at first floor level (Dunning Thornton)

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(b) Stair-step crack sliding, in walls with low axial loads (Bothara)

Figure 10.44: Sliding shear failure in a brick masonry building

Alternatively, masonry piers subjected to shear forces can experience diagonal tension cracking, also known as X-cracking (Figure 10.45). Diagonal cracks develop when tensile stresses in the pier exceed the masonry tensile strength, which is inherently very low. This type of damage is typically observed in long and squat piers and on the bottom storey of buildings, where gravity loads are relatively large and the mortar is excessively strong.

(a) Diagonal tension cracks to a brick pier. Note splitting of bricks (Dizhur)

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(b) Diagonal tension cracks to brick masonry. Note splitting of bricks, indicative of mortar stronger than bricks (Russell 2010)

Figure 10.45: Diagonal tension cracking

In the penetrated walls, where spandrels are weaker than piers, the spandrel may suffer catastrophic damage (Figure 10.46). This could turn squat piers into tall piers, resulting in a reduction in the overall wall capacity and an increase in expected deflections. The increase in deflection will increase the fundamental period of the building and reduce the demands which may be a mitigating effect. In any event the consequences of failure of the spandrels, and the resulting effect on life safety needs to be considered.

As noted in Section 10.2.9, sliding on the DPC layer has also been observed (Figure 10.18).

Figure 10.46: Failure of spandrels. Also note rocking of upper piers and corner cracking of the parapet (Dmytro Dizhur)

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10.3.7 Secondary components/elements

The instability of parapets and chimneys is caused by these elements acting as rocking cantilevers. which can topple when sufficiently accelerated (Figure 10.47). Braced chimneys and parapets also failed during the Canterbury earthquake sequence (Ismail, 2012). Possible reasons include:

- Bracing to the roof caused coupling with the vertical response modes of the roof trusses where the roof structure was flexible.
- Ties tying the parapets to the wall below the diaphragm level did not exist or were deficient.
- Strengthening standards were low (until 2004 the general requirement was to strengthen URM buildings to two thirds of NZSS 1900 (Chapter 8), 1965).
- Spacing between lateral support points too large.
- High vertical accelerations.
- Lack of deformation compatibility between support points (Figure 10.47(b)).

(a) Out-of-plane instability of parapet (Beca)

(b) Chimney at onset of falling (Dizhur)

Figure 10.47: Secondary components/elements

Canopies can be both beneficial and detrimental in relation to life safety (Figure 10.48):

- They are often hung off the face of the buildings so columns supporting their outer edge do not obstruct the footpath or roadway. When subject to vertical loads, these diagonal hangers act to pry the outer layers of brick off the face of the building at the connection point.
- However, if they are sufficiently robust in their decking and fixings or if they are propped, they can provide overhead protection by taking at least the first impact of any falling objects.

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Figure 10.48: Face-load failure of URM façade exacerbated by outward loadings from downward force on canopy. Note the adjacent propped canopy did not collapse. (Dunning Thornton)

10.3.8 Pounding

This failure mechanism only occurs in row-type construction (refer to Section 10.5.4) where there is insufficient space between adjacent buildings so they pound into each other when vibrating laterally during an earthquake. Many examples of pounding damage to URM buildings were observed following the 2010/11 Canterbury earthquake sequence (Figure 10.49).

Figure 10.49: Pounding failure (Cole)

The magnitude of pounding depends upon the floor alignment between adjacent buildings, the difference in stiffness between the buildings, the pounding surface, floor weights, and clearance of structural separation between adjacent buildings if separation is provided.

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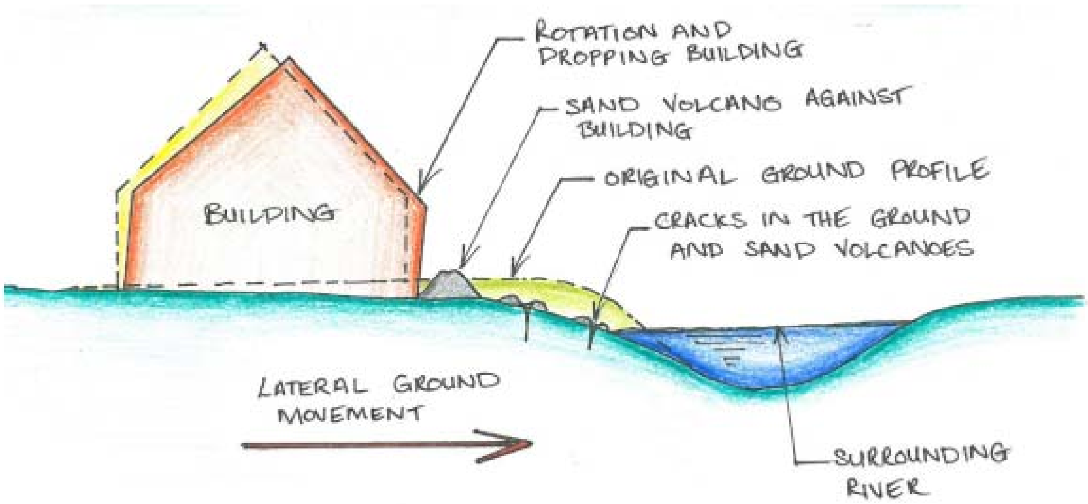
10.3.9 Foundations and geotechnical failure

Foundation damage that can be seen by inspection is commonly from lateral spreading and differential settlement. URM buildings typically have no tying capacity at foundation level, so they split at the weakest point along a wall. “Failure” is often an extremely large displacement (Figure 10.50). However, given the slower and non-cyclic nature of lateral spreading, this is less likely to induce actual collapse until extreme displacements are reached.

(a) Large diagonal cracks and lateral movement of the access ramp caused by ground movement



(b) Settlement and lateral spread towards river

Figure 10.50: Earthquake-induced geotechnical damage to URM buildings (Neill et al., 2014)

10.4 Factors Affecting Seismic Performance of URM Buildings

10.4.1 Number of cycles and duration of shaking

The strength and stiffness of URM degrades rapidly with an increasing number of cycles and the duration of ground shaking (Figure 10.51). In general, a number of cycles of moderate acceleration sustained over time can be much more difficult for an URM building to withstand than a single, much larger peak acceleration (FEMA, 2006). Similarly, damage from higher acceleration, shorter period ground shaking from shallow

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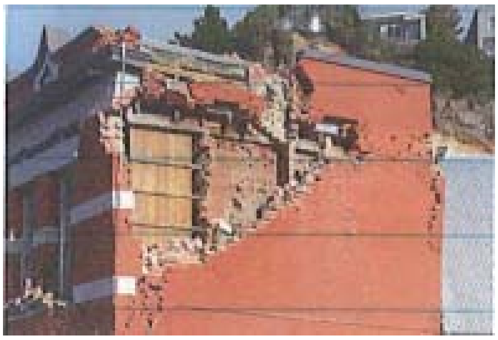
earthquakes could be considerably greater than from deep earthquakes. This could affect stiffer URM buildings far more than flexible frame and timber structures.

(a) Post-September 2010 event – minor visible damage

(b) Post-February 2011 event – wall section on verge of failure



(c) Post-June 2011 event – wall collapse

Figure 10.51: Progressive damage and effect of shaking duration – 2010/11 Canterbury earthquake sequence (Dmytro Dizhur)

Note:

The assessment of damaged buildings is outside the scope of these Guidelines, and therefore progressive deterioration after the main event is not considered. It is assumed that the building will have been appropriately stabilised if this had been required after the main event.

10.4.2 Other key factors

10.4.2.1 General

Other key factors affecting the seismic performance of URM buildings include:

- building form
- unrestrained components
- connections
- wall slenderness
- diaphragm deficiency
- in-plane walls
- foundations
- redundancy
- quality of construction and alterations, and
- maintenance.

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10.4.2.2 Building form

A structurally irregular building suffers more damage than a regular building because of the concentration of both force and displacement demands on certain components. An example of this is buildings along urban streets where the façades facing the street can be highly penetrated, with relatively narrow piers between openings, and the bottom storey could be totally open. This configuration could impose significant torsional demand and soft/weak storey mechanism. This can result in increased displacement demand and may lead to collapse.

10.4.2.3 Unrestrained components

Instability of parapets and chimneys is caused by their low bending strength and high imposed accelerations. When subject to seismic actions, they rock on their supports at the roof line and can topple over when sufficiently accelerated by an earthquake.

10.4.2.4 Connections

URM buildings can show significant resilience to seismic shaking as long as the building and its components can maintain their integrity. The wall-diaphragm anchors serve to reduce the vertical slenderness of a wall and also to make the building components work together as a whole, rather than as independent parts. However, one of the most significant deficiencies in URM buildings in New Zealand is the lack of adequate connections; particularly those between walls and diaphragms.

10.4.2.5 Wall slenderness

Unreinforced face-loaded masonry walls are weak in out-of-plane bending so are susceptible to out-of-plane failures. The earthquake vulnerability of a URM wall to out-of-plane bending is predominantly dictated by its slenderness (the ratio between thicknesses to span of wall). Cavity walls are especially vulnerable as the steel ties connecting the exterior wythes to the backing wall can be weakened by corrosion.

10.4.2.6 Diaphragm deficiency

Diaphragms act as a lid to a box and are essential for tying the walls together and ensuring that lateral loads are transferred to the lateral load-resisting elements. If diaphragms are too flexible, their ability to do this is compromised. Excessive diaphragm displacement imposes large displacement demand on walls, particularly on face-loaded walls, which could result in wall collapse.

10.4.2.7 In-plane walls

These walls provide global strength and stiffness against earthquake load. Their seismic performance is defined by: the slenderness of walls and piers; vertical load; size and location of penetrations; relative strength between mortar and masonry units; and presence of bond beams, built-in timber and DPC.

10.4.2.8 Foundations

Foundation flexibility and deformation affect the local and global earthquake response of URM buildings. However, foundations tend to be quite tolerant to deformations and building failure is rarely caused by ground settlement unless the ground underneath the building liquefies or suffers lateral spreading. Foundation effects or soil structure interaction tend to reduce the force demand on the primary lateral-force-resisting elements,

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such as stiff in-plane loaded walls. At the same time, ground deformation can pose an additional rotational demand on the bottom storey wall under face load. The base fixity of the wall needs to be considered carefully as do the conditions at the wall base that have accumulated over the building's life (such as undermining by broken drains, clay heave or alteration of the surrounding soil or levels), and if these have changed with earthquake-induced liquefaction.

Existing high bearing pressures require careful consideration with respect to possible liquefaction-induced settlements. Settlement of long solid walls is often not a critical consideration for a URM building as the upper floors and roof frame into the walls with pin connections. However, careful consideration of the induced damage to any perpendicular/abutting walls is essential. For taller walls, ratcheting down with cyclic in-plane actions may be a consideration (refer Section 14). With little or no reinforcement in the footings (or ground slabs if present), there will be little resistance to lateral spreading or ground lurch, so vulnerability to these induced displacements should be assessed.

10.4.2.9 Redundancy

Redundancy of a building refers to the alternative load paths able to add to resistance. The ability to redistribute demands through a secondary load path is an important consideration, as a building with low redundancy will be susceptible to total collapse if only one of its structural elements fails.

10.4.2.10 Quality of construction and alterations

URM buildings in New Zealand represent an old building stock which has gone through many changes of occupancy. As a result, there may have been a number of structural modifications at different times which may not have been well considered, such as opening new penetrations in walls and diaphragms, removing existing components and adding new components. Such alterations will affect seismic performance.

10.4.2.11 Maintenance

Older buildings that have been insufficiently maintained will have reduced material strength due to weathering (Figure 10.52), corrosion of cavity ties (Figure 10.53), rotting of timber and other processes that weaken masonry, connection capability, timber and reinforced concrete members. Similarly, water penetration in lime-based masonry will lead to leaching of lime from the mortar.

Figure 10.52: Severely degraded bricks and mortar due to moisture ingress (Ingham & Griffith, 2011)

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The metallic cavity ties used in the original construction of URM cavity walls typically have no corrosion protection so are prone to severe deterioration (Figure 10.53).

Figure 10.53: Metal cavity ties in rusted condition (Dizhur et al, 2011)

10.5 Assessment Approach

10.5.1 General

The assessment of URM buildings requires an understanding of the likely behaviour of a number of building components and how these are likely to interact with each other.

The nature of the construction of this type of building means that each building is unique in terms of construction, quality of the original workmanship and current condition.

It is, therefore, considered important that assessors of this type of building have an appreciation of how these buildings were constructed, their current condition, the observed behaviour of similar buildings in previous earthquakes and a holistic view of the factors likely to affect their seismic performance. These issues have been discussed in Sections 10.2, 10.3 and 10.4 which are considered to be essential reading prior to progressing through the assessment processes outlined in this section.

It is a general recommendation of these guidelines that the capacity of a building should be considered independently from the demands (imposed inertial loads and displacements) placed on it, bringing both together only in the final step of the assessment process. This is no different for URM buildings and is the basis behind the recommended assessment processes outlined below.

Past observations in earthquakes indicate that some components of URM buildings are particularly vulnerable to earthquake shaking and a hierarchy in vulnerability can be identified that can be useful in guiding the assessment process. Figure 10.54 shows a capacity "chain" for a typical URM building with component vulnerability decreasing from left to right on the chain. The capacity of the building will be limited by the capacity of the weakest link in the chain, and the ability of each component to fully develop its capacity will typically be dependent on the performance of components to the left of it on the chain. This suggests that the assessment of component capacities should also proceed from left to right in Figure 10.54.

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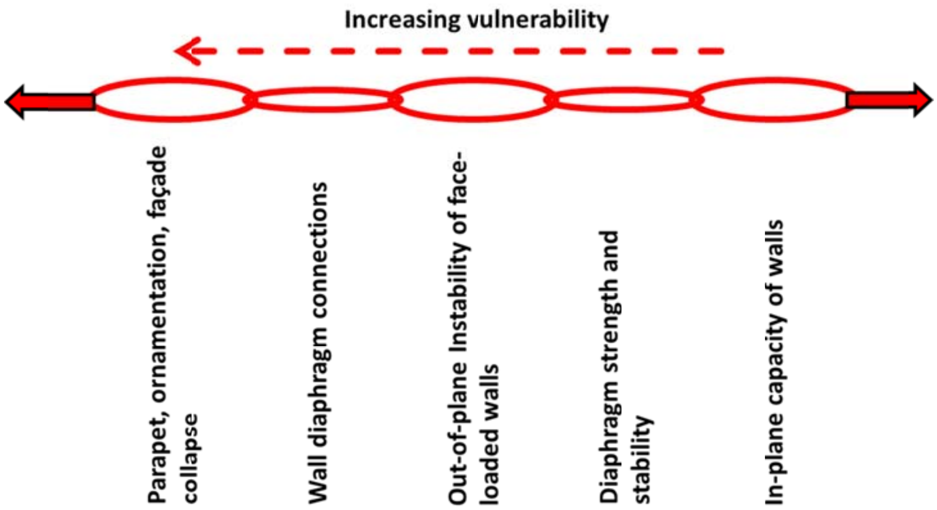

Figure 10.54: The capacity "chain" and hierarchy of URM building component vulnerability

While the critical structural weakness in a structural system will often be readily apparent (e.g. lack of any positive ties from brick walls to floors/roof) it will generally be necessary to evaluate the capacity of each link in the chain to fully inform on the components that require retrofit and the likely cost of this.

URM buildings come in different configurations, sizes and complexity. While complex buildings may require a first-principles approach to the assessment of component capacity and internal actions within components, simplifications are possible for more basic structures. Guidance is provided for both the detailed complete solutions and basic solutions for common simple buildings.

In Section 10.5.2 the assessment process, as it applies to URM buildings, is discussed with particular emphasis on how the approach might be varied depending on the complexity of the building. The assessment approach will also be influenced by any previous strengthening (Section 10.5.3), and its location (including when it is a row building (Section 10.5.4)).

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10.5.2 Assessment process

Key steps involved in the assessment of URM buildings are shown in Figure 10.55 and described below.

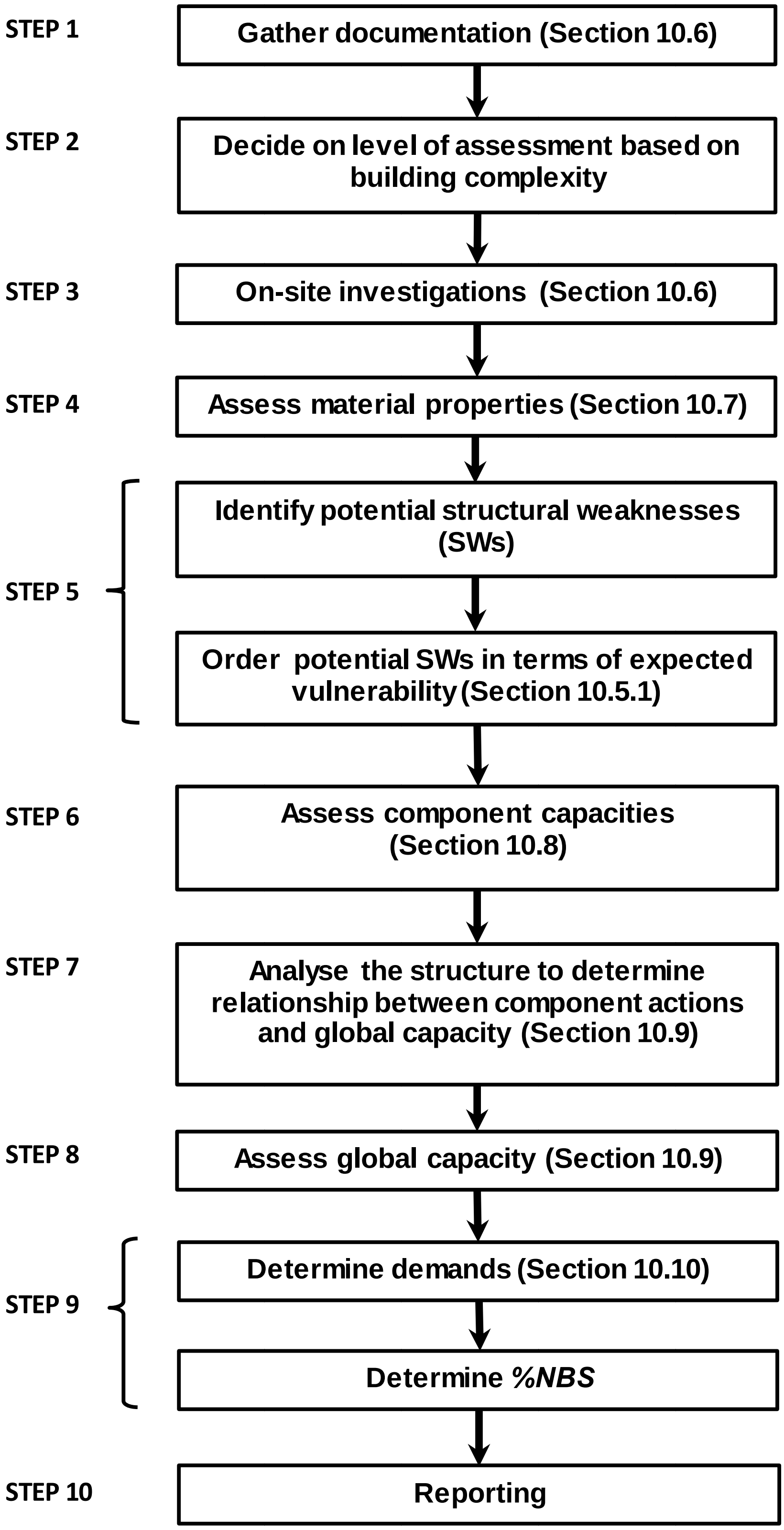

Figure 10.55: Assessment process for URM buildings

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Step 1 Gather documentation

- Collect relevant information and documents about the building including drawings, design feature reports, calculations and specifications, and any historical material test results and inspection reports (if available).
- If the building has been previously altered or strengthened, collect all available drawings, calculations and specifications of this work.
- Study this information before proceeding with the on-site investigation.

Step 2 Consider building complexity

- Determine an assessment strategy based on an initial appraisal of the complexity of the building. This can be reviewed as the assessment progresses.
- Although all aspects will need to be considered for all buildings, simplifications can be made for basic buildings e.g. one or two storey commercial, rectangular in plan. For these buildings the default material strengths are expected to be adequate without further consideration so that on-site testing, other than scratch testing of the bed joints to ascertain mortar type and quality, is not considered necessary. Foundation rotations are also not expected to have a significant effect so can be ignored.
- Concentration of effort should be on assessing the score for face-loaded walls, connections from the walls to the diaphragms and the diaphragms (lateral deflection between supported walls). The score for the walls in plane will depend on the ability (stiffness) of the diaphragm to transfer the shears but the calculations required are likely to be simple irrespective of whether the diaphragms are rigid (concrete) or flexible (timber, steel braced). Behaviour can be assumed to be linear-elastic (ie ignore any non-linear behaviour).
- Complexity is likely to be increased if a building has previously been retrofitted. Not all issues with the building will necessarily have been addressed in historical retrofits. Stiffness compatibility issues will often not have been considered or fully addressed.

Step 3 Investigate on-site

- Refer Section 10.6.
- Evaluate how well the documentation describes the "as constructed" and, where appropriate, the "as strengthened" building.
- Carry out a condition assessment of the existing building.
- Complete any on-site retrieval of samples and test these.
- Identify any site conditions that could potentially affect the building performance. (Refer Section 14).

Step 4 Assign material properties

- Start by using the probable material properties that are provided in Section 10.8, or establish actual probable values through intrusive testing (this may be a step you come back to depending on the outcome of your assessment).
- Recognise that for basic buildings obtaining building-specific material strengths through testing may not be necessary to complete an assessment.

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Step 5 Identify potential structural weaknesses and relative vulnerability

- The first step is to identify all of the various components in the building and then to identify potential SWs related to these.
- The identification of potential SWs in this type of building requires a good understanding of the issues discussed in Sections 10.2, 10.3 and 10.4.
- Early recognition of SWs and their relative vulnerability and interdependence is likely to reduce assessment costs and focus the assessment effort.
- Prior experience is considered essential when identifying the SWs in complex buildings.
- Separate the various components into those that are part of the primary lateral load resisting system and those that are not (secondary components). Some components may be categorised as having both a primary lateral load resisting function (e.g. in-plane walls and shear connections to diaphragms) and a secondary function (e.g. face-loaded walls and supporting connections).
- The relative vulnerability of various components in typical URM buildings is likely to be (refer also Figure 10.54):
  - Inadequately restrained elements located at height; such as street-facing façades, unrestrained parapets, chimneys, ornaments and gable end walls. Collapse of these components may not lead to building collapse but they are potential life-safety hazards and therefore their performance must be reflected in the overall building score.
  - Inadequate connection between face-loaded walls and floors/roof; little or no connection capacity will mean that the walls will not be laterally supported when the inertial wall forces are in a direction away from the building and then it can be easily concluded that the walls and/or connections will be unlikely to score above 34*%NBS*, except perhaps in low-seismic regions. If observations indicate reasonable diaphragm action from the floors and/or roof, adequate connections will mean that the out-of-plane capacity of the face-loaded walls may now become the limiting aspect.
  - Out-of-plane instability of face-loaded walls. If the wall capacity is sufficient to meet the requirements set out for face-loaded walls, then the capacity of the diaphragms becomes important as the diaphragms are required to transfer the seismic loads from the face-loaded walls into the in-plane walls.
  - The in-plane capacity of walls: these are usually the least vulnerable components.

Step 6 Assess component capacities

- Calculate the seismic capacities from the most to the least vulnerable component, in turn. There may be little point in expending effort on refining existing capacities only to find that the capacity is significantly influenced by a more vulnerable item that will require addressing to meet earthquake-prone requirements or target performance levels. Connections from brick walls to floors/roof diaphragms are an example of this. Lack of ties in moderate to high seismic areas will invariably result in an earthquake-prone status for the masonry wall and therefore it may be more appropriate and useful to assess the wall as < 34*%NBS* and also calculate a capacity assuming ties are in place. This will inform on the likely effect of retrofit measures.
- A component may consist of a number of individual elements. For example, the capacity of a penetrated wall (a component) loaded in-plane will need to consider the

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likely behaviour of each of the piers and the spandrel regions between and above and below the openings respectively (the elements). For some components the capacity will be a function of the capacity of individual elements and the way in which the elements act together. To establish the capacity of a component may therefore require structural analysis of the component to determine the manner in which actions in the elements develop.

- For each component assess whether or not exceeding its capacity (this may be more easily conceptualised as failure for these purposes) would lead to a life safety issue. If it is determined that it will not, then that component can be neglected in the assessment of the expected seismic performance of the structure. The same decisions may need to be made regarding the performance of elements within a component.

Step 7 Analyse the global structure

- In general, the complexity and extent of the analysis should reflect the complexity of the building.
- Start with analyses of low sophistication, progressing to greater sophistication only as necessary.
- An analysis of the primary lateral load resisting structure will be required to determine the relationship between the global capacity and the individual component actions.
- The analysis undertaken will need to recognise that the capacity of components will not be limited to consideration of elastic behaviour. Elastic linear analysis will likely be the easiest to carry out but the assessor must recognise that restricting to elastic behaviour will likely lead to a conservatively low assessment score.
- The analysis will need to consider the likely impacts of plan eccentricities (mass, stiffness and/or strength).

Step 8 Assess global capacity

- From the structural analyses determine the global capacity of the building. This will be the capacity of the building as a whole determined at the point that the most critical component of the primary lateral load resisting system reaches its determined capacity.
- It may also be useful to determine the global capacity assuming successive critical components are addressed (retrofitted). This will inform on the extent of retrofit that would be required to achieve a target score.

Step 9 Determine the demands and *%NBS*

- Determine the global demand for the building from Section 5 and assess the global *%NBS* (global capacity/ global demand x 100).
- Assess the demands on secondary components and parts of the building and assess *%NBS* for each (capacity/demand x 100).
- List the *%NBS* values in a table.
- The CSW will be the item in the table with the lowest *%NBS* score and that *%NBS* becomes the score for the building.
- Review the items in the *%NBS* table to confirm that all relate to elements, the failure of which would lead to a life safety issue. If not, revise the assessment to remove the non-life safety element from consideration.

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Step 10 Reporting

- Refer Section 12.

10.5.3 Assessment of strengthened buildings

Seismic assessment of URM buildings that previously have been strengthened is similar to that undertaken for un-strengthened structures except that the performance of previously installed strengthening components has to be taken into account. (Table 10.2 in Section 10.6 provides a detailed list of strengthening techniques used in URM buildings and associated features.)

Issues requiring consideration include the capacity of the installed elements, diaphragm continuity, and deformation compatibility between the original and installed strengthening elements.

10.5.3.1 Wall-to-diaphragm anchors

The effectiveness of existing wall-to-diaphragm anchors needs to be verified. Examples of poorly performing anchors that are known to have been used in previous strengthening projects include:

- Shallow embedment grouted anchors. Anchors installed with low embedment depths (i.e. less than half the wall thickness) were observed to perform poorly under face loads (Moon et al. 2011).
- Grouted plain round bar anchors. Plain round bars have a low bond strength compared with threaded bar or deformed reinforcing bar anchors.
- Mechanical expansion anchors. Mechanical anchors do not generally perform well in URM due to the low tensile capacity of masonry and the limited embedment depths that can be achieved with available mechanical anchors.

The default connector strengths detailed in Section 10.8.4 can be used for existing wall to diaphragm anchors that are in good condition and are known to have been installed and tested in accordance with the requirements of Appendix 10A.

Existing non-headed wall anchors of unknown construction should be proof tested in accordance with the test procedures detailed in Appendix 10A.

Existing headed wall anchors should be tested if there is evidence of significant corrosion or if anchor capacities greater than the default values detailed in Section 10.8.4 are required.

Existing wall-to-diaphragm anchor connections that rely on cross-grain bending of boundary joists should be reviewed. Cross-grain bending will occur in the boundary joist when face-loaded walls pull away from supporting floor diaphragms for the case when wall anchor brackets are not provided (refer Figure 10.56). Timber has low cross-grain bending capacity and, in many instances, has been found to be inadequate to resist the necessary seismic loads in past earthquakes (ICBO, 2000). Capacity is greatly improved if the ribbon board or solid blocking is well-connected to the joists. Where the connection is to a boundary joist, presence of solid blocking between one or more pairs of joists should be checked, with adequate connection to the joists.

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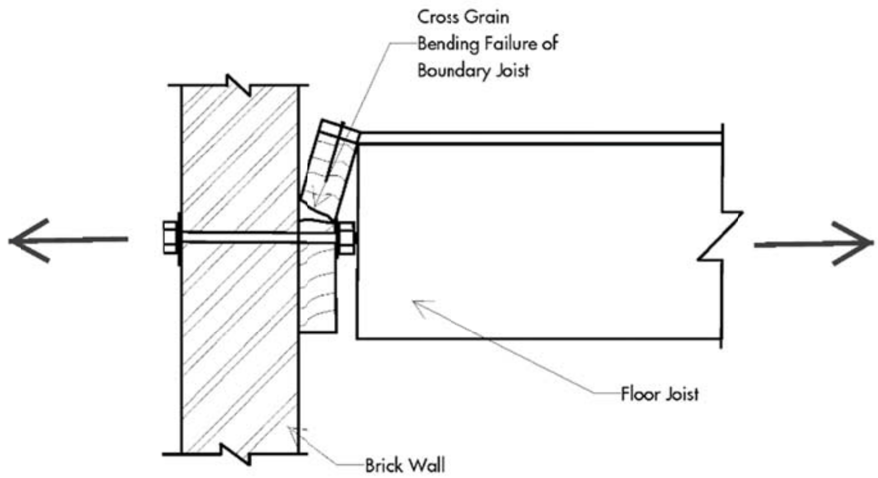

Figure 10.56: Out-of-plane loading cross grain bending failure mechanism (Oliver, 2010)

10.5.3.2 Diaphragm continuity

Detailing of existing strengthened diaphragms should be reviewed to ensure that reliable load paths exist to transfer the inertia loads from the face-loaded URM walls into the body of the diaphragm.

Existing nailed plywood sheathing joints should not be relied upon to transfer tension forces unless adequate detailing is provided at the joint locations (ICBO, 2000). The sub-diaphragm design methodology can be used to assess existing diaphragm strengthening continuity (Oliver, 2010), with checks then made to assess if those discontinuous diaphragms that arise when continuity is not realized or is lost can continue to fulfil the role of structural diaphragms, even if not originally intended to be discontinuous.

10.5.3.3 Deformation compatibility

Flexible lateral load resisting systems, such as structural steel or reinforced concrete moment resisting frames, have been used to strengthen URM buildings (Figure 10.26(a)).

When assessing the effect of strengthening measures such as this, deformation compatibility between the stiff URM structure and the more flexible lateral load resisting system needs to be considered.

An understanding of the non-linear strength-deformation relationship for each strengthening component will be required so that this can be compared with the relationships determined for the URM components and other structural systems that may be present.

Often it will not be possible to mobilise the full capacity of a flexible strengthening component before the deformation capacity of the URM is exceeded. An option available if this found will be to delete the URM from the primary seismic resisting system (assuming there is confidence that a life safety issue does not arise from the failure of the masonry) and reassess the capacity.

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10.5.4 Assessment of row buildings

Row buildings are similar buildings arranged side by side with insufficient seismic gaps to their neighbours, often with common boundary (party) walls: i.e. there is interaction between the individual buildings during a seismic shaking such that they cannot be considered in isolation. Buildings interconnected across boundaries should be considered as one building for the purposes of assessment.

Note:

The guidance below has been inferred from observed building damage only.

The effect of seismic shaking on row buildings is complex but also one of the least researched topics, particularly for URM buildings. It requires a special study which is outside the scope of these guidelines.

The effects of seismic shaking due to a lack of seismic gap can be both favourable (for the building within the row) and unfavourable (for the buildings on the ends of the row). Both of these effects should be accounted for when assessing the building's overall seismic performance. The building or structure within a row could become an end building if adjacent buildings are demolished.

Favourable effects include the potential for the whole block of row buildings to act as one unit and share seismic loads, and buttressing of central buildings by adjacent buildings in a row or an isolated building.

Unfavourable effects include pounding (knee effect and impact) on vertical load-bearing elements; the loss of which could potentially lead to loss of the gravity load path.

Buildings at the ends of rows suffer from two significant additional effects. First, they can be subject to the inertia/pounding effects of not just the adjacent building but some accumulation of effects along the row. Second and more importantly, forces tend to be almost unidirectional, pushing the end buildings off the row. This ratcheting effect is particularly detrimental to masonry structures where strains/crack widths accumulate much more quickly than when elements are able to complete a full return cycle. Therefore, the standard procedures for the assessment of buildings at the ends of rows should be used with care and consideration for these effects.

Note:

These guidelines recommend that all row effects on a particular building from the overall structure are described as part of its analysis and the vulnerabilities recorded. A "building" may be being assessed as if it is on one title, but the building from a structural connectivity point of view may extend for the whole block. The connectivity of the parts should be brought to the Building Consent Authority's (BCA's) attention throughout the assessment or retrofit consent process. Strengthening one "building" as part of a row will reduce the hazard in that section, but the seismic capacity of the overall building may still remain low due to the capacities in the remainder of the structure. The legal and compliance effects of row buildings should be discussed and agreed with owners and BCAs as part of any assessment process.

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10.5.4.1 General performance

The performance of row buildings depends primarily on the alignment (or otherwise) of:

- floor diaphragms
- façades,
- primary transverse bracing elements, when situated against the boundary, and
- common walls

The extent of misalignment of floors increases the bending effect on structures that are common to both buildings. When the extent of misalignment is greater than the depth of the floor, shear failure can also be induced.

Often, even if floors are misaligned, the façades are in the same plane (this is common in URM buildings). As a large proportion of the mass of the building is in the façade, it will not participate in the pounding action between the misaligned floors.

The effect of pounding damage to masonry buildings is generally less than for a frame or rigid diaphragm building as it tends to be more localised. Because of the high stiffness and often low height of these buildings, the impact forces are high frequency and associated with small displacements, and therefore carry less energy. Façades and other walls in the same alignment pound in their strong direction. Pounding between parallel walls where the pounding energy is dispersed over a large area will have a smaller effect than localised punching.

In addition to the above, most URM buildings have timber floors which have little mass to cause pounding. Similarly, with flexible diaphragms the impact energy is absorbed over a larger displacement. However, it is important to consider that URM is a brittle material and is sensitive to impact. Therefore, you should assess if the damage caused is likely to lead to loss of significant vertical load-carrying elements.

10.5.4.2 Building interconnection

If row buildings are not tied together, their relative displacement should be assessed against the length of dependable seating of the floors, or roof elements on the common wall.

If they are tied, note that the performance of elements that provide tying between the buildings (and similarly retrofit ties) can be classified into three types: rigid, elastic unbonded, and ductile. Rigid and elastic unbonded elements transfer force without dissipation of energy. For elastic unbonded elements, if there is sufficient stretch to allow the relative movement of the two structures their different stiffnesses will interact and will interrupt each other’s resonances. Some force will also be lost through pounding as the elements return together. Where floors align, the ties may take the form of simple rods or beams. Where floors misalign, these rods/beams will be coupled to a vertical column element which will (elastically) transfer the floor force across the offset.

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10.6 On-site Investigations

10.6.1 General

You will need to conduct a detailed building inspection in order to assess existing building strength and before preparing any strengthening proposal.

Your on-site investigation should cover the whole building, paying particular attention to the rear of the building and any hidden areas. It should include, but not be limited to, the following aspects.

10.6.2 Form and configuration

- Verify or establish the form and configuration of the building and components, including load paths between components, elements, and systems. As URM buildings may have had many changes of occupancy, there may be significant differences between available documentation and the actual building. Record this if so.
- Note the number of storeys, building dimensions and year of construction. Your notes of building dimensions should include opening locations and their dimensions, and should identify any discontinuities in the structural system.
- Note the structural system and material description, including vertical lateral force-resisting system, basement and foundation system.
- Also note any architectural features that may affect earthquake performance, including unrestrained components/elements such as parapets or chimneys.
- Note adjacent buildings and any potential for pounding and falling hazards. (Also refer to Section 10.5.4 for specific implications for row buildings.)

10.6.3 Diaphragm and connections

- Note the diaphragm types. For timber diaphragms, investigate the timber type, joist and beam spacing, and their connections, membrane and cladding type.
- Note the presence of floor and roof diagonal bracing systems and the dimensions of these elements.
- Examine wall-diaphragm connections and anchorage types (mechanical, adhesive and plate) to identify details and condition. You may need to remove floor or ceiling tiles to investigate connections and anchorage types. Record the condition of these connections, any variation in connection types and other features such as any alterations or deterioration.

Note:

If adhesive anchors are used, these warrant careful investigation. In some cases, a visual inspection will not be sufficient and an on-site testing programme should be considered.

A dribble of epoxy on the wall can indicate that the anchor hole was filled properly. However, it may also indicate that there are voids between segments of adhesive along the length of the anchor; or that the anchor was inserted, taken out and reinserted.

- For pocket type connections, check if the joists/rafters/beams are tightly packed by masonry on both sides or if there is a gap on both sides of the joists/rafters/beams.

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- When inspecting the diaphragm, note the location and size of the penetration accommodating stair or elevator access. Studies have shown that when penetrations are less than 10% of the diaphragm area it is appropriate to reduce in-plane diaphragm stiffness and strength in proportion to the reduction in diaphragm area. However, for larger diaphragm penetrations a special study should be undertaken to establish their influence on diaphragm response.
- Note if the diaphragm has previously been re-nailed at every nail joint using modern nails placed by a nail gun or if it has been varnished.
- Your assessment should also consider the quality of the fixings from any sheathing to the supporting structure to transfer the loads and prevent buckling of the diaphragm. Plaster, especially if cementitious, will act to protect the fixings. However, rusting of nails and screws can cause splitting of timber which can drastically reduce the strength of a sarking board of the supporting framing. We encourage careful examination for rusting or signs of leaks, especially in roof cavities if these are accessible.

10.6.4 Load-bearing walls

- Record the walls' general condition including any deterioration of materials, damage from past earthquakes, or alterations and additions that could affect earthquake performance.
- For multi-wythe construction, record the number of wythes, the distance between wythes, placement of inter-wythe ties, and the condition and attachment of wythes. Note that cavity walls will appear thicker than the actual structural wall.
- Record the bond type of the masonry, including the presence and distribution of headers. If possible, confirm that the bond bricks (headers) are not fake and cover more than one wythe. Check if the collar joint is filled.
- Check any unusual characteristics, such as a mix of walling units or unusual crack patterns.
- Record the type and condition of the mortar and mortar joints (for example, any weathering, erosion or hardness of the mortar) and the condition of any pointing or repointing, including cracks and internal voids. It is important to establish the mortar strength relative to the bricks as stronger mortar can lead to a brittle mode of failure. Investigation of existing damage to masonry walls can reveal their relative strength. Damage to bricks indicates a stronger mortar and weaker brick.

Note:

Visual inspection and simple scratching of the bricks and mortar may be sufficient to investigate the quality of masonry constituents. To be fully effective, your visual inspection should include both faces of the masonry.

Note that the mortar used for pointing is usually far better than the actual main body of the mortar, so scrape the point to full depth so you can investigate this.

The extent of to which detailed testing of the materials should be considered will depend on the importance of the building and the likely sensitivity of the material properties to the assessment result.

- Check any damp areas and the rear part of the building to investigate the quality and deterioration of the masonry and its constituents.

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- Note any horizontal cracks in bed joints, vertical cracks in head joints and masonry units, or diagonal cracks near openings.
- Record the presence of bond beams and their locations, and covered walls. Signs of cracking or decay should be investigated and, where appropriate, include chemical testing. Refer to Section 7 for further information on concrete testing.
- Examine and record any rotting and insect infestation of timber. Investigate timber in contact with masonry, particularly in damp areas.
- Record the presence of any DPC layers.
- Identify any vertical components that are not straight. Bulging or undulations in walls should be observed. Note any separation of exterior wythes, out-of-plumb walls, and leaning parapets or chimneys. Check URM party walls and partitions and investigate whether these are tied to the structural system.
- If opening up is permitted, include areas with built-in timbers (described in Section 10.2.10) so allowance can be made during the analysis. This analysis should allow for the brick capacity only, with no beneficial support from the timber unless specific investigations can prove otherwise. Existing bowing of walls and a lack of vertical load path where timber plates have shrunk can severely reduce face load capacity.

10.6.5 Non load-bearing walls

- Record the material and construction details of the non-load bearing walls. These walls may stiffen the floor diaphragm and brace the main loading walls. Their weight could be a significant component in the total weight.
- Check any unusual wall plaster construction.

10.6.6 Concrete

- Take care when making assumptions relating to the concrete strength and detailing. Intrusive investigation is essential to understand the makeup of the original construction and its constituents properly if any greater than nominal forces are to be transferred.

10.6.7 Foundations

- Note the type, material and structure of the foundation system.
- Check if the bricks are in contact with the soil. Degradation can occur depending on the extent to which the bricks were fired when originally produced, and/or if the soil is damp.

10.6.8 Geotechnical and geological hazards

- Carefully investigate any foundation settlement or deterioration due to vegetation. In particular, check around drains and slopes.
- Note any geological site hazards such as susceptibility to liquefaction and conditions for slope failure and surface fault rupture. Look for past signs of ground movement.

10.6.9 Secondary elements

- Record the details of secondary elements such as parapets, ornamentation, gable walls, lift wells, heavy equipment, canopies and chimneys. Include details of their dimensions

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and location. Also check for the presence of capping stones or other ornamental features as these create additional mass and eccentricity.

- In particular, check if parapets are positioned off-centre to the wall beneath. Inspect parapets to estimate the location of the rocking pivot.

10.6.10 Seismic separation

Investigate seismic separation with adjacent buildings. (Note that an apparent presence of a structural separation is not necessarily an indication that pounding will not occur unless the entire length of the separation is clear of any obstructions between the two buildings (Cole et al., 2011).

10.6.11 Previous strengthening

Verify any strengthening systems that have been used against available drawings and documentation. Record any variations and deterioration observed. Check as-built accuracy and note the type of anchors used, their size and location. Use Table 10.2 to check for particular issues that can arise with different strengthening techniques: record any relevant observations. Also refer to Section 10.5.3 for additional considerations for strengthened buildings, including deformation compatibility between the original and installed strengthening elements.

Table 10.2: Historical techniques used for URM buildings and common features

Structural Mechanism	Technique	Comments/Issues
Chimneys	Internal post-tensioning	Requires well-mapped, understood and not degraded vertical load-path
	Internal steel tube reinforcement	Wrap-around/tie reinforcement to connect to tube important
	Concrete filling	Adds mass Adhesion to surrounding brick often insufficient to tie
	External strapping	Inward collapse needs to be checked, especially if mortar degraded on inside Geometry often means external frames step outward: changes in angle need full resolution not to apply stress concentrations to masonry
	External bracing	Raking braces should have all vertical components of load resolved at each end Compatibility of stiff braced chimney with a flexible diaphragm must be checked
	Removal and replacement with lightweight	Heritage and weathering implications
Parapets (durability and weathering of particular concern)	Vertical steel mullions	Robust attachment to upper levels of brick with little wall/weight above critical Weathering through roof
	Raking Braces	Robust attachment to upper levels of brick with little wall/weight above critical Interaction with roof modes can destabilise Vertical tie-down required to raking braces

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Structural Mechanism	Technique	Comments/Issues
	Steel capping spanning between abutting frames or walls	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Internal Post-tensioning	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	External post-tensioning	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Internal bonded reinforcement	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Near Surface Mounted (NSM) composite strips	Parapet responds differently to different directions of load UV degradation
Face-loaded walls	Vertical steel mullions (Figure 10.23)	Stiffness vs out-of-plane rocking/displacement capability important Regularity/robustness of attachment to wall is important
	Vertical timber mullions	Stiffness vs out-of-plane rocking/displacement capability important Regularity/robustness of attachment to wall is important
	Horizontal transoms spanning between abutting frames or walls	Stiffness and attachment requirements need to consider wall above which gives clamping action to masonry at level of attachment
	Internal post-tensioning	Durability Anchorage level and fixity Level of pre-stress to allow rocking without brittle crushing
	External post-tensioning	As above
	Internal bonded reinforcement	Maximum quantity to ensure ductile failure Anchorage beyond cracking points, and consider short un-bonded lengths
	Composite fibre overlay	Preparation to give planar surface very involved
	Near Surface Mounted (NSM) composite strips	Wall responds differently to different directions of load Bond important if in-plane capacity is not to be weakened
	Reinforced concrete overlay	Wall responds differently to different directions of load
	Reinforced cementitious overlay	Wall responds differently to different directions of load Ductility of reinforcement important for deflection capacity
	Grout saturation/injection	Elastic improvement only: more suitable for low seismic zones and very weak materials

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Structural Mechanism	Technique	Comments/Issues
Connection of walls to diaphragms	Steel angle with grouted bars (Figure 10.24(a))	Bar anchorage Diaphragm/bar eccentricity must be resolved
	Steel angle with bolts/external plate (Figure 10.24(b))	Diaphragm/bar eccentricity must be resolved
	Timber joist/ribbon plate with grouted bars	Bar anchorage Diaphragm/bolt eccentricity causes bending of timber across grain - a potential point of weakness
	Timber joist/ribbon plate with bolts/external plate	Diaphragm/bolt eccentricity causes bending of timber across grain - a potential point of weakness
	Blocking between joists notched into masonry	Joist weak axis bending must be checked Tightness of fit of joists into pockets Degradation of joists
	External pinning to timber beam end	Quality assurance/buildability of epoxy in timber Concentrated localised load Development in masonry (external plate preferred for high loads)
	External pinning to concrete beam or floor	Development in masonry (external plate preferred for high loads) Concrete floor type (hollow pots, clinker concrete)
	Through rods with external plates	Elastic elongation Concentrated localised load
	New isolated padstones	Tightness of fit Resolution of eccentricity between masonry bearing and diaphragm connection
	New bond beams	High degree of intervention
Diaphragm strengthening	Plywood overlay floor or roof sparking (Figure 10.25)	Flexibility Requires continuous chord members and primary resistance elements
	Plywood ceiling	As above, plus existing ceiling battening/fixings may not be robust or may be decayed
	Plywood/light gauge steel composite	Stiffer but less ductile than ply-only Eccentricities between thin plate and connections must be resolved
	Plasterboard ceiling	As ply ceiling but less ductile Prevention of future modification/removal
	Thin concrete overlay/topping	Thickness for adequate reinforcement Additional mass Ductility capacity of non-traditional reinforcement Buckling restraint/bond to existing structure

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Structural Mechanism	Technique	Comments/Issues
	Elastic cross bracing	Stiffness relative to wall out-of-plane capacity Edge distribution members and chords critical Concentration of loads at connections
	Semi-ductile cross bracing (e.g. Proving ring)	As elastic Energy absorption benefit not easily quantified without sophisticated analysis
	Replacement floor over/below with new diaphragm	Design as new structure
In-plane wall strengthening **New primary strengthening elements** (Figure 10.26)	Sprayed concrete overlay	Restraint to existing floor/ roof structure Out-of-plane capacity of wall Ductility capacity if used very dependent on aspect ratio Chords Foundation capacity needs to be checked (uplift/rocking)
	Internal vertical post-tensioning	Ensure pre-stress limited to ensure no brittle failure See out-of-plane issues also
	External vertical post-tensioning	Ensure pre-stress limited to ensure no brittle failure See out-of-plane issues also
	Internal horizontal reinforcement	Coring/drilling difficult Stressing horizontally requires good vertical (perpendicular) mortar placement and quality
	External horizontal post-tensioning	Stressing horizontally requires good vertical (perpendicular) mortar placement and quality
	Bed-joint reinforcement	Workmanship critical Low quantities of reinforcement only possible
	Composite reinforced concrete boundary or local reinforcement elements	Development at ends/nodes Bond to existing
	Composite FRP boundary or local reinforcement elements	As above plus stiffness compatibility with existing
	Nominally ductile concrete walls or punched wall/frame	High foundation loads result
	Nominally ductile reinforced concrete masonry walls	Stiffness compatibility considering geometry (including foundation movement) important
	Nominally ductile steel concentric or cross bracing	Stiffness compatibility assessment critical considering element flexibility, plan position and diaphragm stiffness Drag beams usually required
	Limited ductility steel moment Frame	Flexibility/stiffness compatibility very important
	Limited ductility concrete frame	Flexibility/stiffness compatibility important

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Structural Mechanism	Technique	Comments/Issues
	Limited ductility concrete walls	Assess effectiveness of ductility, including foundation movements Ensure compatibility with any elements cast against Drag beams often required
	Limited ductility timber walls	Flexibility/stiffness compatibility very important Drag beams often required
	Ductile EBF/K-frames	Element ductility demand vs building ductility assessment important Drag beams usually required
	Ductile concrete coupled or rocking walls	Element ductility demand vs building ductility assessment important Ensure compatibility with any elements cast against drag beams often required
	Tie to new adjacent (new) structure	Elastic elongation and robustness of ties to be considered Higher level of strengthening likely to be required
Reinforcement at wall intersections in plan	Removal and rebuilding of bricks with inter-bonding	Shear connection only with capacity reduced considering adhesion and tightness of fit Disturbance of bond to adjacent bricks
	Bed joint ties	Small reinforcement only practical but can be well distributed Care with resolving resultant thrust at any bends
	Drilled and grouted ties	Tension only: consider shear capacity Depth to develop capacity typically large Compatibility with face-load spanning of wall
	Metalwork reinforcing internal corner	Attachment to masonry Small end-distance in abutting wall can mean negligible tension capacity
	Grouting of crack	Shear friction only: tension mechanism also required Stabilises any dilation but does not allow recovery
Foundation strengthening	Mass underpinning	Creates hard point in softer/swellable soils Even support critical
	Grout injection	Creates hard point in softer/swellable soils Difficult to quantify accurately
	Concentric/balanced re-piling	Localised “needles” through walls must provide sufficient bearing for masonry
	Eccentric re-piling with foundation beams	Stiffness of found beams important to not rotate walls out-of-plane
	Mini piling/ground anchors	Cyclic bond less than static bond Testing – only static practical Vulnerable to bucking if liquefaction

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Structural Mechanism	Technique	Comments/Issues
	Pile type: vertical stiffness and pre-loading	Pre-loading dictates load position Pre-loading important if new foundations less stiff than existing Dynamic distribution between new and old likely different than static Effects of liquefaction must be considered: may create limiting upper bound to strengthening level
Façade wythe ties	Helical steel mechanical engagement – small diameter	Low tension capacity, especially if cracked
	Steel mechanical engagement – medium diameter	Some vierendeel action between wythes Durability
	Epoxied steel rods/gauze sleeve	Some vierendeel action between wythes
	Epoxied composite/non-metallic rods	Stiffness
	Brick header strengthening	Additional new headers still brittle; can become overstressed under thermal/seasonal or foundation loadings in combination
Canopies	Reinforce or recast existing hanger embedment	Degradation of steel Depth of embedment to ensure sufficient mass of bricks to prevent pull-out
	New steel/cast iron posts	Propping of canopy can mitigate hazard from masonry falling to pavement Props in addition to hangars are not so critical with regard to traffic damage
	New cantilevered beams	Co-ordination with clerestory/bressumer beam Backspan reaction on floor
	Deck reinforcement to mitigate overhead hazard	Sacrificial/crushable layer to mitigate pavement hazard
	Conversion to accessible balcony	Likely to achieves all of the above objectives for canopies and also has natural robustness as designed for additional live load. Hazard still exists for balcony occupants
Base isolation		A lack of sufficient gap around the building Vertically re-founding the building

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10.7 Material Properties and Weights

10.7.1 General

This section provides default probable material properties for clay brick masonry and other associated materials.

These values can be used for assessment of URM buildings in the absence of a comprehensive testing programme (refer to Appendix 10A for details). However, to arrive at any reliable judgement, some on-site testing such as scratching, etc. as discussed in this section is recommended.

Note:

Before proceeding to on-site intrusive testing, it is important to sensibly understand what information will be collected from any investigation, how that would be used and what value the information will add to the reliability of the assessment. Sensitivity analyses can be used to determine the influence of any material parameter on the assessment outcome and, therefore, and whether testing to refine that material parameter beyond the default values given in this section is warranted.

When assessing the material characteristics of the building, survey the entire building to ensure that the adopted material properties are representative. It may be appropriate to assign different material properties to different masonry walls depending on variations in age, weathered condition or other aspects.

10.7.2 Clay bricks and mortars

Recommended probable default material properties for clay bricks and lime/cement mortars, correlated against hardness, are given in Tables 10.3 and 10.4. The descriptions in these tables are based on the use of a simple scratch test but there are a variety of similar, simple on-site tests you can use.

To ensure that the test is representative of the structural capability of the materials, remove any weathered or remediated surface material prior to assessing the hardness characteristics. This requirement is particularly important for establishing mortar material properties where the surface mortar may be either weathered or previously remediated and may not be representative of the mortar at depth. One recommended technique to establish whether the mortar condition is uniform across the wall thickness is to drill into the mortar joint and inspect the condition of the extracted mortar dust as the drill bit progresses through the joint.

Table 10.3: Probable strength parameters for clay bricks (Almesfer et al, 2014)

Brick hardness	Brick description	Probable brick compressive strength, f'_b (MPa)	Probable brick tensile strength, f_{bt} (MPa)
Soft	Scratches with aluminium pick	14	1.7
Medium	Scratches with 10 cent copper coin	26	3.1
Hard	Does not scratch with above tools	35	4.2

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Table 10.4: Probable strength parameters for lime/cement mortar (Almesfer et al, 2014)

Mortar hardness	Mortar description	Probable mortar compressive strength, f'_j (MPa)	Probable Cohesion, c (MPa)	Probable coefficient of Friction, μ_f $^{\Psi}$
Very soft	Raked out by finger pressure	0-1	0.1	0.3
Soft	Scratches easily with finger nails	1-2	0.3	
Medium	Scratches with finger nails	2-5	0.5	0.6
Hard	Scratches using aluminium pick	To be established from testing	0.7	0.8
Very hard†	Does not scratch with above tools	To be established from testing		

Note:

† When very hard mortar is present it can be expected that walls subjected to in-plane loads and failing in diagonal shear will form diagonal cracks passing through the bricks rather than a stair-stepped crack pattern through the mortar head and bed joints. Such a failure mode is non-ductile. Very hard mortar typically contains cement.

Ψ Values higher than 0.6 may be considered with care/investigation depending upon the nature/roughness of the brick material and the thickness of the mortar with respect to the brick roughness.

Values for adhesion may be taken as half the cohesion values provided in Table 10.4.

In cases where the probable modulus of rupture of clay bricks cannot be established from testing, the following value may be used (Almesfer et al, 2014):

$$f'_r(\mathrm{MPa}) = 0.12 f'_b \qquad \ldots 10.1$$

10.7.3 Compressive strength of masonry

In cases where the compressive strength of masonry cannot be established from the testing of extracted masonry prisms, the probable masonry compressive strength, f'_m, can be established using Equation 10.2 (Lumantarna et al, 2014b). Table 10.5 presents probable compressive strength values of clay brick masonry based on this equation using the brick and mortar probable compressive strength values from Tables 10.3 and 10.4.

$$f'_m(\mathrm{MPa}) = \begin{cases} 0.75 f_b'^{0.75} \mathrm{x}\, f_j'^{0.3} & \text{for } f'_j \geq 1 \text{ MPa} \\ 0.75 f_b'^{0.75} & \text{for } f'_j < 1 \text{ MPa} \end{cases} \qquad \ldots 10.2$$

Table 10.5: Probable compressive strength of clay brick masonry

Mortar strength, f'_j (MPa)	Probable masonry compressive strength, f'_m (MPa)		
	Probable brick compressive strength, f'_b (MPa)		
	14	26	35
0	5.4	8.6	10.8
1	5.4	8.6	10.8
2	6.7	10.6	13.3
5	8.8	14.0	17.5
8	10.1	16.1	20.1

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10.7.4 Direct tensile strength of masonry

The direct tensile strength of masonry, including any cement rendering and plaster, should be assumed to be zero, except when the requirements given in Section 10.8.5.2 for elastic analysis are satisfied for vertical spanning face-loaded walls.

10.7.5 Diagonal tensile strength of masonry

Where specific material testing is not undertaken to determine probable masonry diagonal tension strength, this may be taken as:

$$f_{\mathrm{dt}}(\mathrm{MPa}) = 0.5c + f_{\mathrm{a}}\mu_{\mathrm{f}} \qquad \ldots 10.3$$

where:

c = masonry bed-joint cohesion
μ_{f} = masonry co-efficient of friction
f_{a} = axial compression stress due to gravity loads.

10.7.6 Modulus of elasticity and shear modulus of masonry

The masonry modulus of elasticity, E_{m}, can be calculated by using the masonry probable compressive strength in accordance with Equation 10.4 (Lumantarna et al, 2014b). Note that this value of modulus of elasticity has been established as a chord modulus of elasticity between $0.05f'_{\mathrm{m}}$ and $0.7f'_{\mathrm{m}}$ in order to represent the elastic stiffness appropriate up to maximum strength.

Young's modulus of clay brick masonry can be taken as:

$$E_{\mathrm{m}}(MPa) = 300f'_{\mathrm{m}} \qquad \ldots 10.4$$

Shear modulus of clay brick masonry can be taken as (ASCE 41-13):

$$G_{\mathrm{m}}(MPa) = 0.4\,E_{\mathrm{m}} \qquad \ldots 10.5$$

10.7.7 Timber diaphragm material properties

Refer to Section 11 for timber diaphragm material properties.

10.7.8 Material unit weights

You can use the unit weights in Table 10.6 as default values if you do not have more reliable measurements.

Table 10.6: Unit weights

Material	Unit weight (kN/m^3)
Brick masonry	18
Oamaru stone masonry	16
Timber	5-6

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10.8 Assessment of Component/Element Capacity

10.8.1 General

This section covers the assessment of the capacity of the various components and elements that make up a masonry building.

In the displacement based procedure for face-loaded walls that is presented, the assessment of the demand is an integral part of the procedure.

10.8.2 Strength reduction factors

The assessment procedures in these guidelines are based on probable strengths and, therefore, the strength reduction factor, ϕ, should be set equal to 1.0. The probable strength equations and recommended default probable capacities in this section assume ϕ equals 1.0.

10.8.3 Diaphragms

10.8.3.1 General

Diaphragms in URM buildings fulfil two principal functions. They provide support to the walls oriented perpendicular to the direction of loading and, if stiff enough, they also have the potential to allow shears to be transferred between walls in any level, to resist the storey shear and the torsion due to any plan eccentricities.

The relative lateral stiffness of the diaphragms to the walls providing lateral support is often quite low due to the high stiffness of the walls, particularly for diaphragms constructed of timber or steel bracing.

Flexibility in a diaphragm, if too high, can reduce its ability to provide adequate support to walls and thus affect the response of these walls, or render its ability to transfer storey shears to minimal levels, although this will not generally be an issue if recognised and appropriately allowed for in the global analysis of the building. Considering the effects of diaphragm flexibility is, therefore, essential for proper understanding of both in and out-of-plane response of the walls.

When assessing the capacity of diaphragms it is necessary to consider both their probable strength and deformation capacities.

The probable strength capacity should be determined in accordance with the requirements in these guidelines that relate to the particular construction material of the diaphragm.

The deformation capacity will be that for which the strength capacity can be sustained.

The deformation capacity is also limited to that which it is expected will result in detrimental behaviour of supported walls or of the building as a whole.

The diaphragm deformations should be included when determining the inter-storey deflections for checking overall building deformations against the NZS 1170.5 limit of 2.5%.

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In the sections below recommendations are provided for diaphragm deformation limits to ensure adequate support for face-loaded walls and flexible (timber) and rigid diaphragms. Rigid diaphragms would typically need to be constructed of concrete to achieve the necessary relative stiffness with the walls.

10.8.3.2 Diaphragm deformation limits to provide adequate support to face-loaded walls

In order to ensure that the face-loaded walls are adequately supported, the maximum diaphragm in-plane displacement measured with respect to the diaphragm support walls should not exceed 50% of the thickness of the supported (face-loaded) walls (Figure 10.57). For cavity construction with adequate cavity ties installed, the inner masonry wythe is usually the load-bearing wythe and this criterion will require the maximum acceptable diaphragm displacement to be limited to 50% of the thickness of the inner wythe.

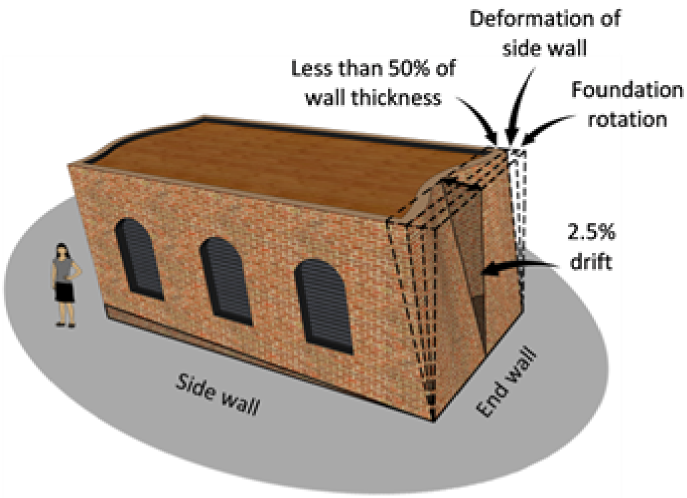

Figure 10.57: Mid-span diaphragm displacement limit for URM building on a flexible foundation

10.8.3.3 Timber diaphragms

General

Most URM buildings in New Zealand have flexible timber floor and ceiling diaphragms. Their in-plane deformation response is strongly influenced by the characteristics of the nail connections (Wilson et al., 2013a) and their global response is most adequately replicated as a shear beam (Wilson et al., 2013b). Responses can be separated into directions either parallel or perpendicular to the orientation of the joists (Wilson, et al., 2013c), refer Figure 10.58, and are significantly influenced by the presence of any floor or ceiling overlay, the degradation of the diaphragm due to aspects such as moisture or insect damage, and any prior remediation such as re-nailing or varnishing (Giongo, et al., 2013). If the diaphragms have had epoxy coatings that have penetrated into the joints between the flooring, this has been observed to result in substantial stiffening. Therefore, we recommend that you undertake a sensitivity analysis, recognising that the effective diaphragm stiffness could be more than given here by an order of magnitude or greater.

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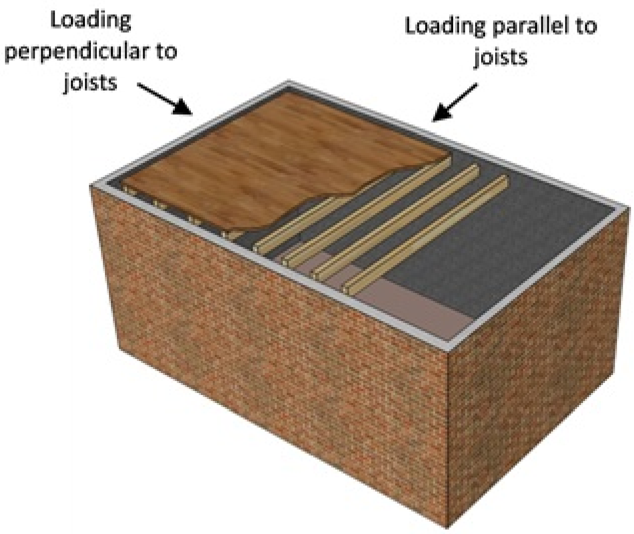

Figure 10.58: Orthogonal diaphragm response due to joist orientation

It is assumed here that the diaphragm is adequately secured to all perimeter walls via pocketing and/or anchorages to ensure that diaphragm deformation occurs rather than global sliding of the diaphragm on a ledge. It is also assumed that the URM boundary walls deform out-of-plane in collaboration with deformation of the flexible timber diaphragm. For non-rectangular diaphragms, use the mean dimensions of the two opposing edges of the diaphragm to establish the appropriate dimensions of an equivalent rectangular diaphragm.

Note:

Timber roofs of unreinforced masonry buildings were often built with both a roof and ceiling lining. As a result, roof diaphragms are likely to be significantly stiffer than the mid-height floor diaphragms if there are no ceilings on the mid-floors. Diagonal sarking in the roof diaphragm will also further increase its relative stiffness compared to the floor diaphragms.

If the diaphragm you are assessing has an overlay or underlay (e.g. of plywood or pressed metal sheeting), consult the stiffness and strength criteria for improved diaphragms. You will still need to consider stiffness and ductility compatibility between the two. For example, it is likely that a stiff, brittle timber lath-and-plaster ceiling will delaminate before any straight sarking in the roof above can be fully mobilised.

While the flooring, sarking and sheathing provide a shear load path across the diaphragm, it is necessary to consider the connections to the surrounding walls (refer to Section 10.8.4) and any drag or chord members. A solid URM wall may be able to act as a chord as it has sufficient in-plane capacity to transfer the chord loads directly to the ground. However, a punched URM wall with lintels only over the openings will have little tension capacity and may be the critical element in the assessment. Timber trusses and purlins, by their nature, only occur in finite lengths: their connections/splices designed for gravity loads may have little tie capacity.

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Probable strength capacity

The probable strength capacity of a timber diaphragm should be assessed in accordance with Section 11 of these guidelines.

Probable deformation capacity

Deformations in timber diaphragms should be assessed using the effective diaphragm stiffness defined below.

The probable deformation capacity should be taken as the lower of the following, assessed for each direction:

- L/33 for loading oriented perpendicular to the joists or L/53 for loading oriented parallel to the joists
- Deformation limit to provide adequate support to face-loaded walls. Refer Section 10.8.3.2.
- Deformation required to meet global inter-storey drift limit of 2.5% in accordance with NZS 1170.5. Refer Section 10.8.3.1.

Effective diaphragm stiffness

To determine the effective stiffness of a timber diaphragm, first assess the condition of the diaphragm using the information in Table 10.7.

Table 10.7: Diaphragm condition assessment criteria (Giongo et al., 2014)

Condition rating	Condition description
Poor	Considerable borer; floorboard separation greater than 3 mm; water damage evident; nail rust extensive; significant timber degradation surrounding nails; floorboard joist connection appears loose and able to wobble
Fair	Little or no borer; less than 3 mm of floorboard separation; little or no signs of past water damage; some nail rust but integrity still fair; floorboard-to-joist connection has some but little movement; small degree of timber wear surrounding nails
Good	Timber free of borer; little separation of floorboards; no signs of past water damage; little or no nail rust; floorboard-to-joist connection tight, coherent and unable to wobble

Next, select the diaphragm stiffness using Table 10.8 and accounting for both loading orientations.

Note:

While other diaphragm characteristics such as timber species, floor board width and thickness, and joist spacing and depth are known to influence diaphragm stiffness, their effects on stiffness can be neglected for the purposes of this assessment.

Pretesting has indicated that re-nailing vintage timber floors using modern nail guns can provide a 20% increase in stiffness.

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Table 10.8: Shear stiffness values† for straight sheathed vintage flexible timber floor diaphragms (Giongo et al., 2014)

Direction of loading	Joist continuity	Condition rating	Shear stiffness†, G_d (kN/m)
Parallel to joists	Continuous or discontinuous joists	Good	350
		Fair	285
		Poor	225
Perpendicular to joists††	Continuous joist, or discontinuous joist with reliable mechanical anchorage	Good	265
		Fair	215
		Poor	170
	Discontinuous joist without reliable mechanical anchorage	Good	210
		Fair	170
		Poor	135

Note:
† Values may be amplified by 20% when the diaphragm has been renailed using modern nails and nail guns
†† Values should be interpolated when there is mixed continuity of joists or to account for continuous sheathing at joist splice

For diaphragms constructed using other than straight sheathing, multiply the diaphragm stiffness by the values given in Table 10.9. If roof linings and ceiling linings are both assumed to be effective in providing stiffness, add their contributions.

Table 10.9: Stiffness multipliers for other forms of flexible timber diaphragms (derived from ASCE, 2013)

Type of diaphragm sheathing		Multipliers to account for other sheathing types
Single straight sheathing		x 1.0
Double straight sheathing	Chorded	x 7.5
	Unchorded	x 3.5
Single diagonal sheathing	Chorded	x 4.0
	Unchorded	x 2.0
Double diagonal sheathing or straight sheathing above diagonal sheathing	Chorded	x 9.0
	Unchorded	x 4.5

For typically-sized diaphragm penetrations (usually less than 10% of gross area) the reduced diaphragm shear stiffness, G'_d, is given by Equation 10.6:

$$G'_{\mathrm{d}}(kN/m) = \frac{A_{\mathrm{net}}}{A_{\mathrm{gross}}} G_{\mathrm{d}} \qquad \ldots 10.6$$

where A_{net} and A_{gross} refer to the net and the gross diaphragm plan area (in square metres).

For non-typical sizes of diaphragm penetration, a special study should be undertaken to determine the influence of diaphragm penetration on diaphragm stiffness and strength. The

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effective diaphragm stiffness must be modified further to account for stiffness of the URM boundary walls deforming in collaboration with the flexible timber diaphragm.

Hence:

$$G'_{\mathrm{d,eff}}(\mathrm{kN/m}) = \alpha_{\mathrm{w}} G'_{\mathrm{d}} \qquad \ldots 10.7$$

where α_w may be determined using any rational procedure to account for the stiffness and incompatibility of deformation modes arising from collaborative deformation of the URM walls displacing out-of-plane as fixed end flexure beams and the diaphragm deforming as a shear beam.

In lieu of a special study, prior elastic analysis has suggested that Equation 10.8 provides adequate values for α_w:

$$\alpha_{\mathrm{w}} \cong 1 + \left(\frac{t_\ell^3}{H_\ell^3} + \frac{t_{\mathrm{u}}^3}{H_{\mathrm{u}}^3}\right)\frac{L^2}{B}\frac{E_{\mathrm{m}}}{G'_{\mathrm{d}}} \qquad \ldots 10.8$$

where

t_ℓ = effective thickness of walls below the diaphragm, m
t_{u} = effective l thickness of walls above the diaphragm, m
H_ℓ = height of wall below diaphragm, m
H_{u} = height of wall above diaphragm, m
E_{m} = Young's modulus of masonry, MPa
B = depth of diaphragm, m.
L = span of diaphragm perpendicular to loading, m.

Refer to Figure 10.59 for definition of the above terms.

For scenarios where the URM end walls are likely to provide no supplementary stiffness to the diaphragm, α_w = 1.0 should be adopted.

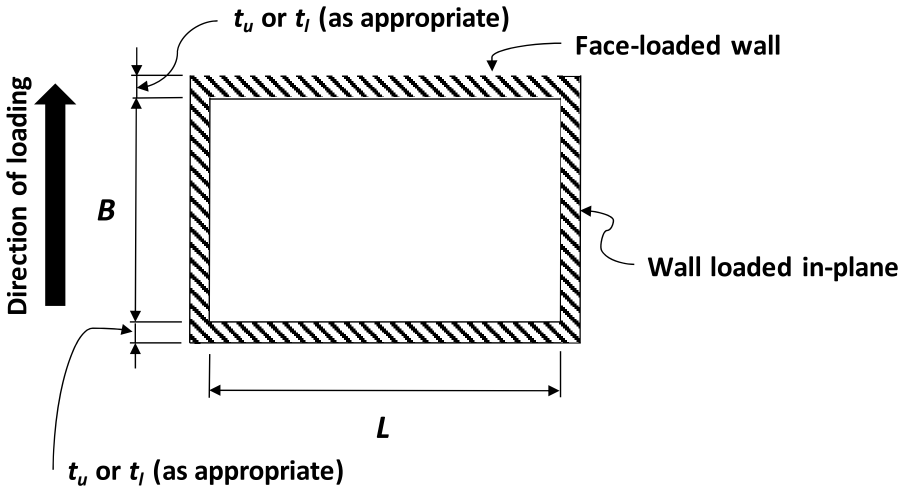

Figure 10.59: Schematics showing dimensions of diaphragm

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10.8.3.4 Rigid diaphragms

When assessing rigid diaphragms, you can use a “strut-and-tie” method. However, investigate the presence of termination details (hooks, thickenings, threads/nuts) carefully as their ability to transfer the loads at the strut-and-tie nodes is likely to govern the diaphragm capacity.

Rigid diaphragms can be assumed to have minimal effect on the response of out-of-plane walls.

10.8.4 Connections

10.8.4.1 General

The probable capacity of diaphragm to wall connections is taken as the lowest probable capacity of the failure modes listed below:

- punching shear failure of masonry
- yield or rupture of connector rod in tension or shear
- rupture at join between connector rod and joist plate
- splitting of joist or stringer
- failure of fixing at joist
- splitting or fracture of anchor plate
- yield or rupture at threaded nut.

Suggested default probable capacities for embedded and plate bearing anchors are provided below together. Guidance on specific assessment of capacities is also provided.

10.8.4.2 Embedded anchors

You can use the probable capacities provided in Tables 10.10 and 10.11 in lieu of specific testing provided that:

- The capacity should not be taken greater than the probable capacities of the anchor itself or the anchor to grout or grout to brick bond.
- When the embedment length is less than four bolt diameters or 50 mm, the pull-out strength should be taken as zero.
- The minimum edge distance to allow full shear strength to be assumed should be 12 diameters.
- Shear strength of anchors with edge distances equal to or less than 25 mm should be taken as zero.

Linear interpolation of shear strength for edge distances between these bounds is permitted (ASCE, 2013).

Simultaneous application of shear and tension loads need not be considered when using the values from Tables 10.10 and 10.11.

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Table 10.10: Default anchor probable shear strength capacities for anchors into masonry units only[1].

Anchorage type	Rod size	Probable shear strength capacity[2], (kN)
Bolts/steel rods fixed through and bearing against a timber member[1,2]	M12	8.5
	M16	15
	M20	18.5
Bolts/steel rods fixed through a steel member (washer) having a thickness of 6 mm or greater	M16	20

Note:
1. Anchors into mortar bed joints will have significantly lower shear capacities
2. Timber member to be at least 50 mm thick and MSG8 grade or better
3. For adhesive connectors embedment should be at least 200 mm into solid masonry

Table 10.11: Default anchor probable tension pull-out capacities for 0m, ≥0.3m and > 3m of wall above the embedment)

Mortar hardness	Single-wythe wall (kN)			Embedment 160 mm[1] into two-wythe wall (kN)			Embedment 250 mm[1] into three-wythe wall (kN)		
	0	≥0.3 m[(3)]	≥3 m	0	≥0.3 m[(3)]	≥3 m	0	≥0.3 m[(3)]	≥3 m
Very soft	0.3	0.5	1	1	1.5	4	1.5	3	8
Soft	1	1.5	3	2.5	4	9	5	8	18
Medium	1.5	2.5	6	4	6.5	15	8	14	31
Hard	2.5	3.5	8	6	9	21	11	19	43
Very hard	>2.5[(4)]	>4[(4)]	>8[(4)]	>6[(4)]	>10[(4)]	>21	>11[(4)]	>20[(4)]	>43[(4)]

Notes:
1. Representative value only: assumes drilling within 50 mm of far face of wall
2. Simultaneous application of tension and shear loading need not be considered
3. These values are intended to be used until there is ≥3 m of wall above the embedment.
4. Values for very hard mortar may be substantiated by calculation but can be assumed to be at least those shown.

The values in Table 10.11 are based on the pull-out of a region of brick, assuming cohesion or adhesion strength of the mortar on the faces of the bricks perpendicular to the application of the load factored by 0.5 and friction on the top and/or bottom faces (refer Figure 10.60), depending on the height of wall above the embedment as follows:

- 0 m (ie at the top of the wall) - adhesion only on the bottom and side faces
- ≥0.3 m but < 3 m – adhesion on the top, bottom and side faces, friction on the top and bottom faces
- ≥3 m – cohesion on the top, bottom and side faces, friction on the top and bottom faces.

A factor of 0.5 has been included in these values to reflect the general reliability of mechanisms involving cohesion/adhesion and friction.

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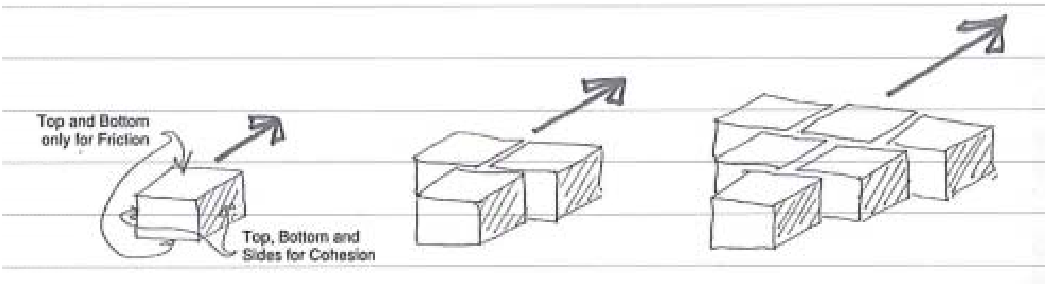

Figure 10.60: Basis for embedded anchor capacity estimation

The designer should select a bar diameter and tested epoxy system that will develop the required bond directly to the bricks and grout system as appropriate. Alternatively, cement mortars can be used but the capacity should be substantiated by site pull-out tests, using the grouting and cleanout methodology proposed by relevant standards/specifications.

For coarse thread screws, use the manufacturer's data for the direct bond to bricks, taking account of the brick compressive strength and ensuring that fixings are into whole bricks rather than mortar courses.

When assessing the capacity of straight or bent adhesive anchors, refer to the product specification and the methodology prescribed by the anchor manufacturer.

For inclined embedded anchors, the horizontal force capacity should be reduced to the horizontal vector component, and checks made for an adequate load-path for the vertical component. If the inclination is less than 22.5 degrees these effects can be considered insignificant and the full capacity of the anchor can be assumed.

10.8.4.3 Plate anchors

For plate anchors, postulate the potential failure surface to estimate its capacity.

A wall punching shear model is shown in Figure 10.61.

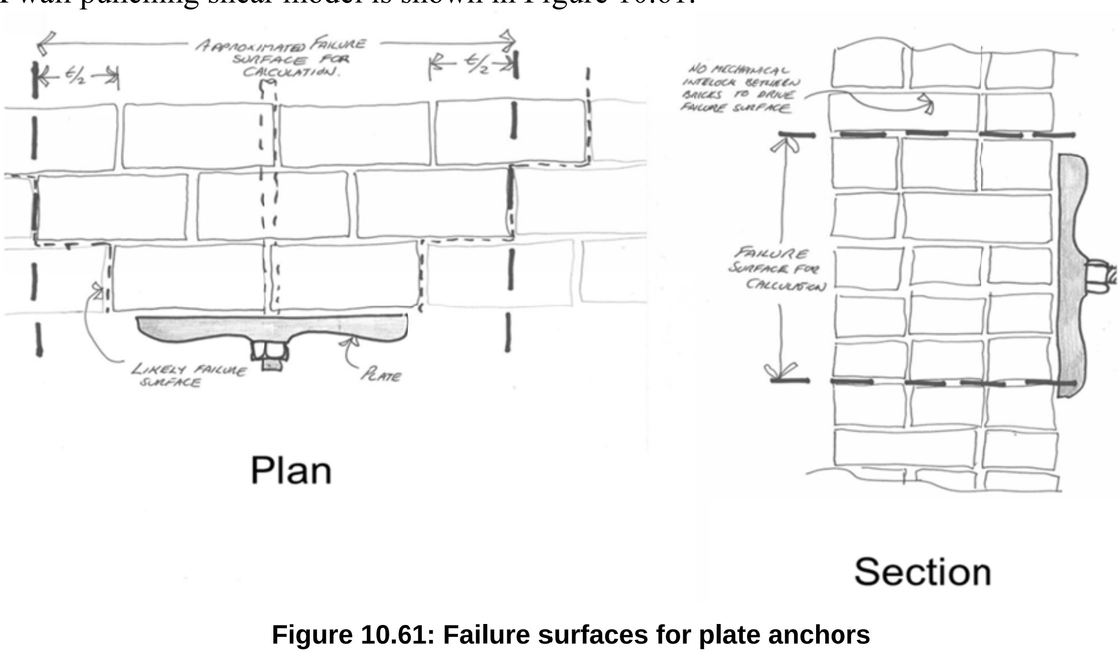

Figure 10.61: Failure surfaces for plate anchors

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10.8.4.4 Capacity of wall between connections

Where the lateral spacing of connections used to resist the wall anchorage force is greater than four times the wall thickness, measured along the length of the wall, check the section of wall spanning between the anchors to resist the local out-of-plane bending caused by the lateral force (FEMA, 2009). This check might be undertaken allowing for arching in the masonry; for example, through the compressive membrane forces that develop when a conical "yield line" pattern develops in the brick around the anchor.

For most applications involving bearing plates, it should be sufficiently accurate to assume a cylinder with a cross section the same shape as the bearing plate but lying outside it all round by half the thickness of the wall. Cohesion may be considered to be acting on the sides of this cylinder.

10.8.5 Wall elements under face load

10.8.5.1 General

This section provides both force-based (assuming elastic behaviour) and displacement-based inelastic methods for assessing face-loaded walls. The force-based methods utilising the direct tensile capacity of the masonry are only appropriate if all of the criteria listed in Section 10.8.5.2 – General are met.

Note:

The procedures in some earlier versions of this document (such as the 1995 "Red Book) that were based on the concept of equating total energy (strain energy of deformation plus potential energy due to shifts of weights) of the rocking wall to that for an elastic oscillator have since been shown to be deficient. These procedures give inconsistent results and are potentially unsafe; particularly where walls are physically hinged at floor levels (i.e. when they are supported on a torsionally flexible beam with no wall underneath) or made of stiff (high modulus of elasticity) masonry.

This update uses the same formulations as the 2006 guidelines but accommodates some of the more significant recent research findings. These are based on work carried out at the University of Auckland and University of Adelaide (Derakhshan et al 2013a, Derakhshan et al, 2013b, Derakhshan et al, 2014a and 2014b). However, we have not included all of the detailed procedures set out in this research (Derakhshan et al, 2014a) as there were some simplifying assumptions that made these procedures less suitable for thicker walls.

Procedures given for assessing face-loaded walls spanning one-way horizontally, or two-way horizontally and vertically, are based on response assuming only weak non-linear effects (i.e. assumption of elastic or nominally elastic response). These are based on less rigorous research and are not as well developed as procedures for walls spanning vertically. Caution is therefore required when using these recommendations.

Further research has been carried out in this area and we expect to include more comprehensive procedures in the next update.

For walls spanning vertically in one direction between a floor and another floor or the roof, or as vertically cantilevered (as in partitions and parapets), assure the lateral restraint of the floors and the roof for all such walls. If this restraint cannot be assured, the methods presented here for one-way vertically spanning walls cannot be used. However, it might

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still be possible to assess such walls by analysing them as spanning horizontally between other walls, columns or other elements, or as two-way assemblages.

Multi-wythe walls can be considered as one integral unit for face-loading if:

- all wythes are interconnected with header courses at least every fourth course and regularly along the length of the wall, or
- testing or special study has confirmed that the wythes are capable of acting as integral units.

Otherwise, consider each wythe as acting independently.

Header courses are typically provided every four to six courses in common bond. This would normally suffice for walls loaded out-of-plane (but note the caution raised above). These header courses would normally pass through the whole wall, with bricks lapping in the interior as required. For example, in triple brick walls the header course on the inside will be either one brick higher or lower than the header course on the outside to allow lapping over the central wythe.

If the above criterion is not met, investigate the sufficiency of the available header course by assuming a vertical shear acting on the centreline of the lower wall equal to $P + W_t + 0.5W_b$. This shear needs to be resisted by header bricks crossing the centreline. For this purpose, you can assume each header brick contributes a shear resistance of $2f_r bt^2/l$, where b, t and l are the breadth, depth and length of the header and f_r is its modulus of rupture of brick in bending.

If a wythe is not integral with the main structural wall, assume the wythe wall piggybacks the backing wall. If both wythes are one brick (110 mm) thick, you can assume they carry their own load independently for out-of-plane checks.

Non-structural masonry (usually single-wythe partitions, acoustic linings or fire linings) should be considered as a mass within the building and the risks for face-load collapse evaluated.

Internal walls with floors on both sides can be assumed to be supported at floor levels but checks on the diaphragms (strength and deformation) and perpendicular walls will still be required.

Walls should be assessed in every storey and for both directions of response (inwards and outwards). Set the rating of the wall at the least value found, as failure in any one storey for either direction of loading will lead to progressive failure of the whole wall.

10.8.5.2 Vertical spanning walls

General

When using an elastic analysis to determine the capacity of a wall section, ignore the direct tensile strength of the masonry unless:

- the demands are calculated assuming $\mu = 1$ and $S_p = 2$, and
- an inspection of the wall reveals no signs of cracking at that section, and
- the in-plane calculations indicate cracking of the brickwork is not expected.

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If you adopt a displacement based approach, the maximum out-of-plane displacement should be limited to 0.6 times the instability displacement for simply supported walls and 0.3 times the instability displacement for cantilever walls, e.g. parapets.

In the case of walls supported against face load, deflection of the supports will need to meet minimum requirements to ensure the walls can respond as assumed. In these guidelines, limits on the deflection of diaphragms are considered a diaphragm capacity issue and are defined in Section 10.8.3.2. These deflection limits should also apply to any other supports to face-loaded walls, for example, the support that may be provided by steel portal or steel bracing retrofits.

Elastic analysis

A simple bending analysis may be performed for the seismic assessment of face-loaded walls using Equation 10.9 provided that the criteria given in Section 10.8.5.2 - General are met. Equation 10.9 is applicable for a unit wall length.

$$M = \frac{t_{\mathrm{nom}}^2}{6}\left(f_{\mathrm{t}}' - \frac{P}{A_{\mathrm{n}}}\right) \quad \ldots 10.9$$

where:

P	=	Load applied to top of panel (N)
A_{n}	=	Net plan area of masonry (mm^2)
M	=	Moment capacity of the panel (Nmm)
t_{nom}	=	Nominal thickness of wall excluding pointing (mm)

$$t_{\mathrm{nom}} = t_{\mathrm{gross}} - np \quad \ldots 10.10$$

where:

p	=	depth of mortar recess (in mm) as shown in Figure 10.62
t_{gross}	=	overall thickness of wall (in mm)
n	=	number of recesses.

n =2 if recesses are provided on both sides; n=1 otherwise.
If the recess is less than 6 mm, it can be ignored.

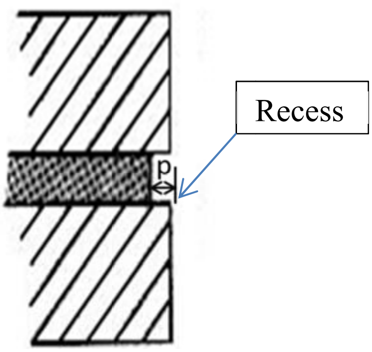

Figure 10.62: Pointing with recess

The imposed moment may be assumed critical at mid-height of walls restrained at the top and bottom, or critical at the base of cantilever walls.

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The direct tensile strength, f'_t, should be ignored in capacity calculations unless there is no sign of pre-cracking in the wall at the section being considered and the demand is assessed assuming fully elastic behaviour and taking S_p =2 (synonymous with applying a 0.5 factor to the capacity) and cracking of the brickwork in the region of the section is not expected for loading in-plane.

Inelastic displacement-based analysis for walls spanning vertically between supports

Follow the steps below to assess the displacement response capability and displacement demand in order to determine the adequacy of the walls.

Note:

Appendix 10B provides some guidance on methods for determining key parameters. Refer to Figure 10B.1 for the notation employed.

We have also provided some approximations you can use (listed after these steps) if wall panels are uniform within a storey (approximately rectangular in vertical and horizontal section and without openings).

Charts are provided in Appendix 10C that allow assessment of *%NBS* for regular walls (vertically spanning and vertical cantilever) in terms of height to thickness ratio of the wall, gravity load on the wall and parameters defining the demand on the wall.

The wall panel is assumed to form hinge lines at the points where effective horizontal restraint is assumed to be applied. The centre of compression on each of these hinge lines is assumed to form a pivot point. The height between these pivot points is the effective panel height h (in mm). At mid-height between these pivots, height $h/2$ from either, a third pivot point is assumed to form.

The recommended Steps for assessment of walls following the displacement-based method are discussed below:

Step1

Divide the wall panel into two parts: a top part bounded by the upper pivot and the mid height between the top and bottom pivots; and a bottom part bounded by the mid-height pivot and the bottom pivot.

Note:

This division into two parts is based on the assumption that a significant crack will form at the mid height of the wall, where an effective hinge will form. The two parts are then assumed to remain effectively rigid. While this assumption is not always correct, the errors introduced by the resulting approximations are not significant.

One example is that significant deformation occurs in the upper part of top-storey walls. In particular, where the tensile strength of the mortar is small the third hinge will not necessarily form at the mid height.

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Step 2

Calculate the weight of the wall parts: W_b (in N) of the bottom part and W_t (in N) of the top part, and the weight acting at the top of the storey, P (in N).

Note:

The weight of the wall should include any render and linings, but these should not be included in t_{nom} or t (in mm) unless the renderings are integral with the wall. The weight acting on the top of the wall should include all roofs, floors (including partitions and ceilings and the seismic live load) and other features that are tributary to the wall.

Step 3

From the nominal thickness of the wall, t_{nom}, calculate the effective thickness, t.

Note:

The effective thickness is the actual thickness minus the depth of the equivalent rectangular stress block. The reduction in thickness is intended to reflect that the walls will not rock about their edge but about the centre of the compressive stress block.

The depth of the equivalent rectangular stress block should be calculated with caution, as the depth determined for static loads may increase under earthquake excitation. Appendix 10B suggests a reasonable value based on experiments, $t = t_{nom}$ (0.975-0.025 P/W). The thickness calculated by this formula may be assumed to apply to any type of mortar, provided it is cohesive. For weaker (and softer) mortars, greater damping will compensate for any error in the calculated t.

Step 4

Assess the maximum distance, e_p, from the centroid of the top part of the wall to the line of action of P. Refer to Figure 10B.1 for definition of e_b, e_t and e_o. Usually, the eccentricities e_b and e_p will each vary between 0 and $t/2$ (where t is the effective thickness of the wall). Exceptionally they may be negative, i.e. where P promotes instability due to its placement.

When considering the restraint available from walls on foundations assume the foundation is the same width as the wall and use the following values for e_b:

0	if the factor of safety for bearing under the foundation, for dead load only (FOS), is equal to 1
$t/3$	if FOS = 3 (commonly the case)
$t/4$	if FOS = 2

Note:

Figure 10B.2 shows the positive directions for the eccentricities for the assumed direction of rotation (angle A at the bottom of the wall is positive for anti-clockwise rotation).

The walls do not need to be rigidly attached or continuous with a very stiff section of wall beyond to qualify for an assumption of full flexural restraint.

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Care should be taken not to assign the full value of eccentricity at the bottom of the wall if the foundations are indifferent and may themselves rock at moments less than those causing rocking in the wall. In this case, the wall might be considered to extend down to the supporting soil where a cautious appraisal should then establish the eccentricity. The eccentricity is then related to the centroid of the lower block in the usual way.

Step 5

Calculate the mid-height deflection, Δ_i, that would cause instability under static conditions. The following formula may be used to calculate this deflection.

$$\Delta_\mathrm{i} = \frac{bh}{2a} \qquad \ldots 10.11$$

where:

$$b = W_\mathrm{b}e_\mathrm{b} + W_\mathrm{t}(e_\mathrm{o} + e_\mathrm{b} + e_\mathrm{t}) + P\left(e_\mathrm{o} + e_\mathrm{b} + e_\mathrm{t} + e_\mathrm{p}\right) - \Psi(W_\mathrm{b}y_\mathrm{b} + W_\mathrm{t}y_\mathrm{t}) \qquad \ldots 10.12$$

and:

$$a = W_\mathrm{b}y_\mathrm{b} + W_\mathrm{t}(h - y_\mathrm{t}) + Ph \qquad \ldots 10.13$$

Note:

The deflection that would cause instability in the walls is most directly determined from virtual work expressions, as noted in Appendix 10B.

Step 6

Assign the maximum usable deflection, Δ_m (in mm), as 0.6 Δ_i.

Note:

The lower value of the deflection for calculation of instability limits reflects that response predictions become difficult as the theoretical limit is approached. In particular, the response becomes overly dependent on the characteristics of the earthquake, and minor perturbances lead quickly to instability and collapse.

Step 7

Calculate the period of the wall, T_p, as four times the duration for the wall to return from a displaced position measured by Δ_t (in mm) to the vertical. The value of Δ_t is less than Δ_m. Research indicates that $\Delta_t = 0.6\Delta_m = 0.36\Delta_i$ for the calculation of an effective period for use in an analysis using a linear response spectrum provides a close approximation to the results of more detailed methods. The period may be calculated from the following equation:

$$T_\mathrm{p} = 4.07\sqrt{\frac{J}{a}} \qquad \ldots 10.14$$

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where J is the rotational inertia of the masses associated with W_b, W_t and P and any ancillary masses, and is given by the following equation:

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[e_b^2 + y_b^2] + W_t[(e_o + e_b + e_t)^2 + y_t^2] + P\left[(e_o + e_b + e_t + e_p)^2\right]\right\} + J_{anc} \quad \ldots 10.15$$

where J_{bo} and J_{to} are mass moment of inertia of the bottom and top parts about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but that contribute to the inertia.

When treating cavity walls, make the following provisions:

- When the veneer is much thinner than the main wythe, the veneer can be treated as an appendage. For inelastic analysis, the veneers can be accounted through J_{anc}.
- If both wythes are a one - brick (110 mm) thick, then these could be treated as independent walls. Allocate appropriate proportion of overburden on them and solve the problem in the usual way.
- Where an accurate solution is the objective, solve the general problem with the kinematic constraint that the two walls deflect the same.

Note:

The equations are derived in Appendix 10B. You can use the method in this appendix to assess less common configurations as necessary.

Step 8

Calculate the design response coefficient $C_p(T_p)$ in accordance with Section 8 NZS 1170.5 taking μ_p =1 and substituting $C(T_p)$:

$$C_i(T_p) = C_{hc}(T_p) \quad \ldots 10.16$$

where:

$C_{hc}(T_p)$ = the spectral shape factor ordinate, $C_h(T_p)$, from NZS 1170.5 for Ground Class C and period T_p, provided that, solely for the purpose of calculating $C_{hc}(T_p)$, T_p need not be taken less than 0.5 sec.

When calculating C_{Hi} from NZS 1170.5 for walls spanning vertically and held at the top, h_i should be taken as the average of the heights of the points of support (typically these will be at the heights of the diaphragms). In the case of vertical cantilevers, h_i should be measured to the point from which the wall is assumed to cantilever. If the wall is sitting on the ground and is laterally supported above, h_i may be taken as half of the height to the point of support.

If the wall is sitting on the ground and is not otherwise attached to the building it should be treated as an independent structure, not as a part. This will involve use of the appropriate ground spectrum for the site.

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Note:

The above substitution for $C_i(T_p)$ has been necessary because the use of the tri-linear function given in NZS 1170.5 (Equations 8.4(1), 8.4(2) and 8.4(3) does not allow appropriate conversion from force to displacement demands. The revised $C_i(T_p)$ converts to the following, with the numerical numbers available from NZS 1170.5 Table 3.1.

$$
\begin{aligned}
C_i(T_p) &= 2.0 && \text{for } T_p < 0.5\text{sec} \\
&= 2.0(0.5/T_p)^{0.75} && \text{for } 0.5 < T_p < 1.5\text{sec} \\
&= 1.32/T_p && \text{for } 1.5 < T_p < 3\text{sec} \\
&= 3.96/T_p^2 && \text{for } T_p > 3\text{sec}
\end{aligned}
$$

Only 5% damping should be applied. Experiments show that expected levels of damping from impact are not realised: the mating surfaces at hinge lines tend to simply fold onto each other rather than impact.

Step 9

Calculate γ, the participation factor for the rocking system. This factor may be taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \qquad \text{…10.17}$$

Note:

The participation factor relates the response deflection at the mid height of the wall to the response deflection for a simple oscillator of the same period and damping.

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ calculate the displacement response, D_{ph} (in mm) as:

$$D_{ph} = \gamma(T_p/2\pi)^2 C_p(T_p).R_p.g \qquad \text{...10.18}$$

where:

$C_p(T_p)$ = the design response coefficient for face-loaded walls (refer Step 8 above, and for more details refer to Section 10.10.3)

T_p = Period of face-loaded wall, sec

R_p = the part risk factor as given by Table 8.1, NZS 1170.5

$C_p(T_p)\ R_p \le 3.6$.

Note that with T_p expressed in seconds, the multiplied terms $(T_p/2\pi)^2 \times C_p(T_p) \times g$ may be closely approximated in metres by:

$$(T_p/2\pi)^2 \times C_p(T_p) \times g = \text{MIN}(T_p/3, 1) \qquad \text{…10.19}$$

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Step 11

Calculate

$$\%NBS = 100 \times \Delta_{\mathrm{m}} / D_{\mathrm{ph}} = 60(\Delta_{\mathrm{i}} / D_{\mathrm{ph}}) \qquad \ldots 10.20$$

Note:

The 0.6 factor applied to Δ_i reflects that response becomes very dependent on the characteristics of the earthquake for deflections larger than $0.6\Delta_i$.

The previous version of these guidelines allowed a 20% increase in %NBS calculated by the above expression. However that is not justified now that different displacements are used for capacity and for the period and the subsequent calculation of demand.

Note:

Steps 12 to 14 are only required for anchorage design.

Step 12

Calculate the horizontal accelerations that would just force the rocking mechanism to form. The acceleration may be assumed to be constant over the height of the panel, reflecting that it is associated more with acceleration imposed by the supports than with accelerations associated with the wall deflecting away from the line of the supports. Express the acceleration as a coefficient, C_{m}, by dividing by g.

Note:

Again, virtual work proves the most direct means for calculating the acceleration. Appendix 10B shows how and derives the following expression for C_m, in which the ancillary masses are assumed part of W_b and W_t.

$$C_m = \frac{b}{(W_b y_b + W_t y_t)} \qquad \ldots 10.21$$

Note:

To account for the initial enhancement of the capacity of the rocking mechanism due to tensile strength of mortar and possible rendering, we recommend that C_m be cautiously assessed when mortar and rendering are present or in the case of retrofit likely to be added. The value of C_{m} may also be too large to use for the design of connections. Accordingly, it is recommended that C_{m} need not be taken greater than the maximum part coefficient determined from Section 8 NZS 1170.5 setting R_{p} and μ_{p} =1.0.

Step 13

Calculate $C_{\mathrm{p}}(0.75)$, which is the value of $C_{\mathrm{p}}(T_{\mathrm{p}})$ for a part with a short period from NZS 1170.5 and define a seismic coefficient for the connections which is the lower of C_{m}, $C_{\mathrm{p}}(0.75)$ or 3.6

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Note:

$C_p(0.75)$ is the short period ordinate of the design response coefficient for parts from NZS 1170.5, and 3.6g is the maximum value of $C_p(T_p)$ required to be considered by NZS 1170.5 when R_p and μ_p =1.0 .

Step 14

Calculate the required support reactions using the contributing weight of the walls above and below the connection (for typical configurations this will be the sum of W_b and W_t for the walls above and below the support accordingly) and the seismic coefficient determined in Step 13.

Step 15

Calculate

$$\%NBS = \frac{\text{Capacity of connection from Section 10.8.4 x 100}}{\text{Required support reaction from Step 14}} \qquad \ldots 10.22$$

Note:

If supports to face-loaded walls are being retrofitted, we recommend that the support connections are made stronger than the wall(s) and not less than required using a seismic coefficient of $C_p(0.75)$, i.e. do not take advantage of a lower C_m value.

Simplifications for regular walls

You can use the following approximations if wall panels are uniform within a storey (approximately rectangular in vertical and horizontal section and without openings) and the inter-storey deflection does not exceed 1% of the storey height. The results are summarised in Table 10.12.

The steps below relate to the steps for the general procedure set out above.

Step 1 Divide the wall as before.

Step 2 Calculate the weight of the wall, W (in N), and the weight applied at the top of the storey, P (in N).

Step 3 Calculate the effective thickness as before, noting that it will be constant.

Step 4 Calculate the eccentricities, e_b, e_t and e_p. Each of these may usually be taken as either $t/2$ or 0.

Step 5 Calculate the instability deflection, Δ_i from the formulae in Table 10.12 for the particular case.

Step 6 Assign the maximum usable deflection, Δ_m, for capacity as 60% of the instability deflection.

Step 7 Calculate the period, which may be taken as $4.07\sqrt{(J/a)}$, where J and a are given in Table 10.12. Alternatively, where the wall is fairly thin (h/t is large), the period may be approximated as:

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$$T_p = \sqrt{\frac{0.28h}{(1+2P/W)}} \qquad \ldots 10.23$$

in which h is expressed in metres.

Step 8 Calculate $C_p(T_p)$ following Equation 10.16.

Step 9 Calculate the participation factor as for the general method, with the numerator of the expression expanded to give $\gamma = Wh^2/8J$. This may be taken at the maximum value of 1.5 or may be assessed by using the simplified expression for J shown in Table 10.12.

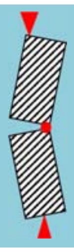
Step 10 Calculate D_{ph} from $C_p(T_p)$, T_p *and* γ in the same manner as for the general method.

Step 11 Calculate *%NBS* in the same manner as for the general method.

Note:

Charts are provided in Appendix 10C that allow the *%NBS* to be calculated directly for various boundary conditions for regular walls spanning vertically, given h/t_{Gross} for the wall, gravity load on the wall and factors defining the demand.

Table 10.12: Static instability deflection for uniform walls – various boundary conditions

Boundary Condition Number	**0**	**1**	**2**	**3**
				
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	t/2
b	$(W/2+P)$t	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)$h	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{(W/12)[h^2+16t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

Note:

1. The boundary conditions of the piers shown above are for clockwise potential rocking.
2. The top eccentricity, e_t, is not related to a boundary condition, so is not included in the table. The top eccentricity, e_t, is the horizontal distance from the central pivot point to the centre of mass of the top block which is not related to a boundary condition.
3. The eccentricities shown in the sketches is for the positive sense. Where the top eccentricity is in the other sense e_p should be entered as a negative number.

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Vertical cantilevers

Parameters for assessing vertical cantilevers, such as partitions and parapets are derived in Appendix 10B. Please consult this appendix for general cases.

For parapets of uniform rectangular cross-section, you may use the following approximations. These steps relate to the steps set out earlier for the general procedure for walls spanning between vertical diaphragms.

Step 1 You do not need to divide the parapet. Only one pivot is assumed to form: at the base.

Step 2 The weight of the parapet is W (in N). P (in N) is zero.

Step 3 The effective thickness is t (in mm) = $0.98t_{\text{nom}}$.

Step 4 Only e_b is relevant. It is equal to $t/2$.

Step 5 The instability deflection measured at the top of the parapet $\Delta_i = t$.

Step 6 The maximum usable deflection measured at the top of the parapet $\Delta_m = 0.3\Delta_i = 0.3t$.

Step 7 The period may be calculated from the assumption that $\Delta_t = 0.8\Delta_m = 0.24\Delta_i$.

$$T_p = \sqrt{0.65h\left[1 + \left(\frac{t}{h}\right)^2\right]} \qquad \ldots 10.24$$

in which h, the height of the parapet above the base pivot, and t, the thickness of the wall, are expressed in metres. The formulation is valid for $P = 0$, $e_b = t/2$, $y_b = h/2$ and approximating $t = t_{\text{nom}}$.

Step 8 Calculate $C_p(T_p)$ (refer to Step 8 of the general procedure for walls spanning vertically between diaphragms).

Step 9 Calculate $\gamma = 1.5/[1+(t/h)^2] \leq 1.5$. …10.25

Step 10 Calculate D_{ph} from $C_p(T_p)$, T_p and γ and as before.

Step 11 Calculate *%NBS* as for the general procedure for walls spanning between a floor and an upper floor or roof, from;

$$\%NBS = 100\,\Delta_m/D_{ph} = 30\,\Delta_i/D_{ph} = 30\,t/D_{ph}. \qquad \ldots 10.26$$

Note:

Steps 12 to 14 are only required for anchorage design.

Step 12 Calculate $C_m = t/h$. …10.27

Step 13 Calculate $C_p(0.75)$ which is the value of $C_p(T_p)$ for a part with a short period from NZS 1170.5. and define a seismic coefficient for the connections which is the lower of C_m, $C_p(0.75)$ and 3.6.

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Step 14 Calculate the base shear from W, C_m and $C_p(0.75)$. This base shear adds to the reaction at the roof level restraint.

Note:

Charts are provided in Appendix 10C that allow the *%NBS* to be calculated directly for various boundary conditions for regular walls cantilevering vertically, given h/t_{Gross} for the wall, gravity load on the wall and factors defining the demand.

Gables

Figure 10.63(a) shows a gable that is:

- free along the vertical edge
- simply supported along the top edge (at roof level), and
- continuous at the bottom edge (ceiling or attic floor level).

This somewhat unusual case is useful in establishing parameters for more complex cases. The following parameters can be derived from this gable:

$$a = \frac{h}{6}(2W + 3P) \qquad \text{...10.28}$$

$$J = \frac{W}{24g}(32t^2 + h^2) + \frac{9Pt^2}{4g} \qquad \text{...10.29}$$

Note:

In the above equations, W and P are total weights, not weights per unit length. Also note that the participation factor now has a maximum value of 2.0 ($t << h$, $P = 0$).

These results can be used for the gable in Figure 10.63(b) to provide a cautious assessment that does not recognise all of the factors that could potentially enhance the performance of such gables, such as the beneficial effects of membrane action

Note:

There are several factors that enhance performance in gables like those shown in Figure 10.63(a), all of which relate to the occurrence of significant membrane action. Guidance on this aspect will be provided in future versions of this document when the necessary research (including testing) has been undertaken. (Please also refer to the following section on walls spanning horizontally and vertically.)

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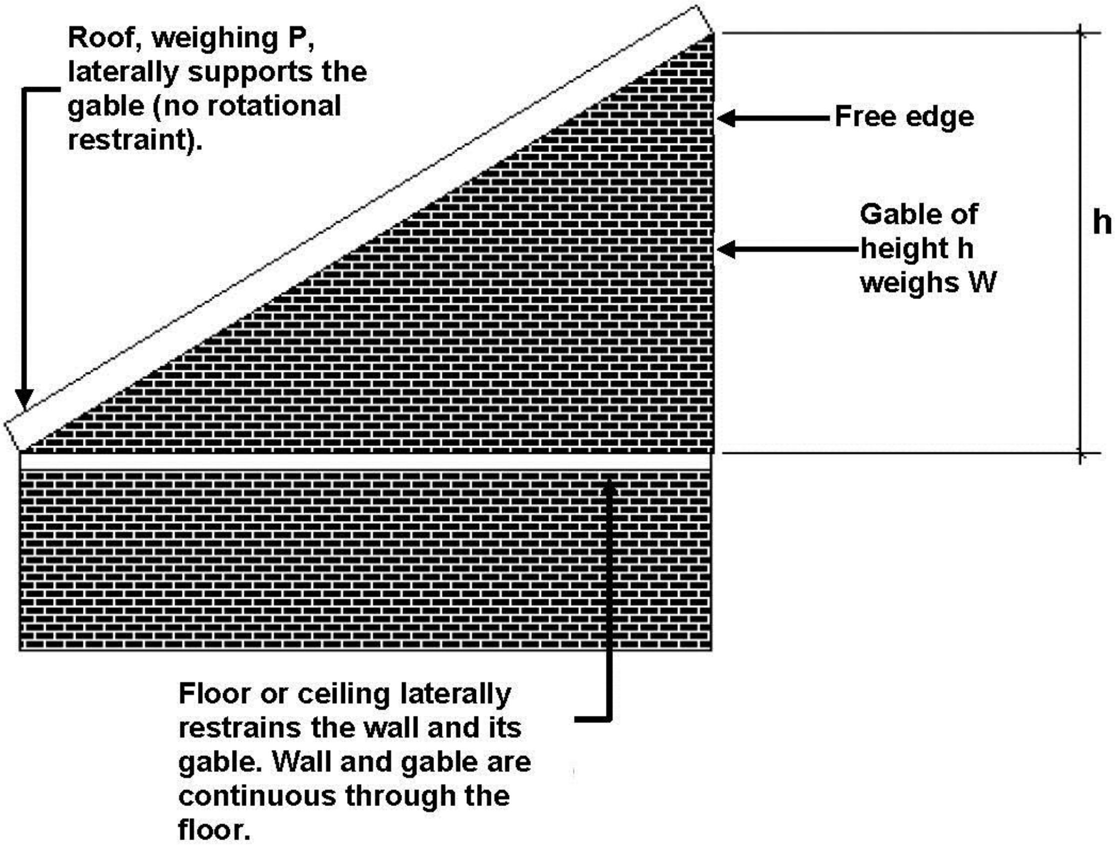
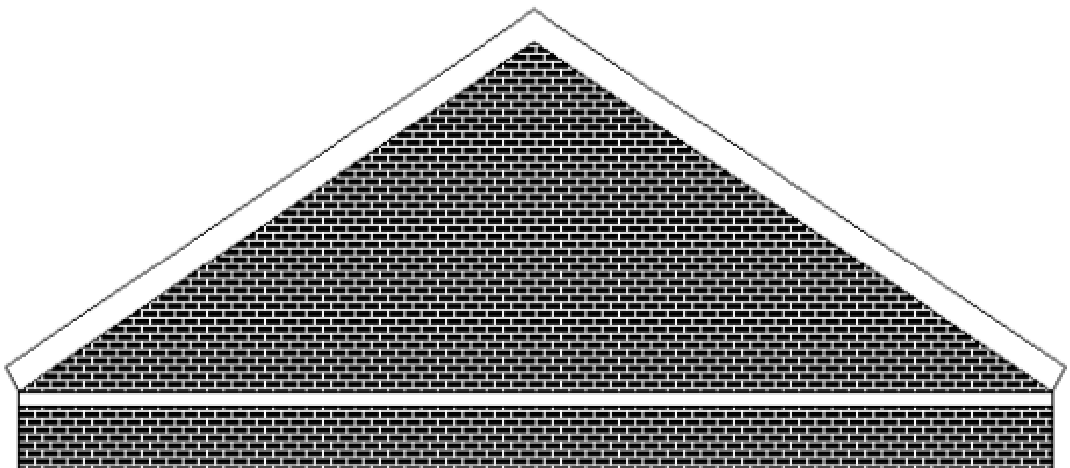


(a) Basic gable wall for defining parameters



(b) Typical gable for which results from (a) can be applied

Figure 10.63: Gable configurations

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10.8.5.3 Horizontal and vertical-horizontal spanning panels

Past earthquakes have shown that URM walls can act as a two-way spanning panel showing yield line patterns (Figure 10.64) similar to those that occur in a two-way spanning slab if the walls are attached to the supports on four sides. However, a special study is recommended if two-way spanning is to be assumed. This study should take into account different elastic properties, displacement compatibility, and any detrimental effects resulting from the expected behaviour of the wall in the orthogonal direction.

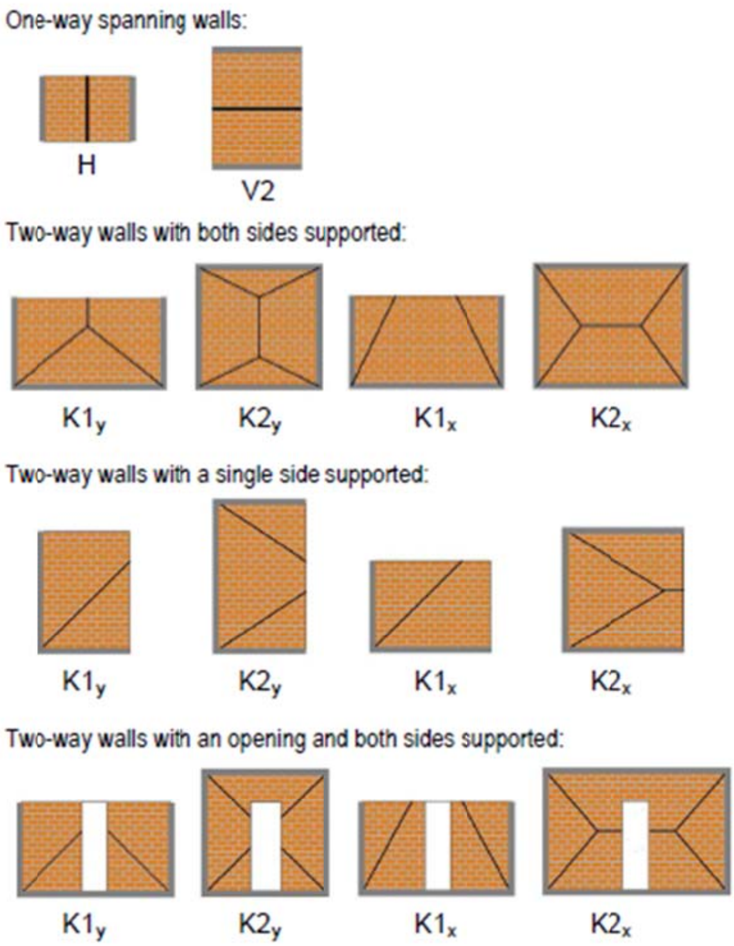

Figure 10.64: Idealised cracking patterns for masonry walls

Note:

Computationally intensive analytical methodologies such as finite element analysis have been shown to predict the out-of-plane strength of two-way spanning URM walls with good reliability. However, their reliance on knowing the precise values of material properties, the high computational effort and the high analytical skill required of the user makes them unsuitable for everyday design use.

The approach prescribed by the Australian masonry code AS 3700: 2011 for ultimate strength design of two-way spanning walls is the so-called virtual work method, developed by Lawrence and Marshall (1996). This is a form of rigid plastic analysis which assumes that, at the point of ultimate strength, the load resistance of the wall is obtained from contributions of moment capacities along vertical and diagonal crack lines in two-way bending mechanisms (Figure 10.64). Comparisons of strength predictions with a large experimental data set have been shown to be largely favourable in the sources mentioned

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before, in spite of numerous shortcomings of the moment capacity expressions used within the method which are still currently prescribed in AS 3700 (AS, 2011).

More recently, Willis et al. (2004) and Griffith et al. (2007) have developed alternative expressions for calculating the moment capacities which incorporate significant improvements over the AS 3700 expressions as they are based on more rational mechanical models, account for the beneficial effects of vertical compression, and are dimensionally consistent. Furthermore, Willis et al. (2004) demonstrated that the expressions perform favourably in predicting the ultimate load capacity when implemented into the virtual work approach.

The currently available research is not sufficient for assessing two-way panels in a typical design office environment. However, significant progress has been made into the behaviour of walls of this kind, e.g. Vaculik, (2012), and we expect to translate this into procedures suitable for design office use and routine assessment in time for the next update.

10.8.6 Walls under in-plane load

10.8.6.1 General

The capacity of wall components will typically be represented by the horizontal shear capacity of the component.

Wall components under in-plane load can be broadly categorised into two main groups: walls without penetration and walls with penetrations.

The capacity of wall components without penetrations should be assessed as outlined in Section 10.8.6.2.

The recommended approach to assessing the capacity of a wall component with penetrations is as follows:

Step 1: Divide the wall component into individual ‘elements’ comparing the ‘piers’ between the penetrations and ‘spandrel’ elements above and below the penetrations.

Step 2: Determine the capacity of the pier elements in a similar manner to walls in accordance with Section 10.8.6.2. This will require an assessment of the axial loads on the element due to gravity loads.

Step 3: Determine the capacity of the spandrel elements in accordance with Section 10.8.6.3. For basic buildings the spandrels may be treated as secondary elements and ignored in the assessment of lateral capacity.

Step 4: Determine if the capacity of the penetrated wall is governed by spandrel or pier capacity. This will need to be evaluated for each spandrel to pier connection. A sway index as defined in Section 10.8.6.4 can be used to do this.

Step 5: Based on the sway index determine if the capacity of each pier element is governed by the pier itself or the abutting spandrel element.

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Step 6: Carry out an analysis of the wall component to determine its capacity based on the capacity of the individual pier elements acting in series. Refer Section 10.8.6.4.

The degree to which a wall on a single line, but extending over several storeys, should be broken down into individual components will depend on the method of analysis used to establish the building's global capacity. This is discussed further in Section 10.9. Typically it is expected that it will be necessary to assess the capacity for each wall line between each storey in the building.

10.8.6.2 In-plane capacity of URM walls and pier elements

The in-plane strength capacity of URM walls and pier elements should be taken as the lower of the assessed diagonal tensile, toe crushing, in-plane rocking or bed joint sliding strength capacities as determined below. This then becomes the mode of behaviour and the basis for the calculation of the deformation capacity. Where DPC layers are present these may also limit the shear that can be resisted.

For the purposes of assessing the wall or pier capacities for each mechanism the yield displacement, Δ_y, may be taken as the sum of the flexural and shear in-plane displacements (making allowance for cracking etc as recommended in Section 10.7.6) when the element is subjected to a lateral shear consistent with achieving the shear strength for that mechanism as given below. Refer also Section 10.9.4.5.

Diagonal tensile capacity

This is one of the most important checks to be carried out.

The maximum diagonal tensile strength of a wall, pier or spandrel without flanges (or where you have decided to ignore them) can be calculated using Equation 10.30 (ASCE 41-13). Refer to the section below if you decide to account for the effect of flanges.

$$V_{\mathrm{dt}} = f_{\mathrm{dt}} A_{\mathrm{n}} \beta \sqrt{1 + \frac{f_{\mathrm{a}}}{f_{\mathrm{dt}}}} \qquad \ldots 10.30$$

where:

β = factor to correct nonlinear stress distribution (Table 10.13)
A_n = area of net mortared/grouted section of the wall web, mm^2
f_{dt} = masonry diagonal tension strength (Equation 10.3), MPa
f_a = axial compression stress due to gravity loads calculated at the base of the wall/pier, MPa.

Table 10.13: Shear stress factor, *β*, for Equation 10.30

Criterion	*β*
Slender piers, where $h_{eff}/l > 1.5$	0.67
Squat piers, where $h_{eff}/l < 0.5$	1.00
Note: Linear interpolation is permitted for intermediate values of h_{eff}/l	

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Refer to Figure 10.65 for the definition of h_{eff}.

This failure mode occurs when the diagonal tensile strength of a wall or pier is exceeded by the principal stresses. It is one of the undesirable failure modes as it causes a rapid degradation in strength and stiffness after the formation of cracking, ultimately leading to loss of load path. For this reason a deformation limit of Δ_y for this failure mode is recommended.

This failure mode is more common where axial stresses are high, piers are squatter and the tensile strength of masonry is low.

Diagonal tension failure leads to formation of an inclined diagonal crack that commonly follows the path of bed and head joints through the masonry, because of the lower strength of mortar compared to brick. However, cracking through brick is also possible if the mortar is stronger. In New Zealand masonry, the crack pattern typically follows the mortar joint.

For conditions where axial stresses on walls or piers are relatively low and the mortar strengths are also low compared to the splitting strengths of the masonry units, diagonal tension actions may be judged not to occur prior to bed-joint sliding. However, there is no available research to help determine a specific threshold of axial stress and relative brick and mortar strengths that differentiates whether cracking occurs through the units or through the mortar joints (ASCE, 2013).

Toe crushing capacity

The maximum toe crushing strength, V_{tc}, of a wall, pier or spandrel can be calculated using Equation 10.31 if no flanges are present or if you have decided to ignore them. If flanges are to be accounted for, refer to the section below.

$$V_{\mathrm{tc}} = (\alpha\, P + 0.5\, P_{\mathrm{w}}) \left(\frac{L_{\mathrm{w}}}{h_{\mathrm{eff}}}\right) \left(1 - \frac{f_{\mathrm{a}}}{0.7 f'_{\mathrm{m}}}\right) \qquad \ldots 10.31$$

where:

α	=	factor equal to 0.5 for fixed-free cantilever wall/pier, or equal to 1.0 for fixed-fixed wall/pier
P	=	superimposed and dead load at top of the wall/pier
P_w	=	self-weight of wall/pier
L_w	=	length of the wall/pier, mm
h_{eff}	=	height to resultant of seismic force (refer to Figure 10.65), mm
f_a	=	axial compression stress due to gravity loads at mid height of wall/pier, MPa
f'_m	=	masonry compression strength, MPa (refer to Section 10.7.3).

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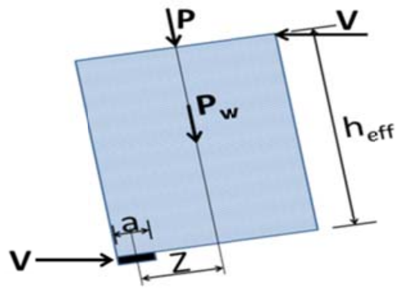

Figure 10.65: A rocking pier

A deformation limit of Δ_y or ϕ_y is recommended for this failure mode for walls/piers and spandrels respectively.

A toe crushing failure mode is not an expected failure mode of low-rise New Zealand walls or piers during in-plane loading. However, it still needs to be assessed; particularly when the walls have been retrofitted with un-bonded post-tensioning or a seismic intervention that inhibits the diagonal tension failure mode.

Rocking capacity

Rocking failure is one of the stable modes of failure. Experimental investigations undertaken by Knox (2012), Anthoine et al. (1995), Costley and Abrams (1996), Franklin et al. (2001), Magenes and Calvi (1995), Moon et al. (2006), Bruneau and Paquette (2004), Xu and Abrams (1992), and Bothara et al. (2010) have confirmed that URM elements exhibiting rocking behaviour have substantial deformation capacity past initial cracking but also exhibit very low levels of hysteretic damping.

A generalised relationship between strength and deformation for the rocking mechanism is shown in Figure 10.66.

The maximum probable rocking strength of a wall (considered over one level) or pier, V_r, can be calculated using Equation 10.32.

$$V_r = 0.9\,(\alpha P + 0.5 P_w)\frac{L_w}{h_{eff}} \qquad \ldots 10.32$$

where:

V_r	=	strength of wall or wall pier based on rocking
α	=	factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed-fixed wall pier.
P	=	superimposed and dead load at the top of the wall/pier under consideration
P_w	=	self-weight of the wall/pier
L_w	=	length of wall or wall/pier, mm
h_{eff}	=	height to resultant of seismic force (refer to Figure 10.65), mm.

When assessing the capacity of walls without openings for the full height of the building, Equation 10.32 will need to be adjusted to account for the different location of the lateral force. This can be assumed to be applied at two thirds of the height of the building from the point of fixity.

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Nonlinear response of rocking URM piers is generally characterised by a negative post-yield slope due to *P-Δ* effects but will be limited by toe crushing, as the effective bearing area at the toe of the rocking pier reduces to zero under increasing lateral displacement (refer Figure 10.66). This latent toe crushing differs from that discussed above as it typically occurs at larger rotations and lower shears.

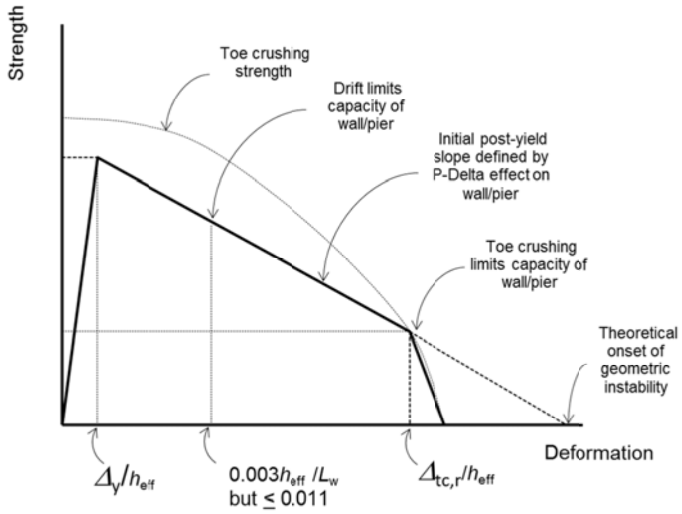

Figure 10.66: Generalised strength-deformation relationship for rocking of unreinforced masonry walls or piers (ASCE 41-13)

Deformation associated with the onset of toe crushing, $\Delta_{tc,r} / h_{eff}$, should be calculated using a moment-curvature or similar analytical approach and a maximum usable strain at the compression fibre of 0.0035. The axial compressive stress on the toe due to gravity loads should be based on an equivalent compression zone of the effective net section of the rocking pier that is in bearing.

Under rare conditions, the geometric stability of the rocking pier due to *P-Δ* effects may govern the ultimate deformation capacity. In the absence of substantiating test results, assume elastic unloading hysteretic characteristics for rocking URM in-plane walls and wall piers.

Note:

It is recommended that the capacity of a rocking wall/pier be limited to that consistent with a wall/pier lateral drift equal to the lower of $0.003h_{eff}/L_w$ or 0.011. The lateral performance of a rocking wall is considered to be less reliable and not to provide the level of resilience considered appropriate when the deflections exceed these values. Wall/pier elements that are not part of the seismic resisting system and which have a thickness greater than 350 mm (3 wythes), are expected be able to provide reliable vertical load carrying capacity at higher deflections approaching twice the limits given above. These greater limits can also be used for all wall/pier elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary.

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Assumption of fixity or cantilever action depends on the stiffness and overall integrity of the spandrels above and below the rocking pier and on how effectively spandrels can transmit vertical shears and bending. Conversely, wall spandrels that are weak relative to adjacent piers may not provide fixity at the tops and bottoms of piers and may result in piers acting as cantilevers. In general, deep spandrels could provide fixed-fixed boundary conditions.

Note that if the self-weight of the pier is large and boundary conditions are fixed-fixed, Equation 10.32 may overestimate the rocking capacity.

This behaviour mode is common where axial stresses are low, walls or piers are slender (height to length ratio > 2) and mortar strength are relatively better.

Bed-joint sliding shear capacity

Bed-joint sliding failure is one of the stable modes of failure. Investigations undertaken by various researchers have confirmed that URM elements exhibiting bed-joint sliding behaviour have substantial deformation capacity past initial cracking.

The recommended generalized force-deformation relationship for URM walls and wall piers governed by bed-joint sliding or sliding stair-stepped failure modes is illustrated in Figure 10.67. A simplified form of the ASCE 41-13 force-deformation relationship has been adopted.

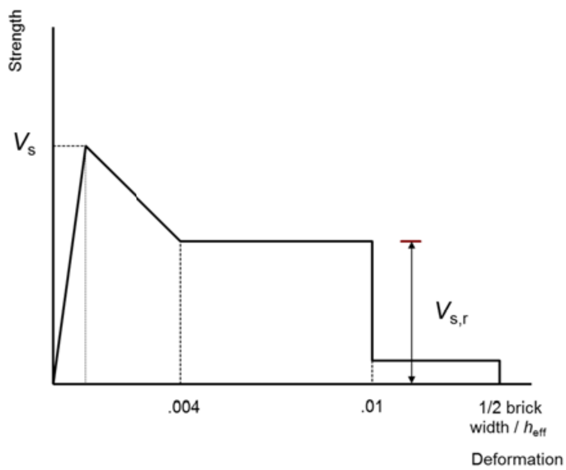

Figure 10.67: Generalised force-deformation relationship for unreinforced masonry walls or piers governed by bed-joint sliding or stair-stepped sliding

The maximum probable sliding shear strength, V_s, can be found from Equation 10.33.

$$V_s = 0.7(t_{nom}L_w c + \mu_f(P + P_w)) \quad \ldots 10.33$$

where:

μ_f	=	masonry coefficient of friction
P	=	superimposed and dead load at top of the wall/pier
P_w	=	self-weight of wall/pier above the sliding plane being considered

The 0.7 factor is to reflect the overall reliability of the sliding mechanism calculation.

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The capacity for bed-joint sliding in masonry elements is a function of bond and frictional resistance. Therefore, Equation 10.33 includes both factors. However, with increasing cracking, the bond component is progressively degraded until only the frictional component remains. The probable residual wall sliding shear capacity, $V_{s,r}$, is therefore found from Equation 10.33 setting the cohesion, c, equal to 0.

Note:

It is recommended that the bed joint sliding capacity of a rocking wall/pier be limited to a lateral drift of 0.003. The lateral performance of a wall/pier is considered to be unreliable and not able to provide the level of resilience considered appropriate when the deflections exceed this value. Wall/pier elements that are not part of the seismic resisting system are expected be able to provide reliable vertical load carrying capacity at higher drifts, approaching 0.075. These greater limits can also be used for all wall/pier elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary.

Slip plane sliding

A DPC layer, if present, will be a potential slip plane, which may limit the capacity of a wall.

The capacity of a slip plane for no slip can be found from Equation 10.34:

$$V_{\mathrm{dpc}} = \mu_{dpc}\,(P + P_w) \qquad \ldots 10.34$$

where:

μ_{dpc} = DPC coefficient of friction. Typical values are 0.2-0.5 for bituminous DPC, 0.4 for lead, and higher (most likely governed by the mortar itself) for slate DPC

Other terms are as previously defined.

Note:

Where sliding of a DPC layer is found to be critical, testing of the material in its current/in situ state may be warranted. Alternatively, parametric checks, where the effects of low/high friction values are assessed, may show that the DPC layer is not critical in the overall performance.

Sliding on a DPC slip plane does not necessarily define the deformation capacity of this behaviour mode.

Evaluating the extent of sliding may be calculated using the Newmark sliding block (Newmark, 1965) or other methods. However, exercise caution around the sensitivity to different types of shaking and degradation of the masonry above/below the sliding plane. Where sliding is used in the assessment to give a beneficial effect, this should be subject to peer review.

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Effect of wall and pier flanges

It is common practice to ignore the effects of flanges on the walls or piers while assessing the in-plane capacity of walls and piers. However, experimental research undertaken by Costley and Abrams (1996), Bruneau and Paquette (2004), Moon et al. (2006), Yi et al. (2008) and Russell & Ingham (2010) has shown that flanges have the potential to influence the response of in-plane walls. Flanged walls can have considerably higher strength and stiffness than those without flanges. The assessment could be particularly non-conservative where estimated rocking, sliding shear, or stair-step cracking strength (which are stable modes of failure), are close to the diagonal tensile strength of pier and walls. The recommended approach is to assess how much flange is required for diagonal tension to be the critical behaviour mode and based on this determine if further investigation is required.

Note:

One of the preconditions for taking into account the effect of the flanges is that they should remain integral with the in-plane piers and walls during the seismic shaking. Therefore, the integrity of the connections must be ascertained before ignoring or including them.

If flanges are taken into account, it is common to assume that the lengths of flanges acting in compression are the lesser of six times the thicknesses of the in-plane walls or the actual lengths of the flanges. It is also common to assume that equivalent lengths of tension flanges (to resist global or component overturning) are based on likely crack patterns relating to uplift in flange walls (Yi, et al., 2008). Other approaches that either model or consider different flange lengths qualitatively may result in a variety of crack patterns and corresponding sequences of actions.

10.8.6.3 URM spandrel capacity

General

The recommended generalized force-deformation relationship for URM spandrels is illustrated in Figure 10.68. The recommended generalized force-deformation relationship is based on experimental work undertaken by Beyer and Dazio (2012a and 2012b), Knox (2012) Graziotti et al (2012) and Graziotti et al (2014) and as recommended by Cattari et al (2014).

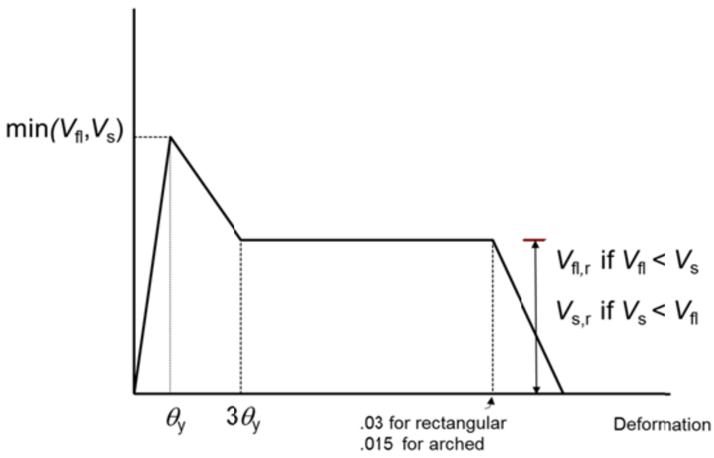

Figure 10.68: Generalized force-deformation relationship for unreinforced masonry spandrels

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Expected in-plane strength of URM spandrels should be the lesser of the flexural and shear strengths.

θ is the chord rotation of the spandrel, relative to the piers.

Note:

It is considered prudent to limit the deformation capacity of a spandrel panel to a panel drift of $3\theta_y$ if its capacity is to be relied on as part of the seismic resisting system. Panel chord rotation capacities beyond 0.02 or 0.01 for rectangular and arched spandrels respectively, for panels that are not assumed to be part of the lateral seismic resisting system, are not recommended as the performance of the spandrel (ie ability to remain in place) could become unreliable at rotations beyond these limits. These greater limits can also be used for all spandrel elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary and therefore the need to distinguish, in advance, between elements of the lateral and non-lateral load resisting systems is not required.

Two generic types of spandrel have been identified: rectangular and those with shallow arches. Recommendations for the various capacity parameters for these two cases are given in the following sections.

Investigations are continuing on appropriate parameters for deep arched spandrels. In the interim, until more specific guidance is available, it is recommended that deep arched spandrels be considered as equivalent rectangular spandrels with a depth that extends to one third of the depth of the arch below the arch apex.

The geometrical definitions used in the following sections are shown on Figure 10.69.

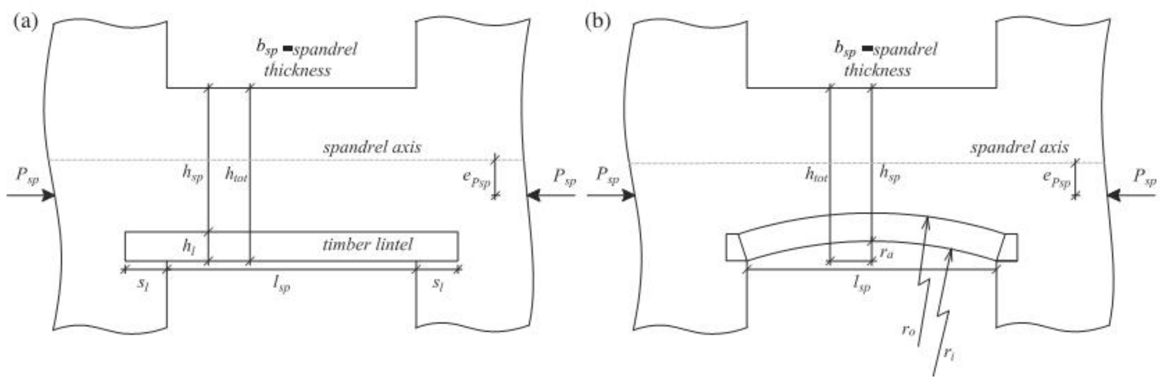

Figure 10.69: Geometry of spandrels with timber lintel (a) and shallow masonry arch (b) (Beyer, 2012)

Rectangular spandrels

The expected in-plane strength of URM spandrels with and without timber lintels can be determined following the procedures detailed below.

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Note:

There is limited experimental information on the performance of URM spandrels with lintels made from materials other than timber. However, URM spandrels with steel lintels are expected to perform in a similar manner to those with timber lintels.

When reinforced concrete lintels are present the capacity of the spandrel can be calculated neglecting the contribution of the URM.

Peak flexural strength

The peak flexural strength of rectangular spandrels can be estimated using Equation 10.35 (Beyer, 2012). Timber lintels do not make a significant contribution to the peak flexural capacity of the spandrels so can be ignored.

$$V_{\mathrm{fl}} = (f_{\mathrm{t}} + p_{\mathrm{sp}})\frac{h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{3l_{\mathrm{sp}}} \qquad \ldots 10.35$$

where:

f_t = equivalent tensile strength of masonry spandrel
p_{sp} = axial stress in the spandrel
h_{sp} = height of spandrel excluding depth of timber lintel if present
b_{sp} = width of spandrel
l_{sp} = clear length of spandrel between adjacent wall piers.

Unless the spandrel is prestressed the axial stress in the spandrel can be assumed to be negligible when determining the peak flexural capacity.

Equivalent tensile strength of masonry spandrel, f_t, can be estimated using Equation 10.36:

$$f_{\mathrm{t}} = 1.3(c + 0.5\mu_{\mathrm{f}} p_{\mathrm{p}}) + \frac{c}{2\mu_{\mathrm{f}}} \qquad \ldots 10.36$$

where:

p_p = mean axial stress due to superimposed and dead load in the adjacent wall piers
μ_f = masonry coefficient of friction
c = masonry bed-joint cohesion.

Residual flexural strength

Residual flexural strength of rectangular URM spandrels can be estimated using Equation 10.37 (Beyer, 2012). Timber lintels do not often make a significant contribution to the residual flexural capacity of URM spandrels so they can be ignored.

$$V_{\mathrm{fl,r}} = \frac{p_{\mathrm{sp}}\, h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{l_{\mathrm{sp}}}\left(1 - \frac{p_{\mathrm{sp}}}{0.85 f_{\mathrm{hm}}}\right) \qquad \ldots 10.37$$

where:

p_{sp} = axial stress in the spandrel
f_{hm} = compression strength of the masonry in the horizontal direction ($0.5f'_m$).

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Axial stresses are generated in spandrel elements due to the restraint of geometric elongation. Results from experimental research indicate that negligible geometric elongation can be expected when peak spandrel strengths are developed (Beyer, 2012; and Graziotti et al., 2012), as this is at relatively small spandrel rotations. As a result, there is little geometric elongation. Significant geometric elongation can occur once peak spandrel strengths have been exceeded, and significant spandrel cracking occurs within the spandrel, as higher rotations are sustained in the element. An upper bound estimate of the axial stress in a restrained spandrel, p_{sp}, can be determined using Equation 10.38 (Beyer, 2014):

$$p_{sp} = (1+\beta_s) f_{dt} \frac{l_{sp}}{2\sqrt{l_{sp}^2 + h_{sp}^2}} \qquad ...10.38$$

where:

f_{dt} = masonry diagonal tension strength

β_s = spandrel aspect ratio (l_{sp}/h_{sp}).

Equation 10.38 calculates the limiting axial stress generated in a spandrel associated with diagonal tension failure of the spandrel. The equation assumes the spandrel has sufficient axial restraint to resist the axial forces generated by geometric elongation.

In most typical situations you can assume that spandrels comprising the interior bays of multi-bay pierced URM walls will have sufficient axial restraint such that diagonal tension failure of the spandrels could occur.

Spandrels comprising the outer bays of multi-bay pierced URM walls typically have significantly lower levels of axial restraint. In this case the axial restraint may be insufficient to develop a diagonal tension failure in the spandrels. Sources of axial restraint that may be available include horizontal post-tensioning, diaphragm tie elements with sufficient anchorage into the outer pier, or substantial outer piers with sufficient strength and stiffness to resist the generated axial forces. For the latter to be effective the pier would need to have enough capacity to resist the applied loads as a cantilever.

It is anticipated that there will be negligible axial restraint in the outer bays of many typical unstrengthened URM buildings. In this case you can assume the axial stress in the spandrel is nil when calculating the residual flexural strength.

Peak shear strength

Peak shear strength of rectangular URM spandrels can be estimated using Equations 10.39 and 10.40 (Beyer, 2012). Timber lintels do not make a significant contribution to the peak shear capacity of URM spandrels so can be ignored.

$$V_{s1} = \frac{2}{3}\left(c + \mu_f p_{sp}\right) h_{sp} b_{sp} \qquad ...10.39$$

$$V_{s2} = \frac{f_{dt}}{2.3\left(1+\frac{l_{sp}}{2h_{sp}}\right)} \sqrt[2]{1+\frac{p_{sp}}{f_{dt}}}\, h_{sp} b_{sp} \qquad ...10.40$$

Unless the spandrel is prestressed you can assume the axial stress in the spandrel is negligible when determining the peak shear capacity. Equation 10.39 is the peak shear

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strength associated with the formation of cracks through head and bed joints over almost the entire height of the spandrel: use this equation when the mortar is weaker than the brick. If the mortar is stronger than the brick and fracture of the bricks will occur, use Equation 10.40.

Residual shear strength

Once shear cracking has occurred the URM spandrel can no longer transfer in-plane shear demands. When present, timber lintels acting as beams (simply supported at one end and fixed at the other) can transfer the vertical component of the spandrel load, *F*, to the adjacent pier (Figure 10.70).

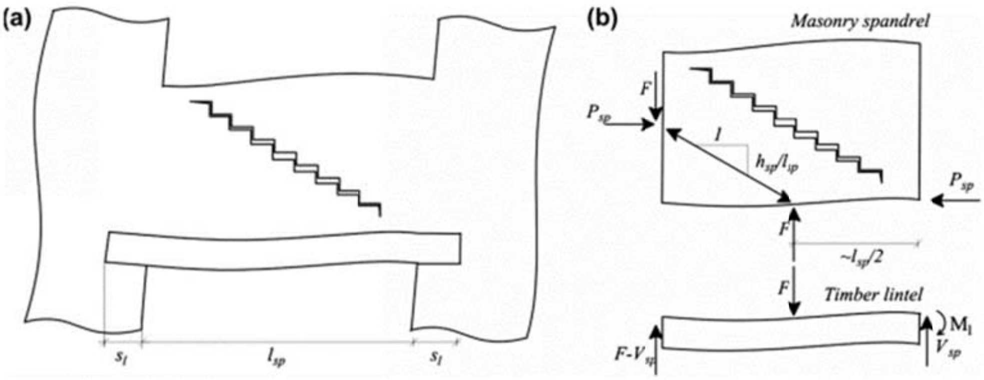

Figure 10.70: Shear mechanism of URM spandrels with timber lintels (Beyer, 2012)

Residual shear strength of cracked rectangular URM spandrels with timber lintels can be estimated as the minimum of Equation 10.41 or the capacity of the timber lintel to resist the applied load (Beyer, 2012). When no timber lintel is present the residual shear capacity of URM spandrels is negligible and can be assumed to be nil.

$$V_{\mathrm{s,r}} = \frac{11}{16} p_{\mathrm{sp}} \frac{h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{l_{\mathrm{sp}}} \qquad ...10.41$$

The applied load, *F*, to be resisted by the timber lintel can be calculated as:

$$F = p_{\mathrm{sp}} \frac{h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{l_{\mathrm{sp}}} \qquad ...10.42$$

You can calculate spandrel axial stresses, p_{sp}, in accordance with the procedures outlined above. Confirm the ability of the timber lintel to sustain the applied load.

Spandrels with a shallow arch

Peak flexural strength

You can estimate peak flexural capacity of a URM spandrel with a shallow arch using Equation 10.43 (Beyer 2012):

$$V_{\mathrm{fl}} = h_{\mathrm{sp}} b_{\mathrm{sp}} \left(f_{\mathrm{t}} \frac{h_{\mathrm{sp}}}{3 l_{\mathrm{sp}}} + p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}} \right) \qquad ...10.43$$

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where α_a is the arch half angle of embrace computed as:

$$\alpha_a = \tan^{-1}\left(\frac{l_{sp}}{2(r_i - r_a)}\right) \quad ...10.44$$

where dimensions r_i, r_a and l_{sp} are defined in Figure 10.69. The arch is considered shallow if the half angle of embrace, α_a, satisfies Equation 10.45 where r_o is also defined in Figure 10.69.

$$\cos \alpha_a \geq \frac{r_i}{r_o} \quad ...10.45$$

Unless the spandrel is prestressed you can assume the axial stress in the spandrel is negligible when determining the peak flexural capacity.

Residual flexural strength

You can estimate the residual flexural capacity of a URM spandrel with a shallow arch using Equation 10.46 (Beyer 2012) and by referring to Figure 10.69.

$$V_{fl,r} = \frac{p_{sp}\, h_{sp} h_{tot} b_{sp}}{l_{sp}}\left(1 - \frac{p_{sp}}{0.85 f_{hm}}\right) \quad \ldots 10.46$$

where dimension h_{tot} is defined in Figure 10.69. You can calculate spandrel axial stresses, p_{sp}, with the procedures set out in the previous section.

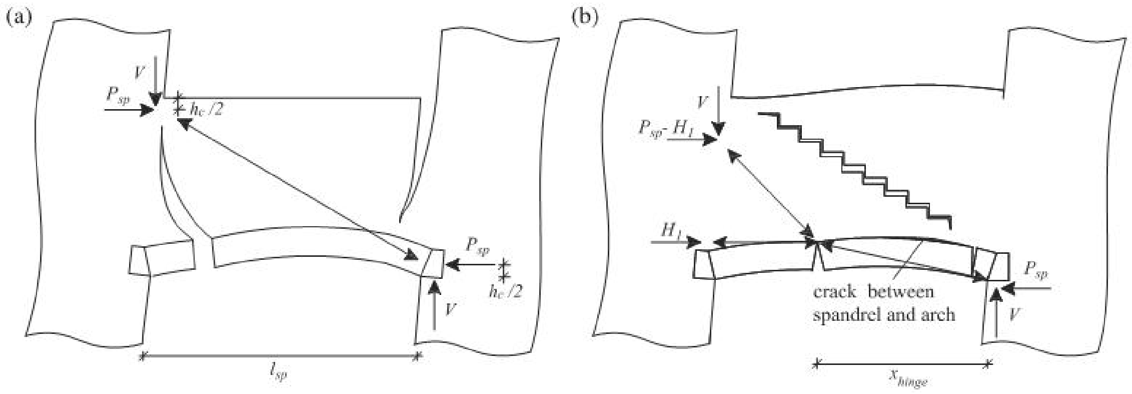

Figure 10.71: Spandrel with shallow arch. Assumed load transfer mechanism after flexural (a) and shear (b) cracking. (Beyer, 2012)

Peak shear strength

You can estimate peak shear strength of a URM spandrel with a shallow arch using Equations 10.47 and 10.48 (Beyer, 2012):

$$V_{s1} = h_{sp} b_{sp}\left[\frac{2}{3}\left(c + \mu_f p_{sp}\right) + p_{sp} \tan \alpha_a\right] \quad ...10.47$$

$$V_{s2} = h_{sp} b_{sp}\left(\frac{f_{dt}}{2.3\left(1+\frac{l_{sp}}{2h_{sp}}\right)}\sqrt{1 + \frac{p_{sp}}{f_{dt}}} + p_{sp} \tan \alpha_a\right) \quad ...10.48$$

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Unless the spandrel is prestressed you can assume the axial stress in the spandrel is negligible when determining the peak shear capacity. Equation 10.47 is the peak shear strength associated with the formation of cracks through head and bed joints over almost the entire height of the spandrel: it applies when the mortar is weaker than the brick. Use Equation 10.48 if the mortar is stronger than the brick and fracture of the bricks will occur.

Residual shear strength

Once shear cracking has occurred the URM spandrel itself can no longer transfer in-plane shear demands (Figure 10.71). The residual capacity of the lintel is therefore equivalent to the shear capacity of the arch which you can compute as follows (Beyer, 2012):

$$V_{\mathrm{s,r}} = h_{\mathrm{sp}} b_{\mathrm{sp}} p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}} \qquad \ldots 10.49$$

You can calculate spandrel axial stresses, p_{sp}, in accordance with the procedures provided in the previous section.

10.8.6.4 Analysis methods for penetrated walls

This section provides an overview of analysis methods that can be used to assess the capacity of a penetrated wall made up of components and of elements. Recommendations made regarding modelling assumptions for global analyses in Section 10.9.4 also apply to analyses of URM components.

Analysis of in-plane loaded URM walls and perforated walls can be carried out using the simplified "pier only" model shown in Figure 10.72 (Tomazevic, 1999). This analysis procedure assumes that the spandrels are infinitely stiff and strong, and therefore that the wall piers will govern the seismic response of the building. This simplified procedure may lead to non-conservative assessments for those structures which contain weak spandrels, or for structures assessed on the assumption that piers of dissimilar width rock simultaneously with shears calculated pro rata on the rocking resistance.

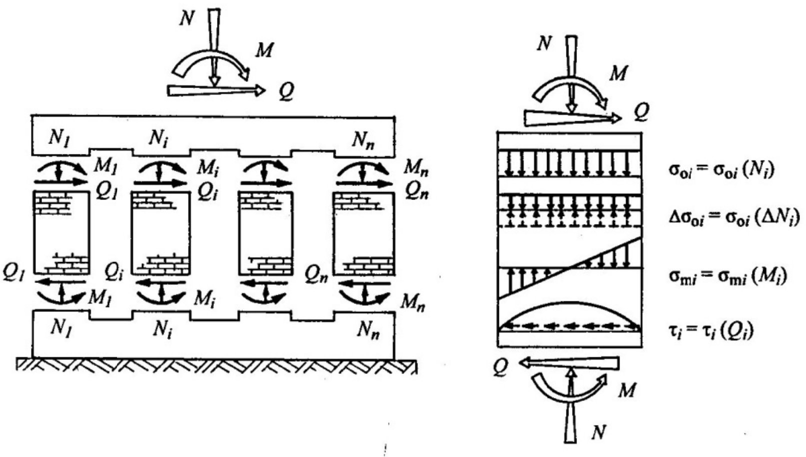

Figure 10.72: Forces and stresses in in-plane piers (Tomazevic, 1999)

Linear and nonlinear equivalent frame models as shown in Figure 10.73 (Magenes et al, 2006) can be used to analyse the in-plane response of perforated URM walls. Work by Knox has extended the equivalent frame model to include weak spandrel behaviour (Knox 2012).

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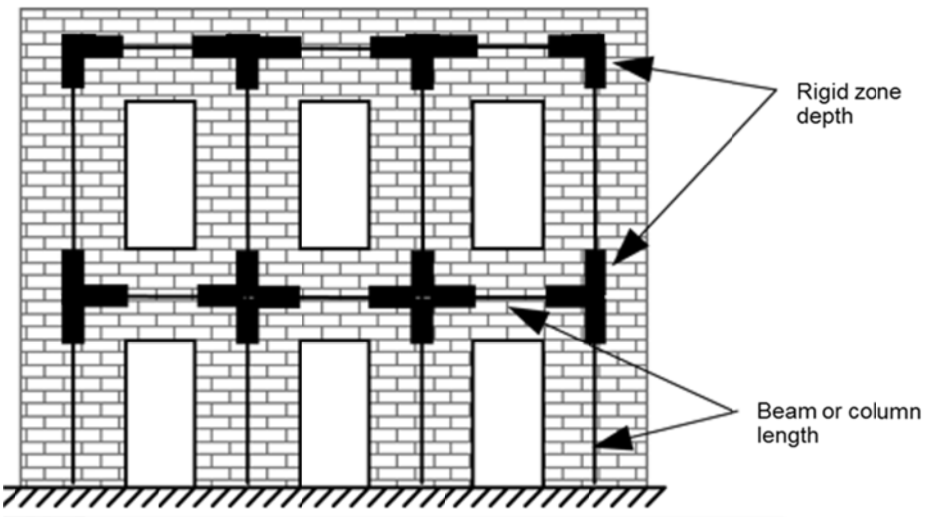

Figure 10.73: Equivalent frame

To investigate whether perforated wall behaviour is governed by spandrel or pier capacity a sway potential index, S_i, can be defined for each spandrel-pier joint by comparing the demand: capacity ratios for the piers and spandrels at each joint:

$$S_i = \frac{\frac{\Sigma V^*_{u,Pier}}{\Sigma V_{n,Pier}}}{\frac{\Sigma V^*_{u,Spandrel}}{\Sigma V_{n,Spandrel}}} \quad \ldots 10.50$$

where:

$\Sigma V^*_{u,Pier}$ = sum of the 100%*NBS* shear force demands on the piers above and below the joint calculated using K_R = 1.0

$\Sigma V_{n,Pier}$ = sum of the piers' capacities above and below the joint.

$\Sigma V^*_{u,Spandrel}$ = sum of the 100%*NBS* shear force demands on the spandrels to the left and right of the joint calculated using K_R = 1.0

$\Sigma V_{n,Pier}$ = sum of the spandrel capacities to the left and right of the joint.

When S_i > 1.0 a weak pier - strong spandrel mechanism may be expected to form and when S_i < 1.0 a strong pier - weak spandrel mechanism may be expected to form.

You can also carry out non-linear analysis of URM piers and spandrels using 2D plane stress elements or solid 3D elements. This method has the advantage that the stress and strains developed in the URM components can be assessed directly and deformation compatibility is maintained. Compression-only gap elements can be included in the analysis model to account for pier rocking (Knox, 2012).

For URM walls with openings of differing sizes and relatively weaker piers compared to stronger spandrels, Moon et al (2004) have recommended that the effective height of each rocking pier be represented as the height over which a diagonal compression strut is most

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likely to develop in the pier at the steepest possible angle that would offer the least lateral resistance (Figure 10.74). As a result, effective heights for some rocking piers adjacent to unequal size openings will vary depending upon the direction of loading. The angles to the piers generally depend on bed and head joint dimensions and stair-step cracking along mortar joints. If the diaphragms are rigid or reinforced concrete bands are provided, the effective height of the piers may be limited to the bottom of the diaphragm or the concrete band, as appropriate.

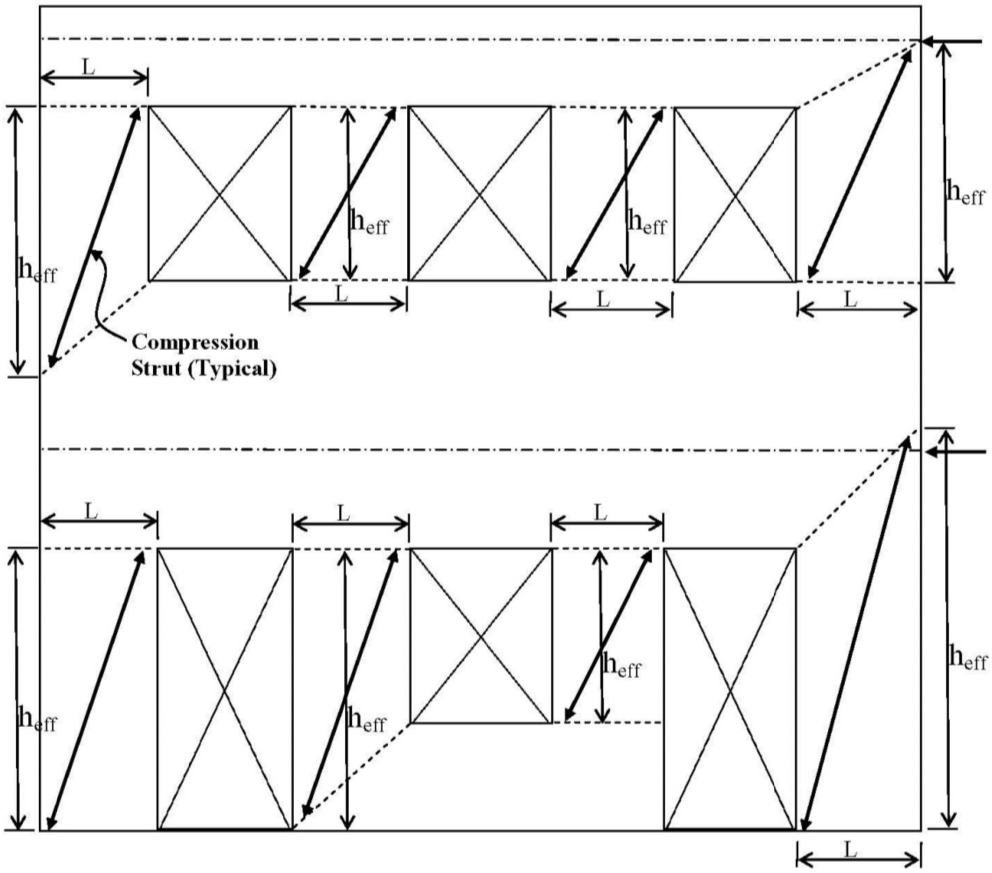

Figure 10.74: URM rocking pier effective heights based on development of diagonal compression struts that vary with direction of seismic force (ASCE 41-13).

The capacity of a penetrated wall component at particular level can also be determined from the capacity (strength and deformation) of the individual wall/pier elements assuming that displacement compatibility must be maintained along the component and using the force deformation relationships defined above for the governing mode of behaviour of each element. This can also be extended to multiple levels, if required, and the capacity of the whole wall determined if you have some knowledge of the lateral load distribution with height. This can be considered a variant of the SLAMa approach described elsewhere in these guidelines.

10.8.7 Other items of a secondary nature

Items of a secondary nature such as canopies and architectural features should be assessed for parts and components loads.

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10.9 Assessment of Global Capacity

10.9.1 General

The global capacity of the building is the strength and deformation capacity of the building taken as a whole, ignoring the performance of secondary elements. For this purpose face-loaded masonry walls are considered to be secondary elements unless the wall is providing primary support to the building or building part, e.g. by cantilever action of the wall. Diaphragms distributing lateral shears between lateral load resisting components (as opposed to providing support to face-loaded walls) are considered to be primary components and therefore the capacity of primary load paths through diaphragms and through connections from walls to diaphragms need to be considered when assessing the global capacity.

The global capacity of the building is likely to be significantly influenced by the relative in-plane stiffness of the diaphragms compared with the in-plane lateral stiffness of the masonry walls. Timber and cross-braced steel diaphragms will typically be "flexible" in this sense and this allows simplifications to be made in the assessment of global capacity as outlined below. Assuming high diaphragm stiffness where this is not assured can lead to erroneous assessment results, e.g. non-conservative assessments of diaphragm accelerations and inaccurate estimates of load distribution between lateral load resisting elements (Oliver, 2010). Flexible diaphragms can be explicitly modelled in 3D analysis models using linear or nonlinear 2D plane stress or shell elements, but care is required and the additional complexity will rarely be warranted for basic buildings. Well-proportioned concrete floor and roof slabs in small buildings may be assumed to be rigid.

Consideration of the non-linear capacity of masonry components is encouraged as it often leads to a higher global capacity than if the component capacities are limited to yield (elastic) levels. Consideration of non-linear behaviour requires a displacement based assessment approach. In many situations this is reasonably easy to implement and is recommended for the greater understanding of building seismic behaviour that it often provides.

When more than one lateral load mechanism is present, or when there are components of varying strengths and stiffness, a displacement based approach is considered essential to ensure displacement compatibility is achieved and the global capacity is not overstated. This is often the case for masonry buildings, particularly those that have been previously retrofitted with flexible and assumed ductile (low strength) systems.

When assessing the global capacity it will be necessary to complete an analysis of the building structure to assess the relationship between the individual component capacities and the global demands. Simple hand methods of analysis are encouraged in preference to overly sophisticated methods which may imply unrealistic transfers in shear between components that will be difficult to achieve in practice and may go unrecognised in the assessment. When sophisticated analyses are used, it is recommended that simpler methods are also used to provide order of magnitude verification.

The objective of global capacity assessment is to find the highest globally applied load/displacement that is consistent with reaching the strength/deformation capacity in the most critical component. The recommended approach for URM buildings is described in

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Figure 10.75. The global strength capacity can be referred to in terms of base shear capacity. The deformation capacity will be the lateral displacement at h_{eff} for the building consistent with the base shear capacity accounting for non-linear behaviour as appropriate.

This section provides guidance on the assessment of the global capacity for both basic and complex buildings. It also provides guidance on methods of analysis and modelling parameters.

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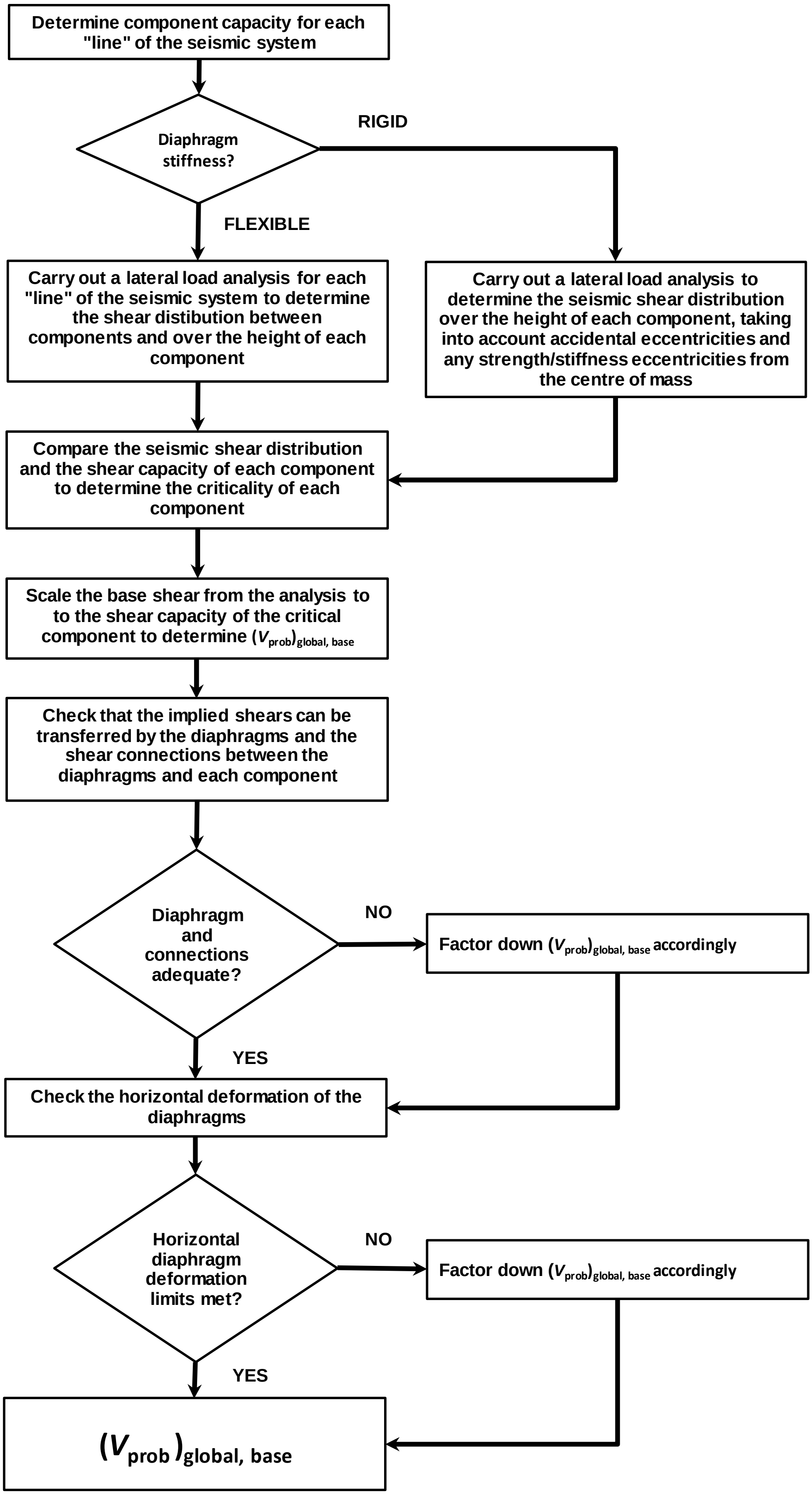

Figure 10.75: Global capacity assessment approach for URM buildings

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10.9.2 Global capacity of basic buildings

Determining the global capacity of basic URM buildings can be a simple exercise. Consider, for example, the single storey buildings shown in Figure 10.76. If the roof diaphragm is flexible the global capacity in each direction will be the lowest component capacity on any system line in that direction when there are only two system lines. When there are more than two system lines then the global capacity in a direction will be the capacity of the line in that direction which has the lowest value of V_{prob}/tributary mass, where V_{prob} in this context is the sum of the component probable capacities along the particular line of the seismic system.

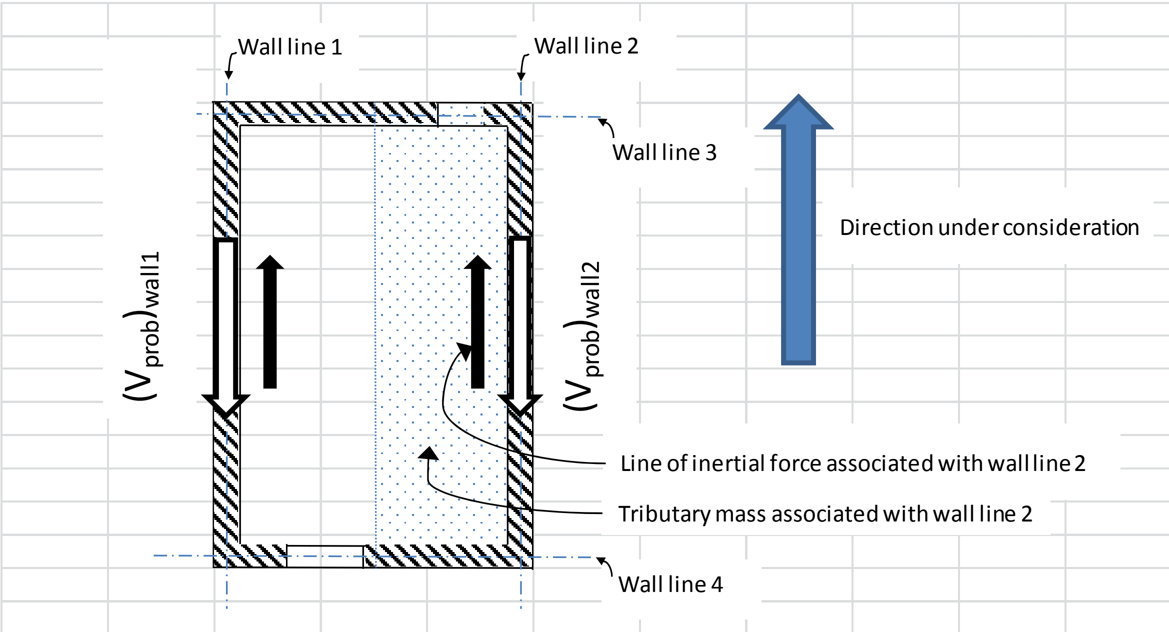

Figure 10.76: Relationship between demand and capacity for a basic building with a flexible diaphragm

For such buildings there would be little to gain from consideration of the non-linear behaviour of the components when determining the global capacity. An understanding of the non-linear capability, without jeopardising the vertical load carrying capacity, will, however, provide confidence that the building has resilience. If the demand is to be calculated in accordance with Section 10.10.2.2, non-linear behaviour is assumed if K_A is greater than 1.

Some small buildings with flexible diaphragms will not have identifiable or effective lateral load paths to provide lateral resistance to all parts of the building. An example of this is the open front commercial building where the sole means of lateral support might be cantilever action of the ends of the side walls, the capacity of which will be highly dependent on the restraint available from the wall foundation, and likely to be negligible.

Basic buildings of two or three stories with flexible diaphragms can be considered in a similar fashion, after first completing a simple analysis to determine the variation in shear over the height of each line of the seismic system. The global capacity of such buildings will be limited to the capacity of the line where $(V_{prob})_{line,i}/\beta_i$ is the lowest. $(V_{prob})_{line,i}$ is the sum of the component capacities along a line of the seismic system at level i and β_i is the

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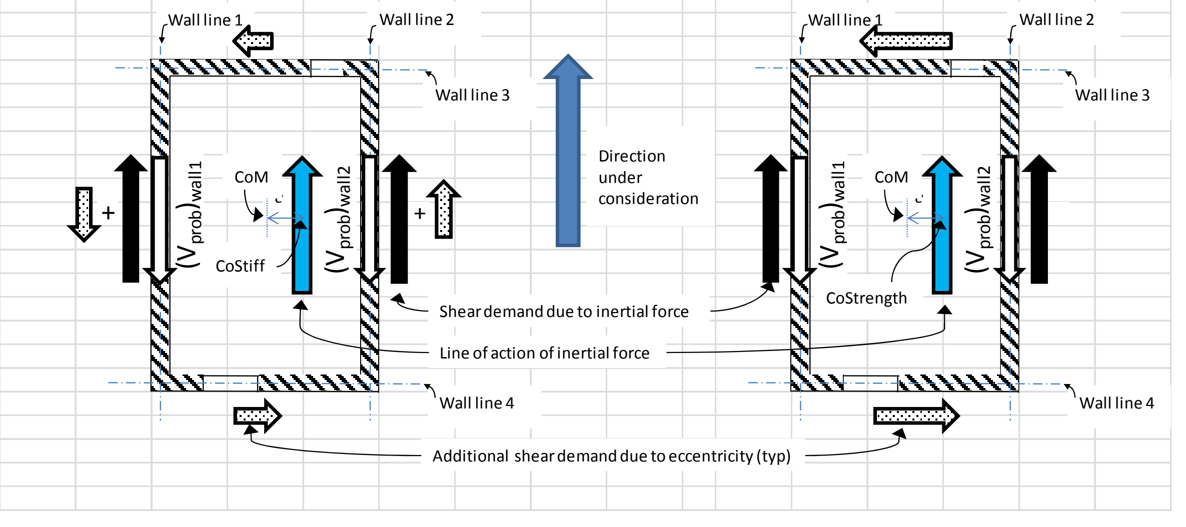
ratio of the applied shear at level i to the shear at the base of the line under consideration. For most basic buildings β_i will be the same for all lines of the seismic system.

The presence of rigid diaphragms in basic buildings introduces an additional level of complexity into the building analysis. However, this analysis can still be kept quite simple for many buildings.

For buildings with rigid diaphragms it will be necessary to consider the effect of the demand and resistance eccentricities (accidental displacement of the seismic floor mass and the location of the centre of stiffness or strength as appropriate). Refer Figure 10.77. If the lines of the seismic system in the direction being considered have some non-linear capability it is considered acceptable to resist the torque resulting from the eccentricities solely by the couple available from the lines of the seismic system perpendicular to the direction of loading. This will lead to a higher global capacity in many buildings than would otherwise be the case. If this approach is to be followed it would be more appropriate to consider the centre of strength rather than the centre of stiffness when evaluating the eccentricities.

NZS 1170.5 requires that buildings not incorporating capacity design be subjected to a lateral action set comprising 100% of the specified earthquake actions in one direction plus 30% of the specified earthquake actions in the orthogonal direction. The 30% actions perpendicular to the direction under consideration are not shown in Figure 10.77 for clarity and, suitably distributed, would need to be added to the shears to be checked for the perpendicular walls. These are unlikely to be critical for basic buildings. If the diaphragm is flexible, concurrency of the lateral actions should be ignored.



a) Linear elastic **b) With non-linear capability on wall lines 1 and 2**

Figure 10.77: Relationship between demand and capacity for a basic building with rigid diaphragms

In the above discussion it has been assumed that the diaphragms are stiff enough to provide the required support to the face-loaded walls orientated perpendicular to the direction of loading. Diaphragms are considered as primary structural components for the transfer of these actions and their ability to do so may affect the global capacity of the building in that direction. Limits have been suggested in Section 10.8.3.2 for the maximum diaphragm

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deflections to ensure adequate wall support. These limits are likely to be exceeded in flexible diaphragms, even in small basic buildings, and should be checked. If the limits are exceeded, the global capacity of the building in that direction will need to be reduced accordingly.

10.9.3 Global capacity of complex buildings

Many complex URM buildings will be able to be assessed adapting the recommendations outlined above for basic buildings. However, the assessment of complex buildings will often require a first principles approach and a good understanding of the past performance of such buildings.

The overall objective discussed in 10.9.1 remains. However, the more complex the building the more likely it will be necessary to utilise more complicated analysis techniques simply to keep track of element actions and applied inertial forces. It is recommended that simple techniques be used in all cases to identify the primary load paths and to verify the order of magnitude of the outputs.

Use of linear-elastic analysis techniques and limiting component capacities to elastic behaviour may significantly underestimate the global capacity of complex buildings. However, non-linear considerations can completely alter the mechanisms that can occur.

Aspects that are likely to require specific consideration in the assessment of complex buildings include:
- foundation stiffness
- diaphragm stiffness
- non-linear behaviour of multi-storey, penetrated walls, and development of sway mechanisms
- potential soft storeys
- non horizontal diaphragms.

10.9.4 Global analysis

10.9.4.1 Selection of analysis methods

Four analysis methods are generally considered:
- equivalent static analysis (linear static)
- modal response analysis (linear dynamic)
- non-linear pushover (nonlinear static)
- non-linear time history (nonlinear dynamic).

Linear analysis techniques supplemented with simple non-linear techniques (e.g. adapted SLaMA) are likely to be appropriate for all but the most complex of New Zealand's URM buildings.

Nonlinear analysis techniques are appropriate for buildings which contain irregularities and when higher levels of non-linear behaviour are anticipated. If nonlinear pushover analysis procedures are used, include appropriate allowances in the analysis for anticipated cyclic strength and stiffness degradation.

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Non-linear time history analyses can be used to analyse most URM buildings. They are able to account explicitly for cyclic strength and stiffness degradation. These analyses are complex. They should not be undertaken lightly and then only by those that have experience in the processes involved. A full appreciation of the reliability of the input parameters and the likely sensitivity of the outputs to these is required. Refer to relevant references for non-linear acceptance criteria.

Note:

Non-linear modelling of URM walls is feasible, but experience to date suggests that analytical results will not always provide reliable estimates of performance because of the variability in actual material strength and condition. Any analytical modelling should include several analyses to test sensitivity to material variation, modelling method and earthquake motion.

Special care is required with the application of damping, especially when considering a mix of low and high period modes. The resulting force reduction from damping for the mode considered should be investigated by a special study for finite element analysis. For assessing URM buildings, Caughey damping rather than Raleigh damping should be considered.

10.9.4.2 Mathematical modelling

Mathematical models used for linear analysis techniques should include the elastic, un-cracked in-plane stiffness of the primary lateral load-resisting elements. Consider both shear and flexural deformations.

If using non-linear analysis techniques, the mathematical model should directly incorporate the non-linear load-deformation characteristics of individual in-plane elements (i.e. backbone curves). Include cyclic degradation of strength and stiffness in the component modelling when appropriate. Recommended nonlinear analysis parameters for non-brittle URM failure modes are given in Section 10.8.6.2.

10.9.4.3 Fundamental period

The mass of URM buildings is normally dominated by the mass of the masonry. However, stiffness will depend on the relative flexibility of the walls, the floor diaphragms and the ground (foundation rotation). While the period of these structures can be quite difficult to calculate with precision and there are several modes of vibration to consider, it will often fall within the plateau section of the spectra, so precision is not required. For larger buildings (tall or long), especially those with long flexible diaphragms, special consideration of these effects may be required.

In the case of large buildings, it may not be sufficient to consider all parts of the building loaded at the same time and having the same time period. Commonly used methods include sub-structuring: i.e. subdividing the structure into sections, each including its elements and all mass tributary to it. Each section is then analysed separately and checked for compatibility with neighbouring sections along the margins between the sections. These sections should typically be no more than one third of the building width or more than 30 m.

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Note:

The effective period of individual sections of URM buildings will often still be short and, if this is the case, this final step will not be required.

10.9.4.4 Seismic mass

URM buildings are essentially systems with mass distributed over the height, with barely 10-20% of the seismic mass contributed by floors and roof. This is especially the case for buildings with timber floors and lightweight roofs. In this context, the concept of a lumped mass system is problematic. However, unless a more sophisticated analysis has been undertaken to capture the effect of distributed mass systems, an assessment based on masses lumped at diaphragm levels is acceptable as loads from the face-loaded walls would be transferred to the in-plane walls through the diaphragm.

However, for shear checks at the base of the in-plane walls and piers of any storey, the seismic demand should include accumulated floor level forces from the upper storeys and the seismic force due to the total mass of the in-plane wall above the level being considered. This is in contrast to assessments of concrete construction, where the mass of the lower half of the bottom storey is ignored when estimating the active mass for the base shear.

10.9.4.5 Stiffness of URM walls and wall piers subject to in-plane actions

The stiffness of in-plane URM walls subjected to seismic loads should be determined considering flexural, shear and axial deformations. The masonry should be considered to be a homogeneous material for stiffness computations with an expected elastic modulus in compression, E_m, as discussed in earlier sections.

For elastic analysis, the stiffness of an in-plane URM wall and pier should be considered to be linear and proportional with the geometrical properties of the un-cracked section, excluding any wythe, that does not meet the criteria given in Section 10.2.4.3.

Laboratory tests of solid shear walls have shown that behaviour can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered wall, k, can be calculated using Equation 10.51:

$$k = \frac{1}{\frac{h_{\mathrm{eff}}^3}{3E_m I_g} + \frac{h_{\mathrm{eff}}}{A_n G_m}} \qquad \ldots 10.51$$

where:

h_{eff} = wall height, mm
A_n = net plan area of wall, mm^2
I_g = moment of inertia for the gross section representing uncracked behaviour, mm^4
E_m = masonry elastic modulus, MPa
G_m = masonry shear modulus, MPa.

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The lateral in-plane stiffness of a pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation 10.52:

$$k = \frac{1}{\frac{h_{\mathrm{eff}}^3}{12 E_{\mathrm{m}} I_{\mathrm{g}}} + \frac{h_{\mathrm{eff}}}{A_{\mathrm{n}} G_{\mathrm{m}}}} \qquad \ldots 10.52$$

Note that a completely fixed condition is often not present in actual buildings.

10.10 Assessment of Earthquake Force and Displacement Demands

10.10.1 General

This section sets out the procedures for estimating both force and displacement demands on URM buildings and their parts.

Section 5 describes how the earthquake demands are to be assessed.

For the purposes of defining seismic demands, the structural system which carries seismic load and provides lateral resistance to the global building should be considered the primary seismic resisting system (primary structure). The components which do not participate in the overall resistance of the structure and which rely on the primary structure for strength and/or stability should be assumed to be parts and components. Parts and components need to be assessed for any imposed deformations from the primary seismic resisting system.

Therefore all in-plane walls and diaphragms are classified as primary structure. Everything else, such as face-loaded walls and parapets, and ornamentation, are considered to be parts and components.

10.10.2 Primary structure

10.10.2.1 General

Determine the horizontal demands on the primary structure, in accordance with Section 5 taking μ =1, S_{p} = 1 and ξ_{e} = 15%. Although μ is set at 1 it is intended that the benefits of any non-linear deformations from the assessment of the capacity are also taken.

10.10.2.2 Basic buildings

For basic buildings, a force-based assessment of in-plane demands for walls/piers and spandrels may be determined using a horizontal demand seismic coefficient, $C(T_1)$, given by Equation 10.53 where a load reduction factor, K_{R}, has been used in lieu of the ratio of the structural performance factor and structural ductility factor given in NZS 1170.5.

$$C(T_1) = C_{\mathrm{h}}(T_1)\, Z\, R_{\mathrm{u}}\, N(T_1, D)/K_{\mathrm{R}} \qquad \ldots 10.53$$

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where:

$C_h(T_1)$ = the spectral shape factor determined from Clause 3.1.2, NZS 1170.5 for the first mode period of the building, T_1, g

Z = the hazard factor determined from Clause 3.1.4, NZS 1170.5

R_u = the return period factor, R_u determined from Clause 3.1.5, NZS 1170.5

$N(T_1,D)$ = the near fault factor determined from Clause 3.1.6, NZS 1170.5

K_R = the seismic force reduction factor determined from Table 10.14.

Table 10.14: Recommended force reduction factors for linear static method

Seismic performance/ controlling parameters	Force reduction factor, K_R	Notes
Pier rocking, bed joint sliding, stair-step failure modes	3	Failure dominated by strong brick-weak mortar
Pier toe failure modes	1.5	
Pier diagonal tension failure modes (dominated by brick splitting)	1.0	Failure dominated by weak brick-strong mortar
Spandrel failure modes	1.0	

Note:

The concept of a ductility factor (deflection at ultimate load divided by the elastic deflection) can be meaningless for most URM buildings. The introduction of K_R primarily reflects an increase in the damping available and therefore reduced elastic response rather than ductile capability assessed by traditional means. Therefore the displacements calculated from the application of $C(T_1)$ are the expected displacements and should not be further modified by K_R.

These force reduction factors apply in addition to relief from period shift (if any).

Redistribution of seismic demands between individual elements of up to 50% is permitted when K_R = 3.0 applies, provided that the effects of redistribution are accounted for in the analysis.

When there are mixed behaviour modes among the walls/piers in a line of resistance, you can ignore the capacity of any piers for which K_R is less than the value that has been adopted for the line of resistance. Otherwise, consider lower force reduction factors. If you have adopted higher force reduction factors, carefully evaluate the consequences of loss of gravity load support from any walls/piers that have been ignored.

If there are mixed failure modes among the walls and piers in a line of resistance, the displacement compatibility between these piers and walls should be evaluated.

For the case of perforated walls when a strong pier – weak spandrel mechanism governs the wall behaviour K_R = 1.0 shall be adopted for the wall line as a whole, or the capacities of the spandrels can be ignored. When the contribution of the spandrels is ignored the higher K_R factors detailed in Table 10.14 may be used provided the consequences of loss of the ignored spandrels are considered.

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10.10.3 Parts and components

Refer to Section 8 of NZS 1170.5 for determination of seismic demands on parts and components.

For face-loaded walls, assessed using the displacement-based method in Section 10.8.5, the demands are included within the method. Note that the Part Spectral Shape Coefficient, $C_i(T_p)$, defined in NZS 1170.5 has been replaced with a formulation that better converts into a displacement spectrum for this purpose.

10.10.4 Vertical demands

Vertical ground motions in close proximity to earthquake sources can be substantial.

However, opinion is divided on how significant vertical accelerations are on the performance of URM buildings.

While vertical ground accelerations could potentially reduce the gravity and compression forces in the walls, reducing their stability, and reducing the pull-out strength of ties installed to restrain them back to the diaphragms, there is evidence to suggest that there is typically a time delay between the maximum vertical accelerations and the maximum horizontal accelerations, meaning that they are unlikely act together at full intensity.

In advance of further investigations on this subject, it is considered reasonable to ignore vertical accelerations when assessing the stability of masonry walls and the capacity of embedded anchors.

When vertical accelerations are considered the demands may be determined from NZS 1170.5.

10.10.5 Flexible diaphragms

10.10.5.1 General

Masonry walls loaded in-plane are typically relatively rigid structural elements. Consequently, the dominant mode of response for buildings containing flexible diaphragms is likely to be the response of the diaphragms themselves, due to inertial forces from diaphragm self-weight and the connected URM boundary walls responding out-of-plane.

Seismic demands on flexible diaphragms in URM buildings which are braced by URM walls should, therefore, be based on the period of the diaphragm and a horizontal seismic coefficient assuming that the diaphragm is supported at ground level (i.e. no amplification to reflect its height in the building). The seismic coefficient to be used is therefore $C(T)$ from NZS 1170.5 (i.e. S_p and μ =1) where T is the first horizontal mode period of the diaphragm.

If the diaphragm is braced by flexible (i.e. non-URM) lateral load resisting elements, the seismic demands can be determined using a seismic coefficient equal to F_i/m_i, with a lower limit of $C(0)$ where F_i is the equivalent static horizontal force determined from

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NZS 1170.5 at the level of the diaphragm and m_i is the seismic mass at that level. The intention is indicated in Figure 10.78. This requirement recognises that more flexible lateral load-resisting elements may cause the amplification of ground motions in the upper storeys.

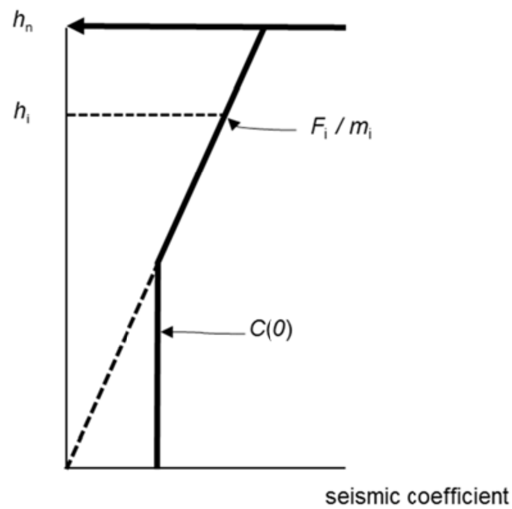

Figure 10.78: Distribution of acceleration with height for evaluating the demand on flexible diaphragms braced off flexible lateral load resisting elements

10.10.5.2 Timber diaphragms

The diaphragm in-plane mid-span lateral displacement demand, Δ_d, is given by Equation 10.54.

$$\Delta_d\ (\text{mm}) = \frac{3}{16}\frac{C(T_d)W_{\text{trib}}L}{BG'_{\text{d,eff}}} \qquad \text{...10.54}$$

where:

$C(T_d)$	=	seismic coefficient at required height for period, T_d, determined in accordance with Section 10.10.5.1
W_{trib}	=	uniformly distributed tributary weight, kN/m
L	=	span of diaphragm, m
B	=	depth of diaphragm, m
$G'_{\text{d,eff}}$	=	effective shear stiffness of diaphragm, refer Equation 10.55, MPa
T_d	=	lateral first mode period of the diaphragm determined in accordance with Equation 10.55, sec.

The period, T_d, of a timber diaphragm, based on the deformation profile of a shear beam excited in an approximately parabolic distribution, is given by Equation 10.55 (Wilson et al, 2013c).

$$T_d(sec) = 0.7 \times \sqrt{\frac{W_{\text{trib}}L}{G'_{\text{d,eff}}B}} \qquad \text{...10.55}$$

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where:

W_{trib} = total tributary weight acting on the diaphragm, being the sum of the weight of the tributary face-loaded walls both half-storey below and above the diaphragm being considered (i.e. the product of the tributary height, thickness and density of the out-of-plane URM walls tributary to the diaphragm accounting for wall penetrations) and diaphragm self-weight plus live load ($\psi_E \, x \, Q_i$ as per NZS 1170.5 Section 4.2).
Other terms are as defined for Equation 10.54.

10.10.6 Rigid diaphragms

Rigid diaphragms are primary structure and the demands are determined in accordance with NZS 1170.5 as outlined in Section 10.10.2.

10.10.7 Connections providing support to face-loaded walls

The demands on connections providing support to face-loaded masonry walls shall be calculated in accordance with Steps 12, 13 and 14 in Section 10.8.5.2.

Assume that the demand is uniformly distributed across all anchorages located at the specific wall-diaphragm interface. Repeat the exercise for the orthogonal loading direction, reversing loading regimes for a given anchorage.

10.10.8 Connections transferring diaphragm shear loads

Wall-diaphragm connections required to transfer shears from diaphragms to walls (loaded in-plane) should be considered to be primary structure and therefore the demands are evaluated in accordance with Section 10.10.2. The demand may be assumed to be uniformly distributed along the wall to diaphragm connection.

Unless capacity design principles are applied, the demands should be assessed assuming $\mu = S_p = 1$.

10.11 Assessment of *%NBS*

$$\%NBS = \text{Capacity/Demand x100} \qquad \ldots 10.56$$

where capacities and demands can be defined in terms of strength or deformation as appropriate.

The *%NBS* score (or rating) for the building is the minimum of the *%NBS* values assessed for each individual secondary component and the *%NBS* for the global performance of the primary structure for each principal direction.

The *%NBS* for the global performance of the building will be a function of individual scores for components and elements, and the hierarchy of these scores should also be noted.

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The item with the lowest *%NBS* score is referred to as the critical structural weakness, CSW. All other items with a score below 67*%NBS* are referred to as structural weaknesses, SWs.

It is an important aspect of the assessment process that all of the individual *%NBS* scores that have been evaluated are reported as this will provide a complete picture of the issues associated with the building's seismic performance and will aid in the development of a retrofit program if this is to be considered.

Although the impact on life safety of elements will have been considered when evaluating their effect on the capacity of the component, it is important that the list of structural weaknesses is reviewed again to ensure that any weaknesses, that do not directly lead to a life safety issue, do not appear in the list of structural weaknesses and do not limit the score of the building.

10.12 Improving Seismic Performance of URM Buildings

The overarching problem is that New Zealand's URM building stock is simply not designed for earthquake loads and lacks a basic degree of connection between structural components to allow all parts of the building to act together (Goodwin et al., 2011).

The basic approach to improving the seismic performance of URM buildings is to:

- secure all unrestrained parts that represent falling hazards to the public (e.g. chimneys, parapets and ornaments)
- improve the wall-diaphragm connections or provide alternative load paths; improve the diaphragm; and improve the performance of the face-loaded walls (gables, facades and other walls) by improving the configuration of the building and in-plane walls
- strengthen specific structural elements, and
- consider adding new structural components to provide extra support for the building.

When you are developing strengthening options, note that differing levels of seismic hazard will mean that a solution advised in a high seismic area could be too conservative in a low seismic area. Also note that even though a building may have more than 34*%NBS* seismic capacity, if that is limited by a brittle mode of failure and/or the failure mode could trigger a sequence of failure of other elements, the risk of failure of the limiting element should be carefully assessed and mitigated.

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Appendix 10A: On-site Testing

10A.1 General

While the seismic response of URM buildings is significantly influenced by characteristics such as boundary conditions and the behaviour of inter-element connections, on-site testing of material properties improves the reliability of the seismic assessments, the numerical models that describe the seismic behaviour of URM buildings and may lead to less conservative retrofit designs. However, the non-homogenous nature of masonry combined with the age of URM buildings make it difficult to reliably predict the material properties of masonry walls.

It is recommended that field sampling or field testing of URM elements is conducted. Field sampling refers to the extraction of samples from an existing building for subsequent testing offsite, while field testing refers to testing for material properties in-situ. A set of techniques are described in subsequent sections that can be used to determine masonry material properties.

Before proceeding to on-site testing, it is important to sensibly understand what information will be collected from the investigation, how that would be used and what value the information will add to reliability of the assessment. Before deciding an investigation programme, sensitivity analyses should be undertaken to determine what assessment parameters are more important and likely to influence the assessment result and whether the default parameter values given are likely to be appropriate/sufficient.

Only rarely should on-site testing be considered necessary for basic buildings.

10A.2 Masonry Assemblage (Prism) Material Properties

If masonry assemblage (prism) samples are to be extracted for laboratory testing they should be single leaf and at least three bricks high. If they are two leafs thick or more, cut them into single leaf samples. If present, remove rendering plaster from both sides of the samples. Cap the prepared samples using gypsum plaster to ensure uniform stress distribution.

Test individual brick units and mortar samples as per Section 10A.3 when sampling of larger assemblages is not permitted or practical. Masonry properties can then be predicted using the obtained brick and mortar properties as set out in Section 10.7.

10A.2.1 Masonry compressive strength

Determine masonry compressive strength in accordance with ASTM C 1314-03b (ASTM 2003a). Figure 10A.1 shows a typical prism sample before testing. Aluminium frames are attached to the sample ends and a displacement gauge spans between the frames to measure the sample displacement.

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ASTM C 1314-03b (ASTM 2003a) also enables you to determine the masonry modulus of elasticity (further detailed in Section 10A.2.2.1).

Figure 10A.1: Example of extracted sample with test rig attached for the prism compression test

10A.2.2 Masonry modulus of elasticity

10A.2.2.1 Laboratory calibrated displacement measurement

Laboratory calibrated displacement measurement devices may be attached to the masonry prisms during the compression tests detailed in Section 10A.2.1. Incorporate a minimum of two measurement devices to record displacements at opposing sample faces. Their gauge lengths should cover the distance from the middle of the top brick to the middle of the bottom brick. Use the recorded measurement to derive the masonry stress-strain relationship and subsequently the masonry modulus of elasticity, E_m. The stress and strain values considered in the calculation of E_m are those between 0.05 and 0.70 times the masonry compressive strength (f'_m).

10A.2.2.2 In situ deformability test incorporating flat jacks

Flat jack testing is a versatile and effective technique that provides useful information on the mechanical properties of historical constructions. In-situ measurements of masonry modulus of elasticity should be performed in accordance with the ASTM C 1197 - 04 (ASTM 2004) in situ deformability test.

Note:

Extensive studies have been conducted to confirm the reliability of this test, including the work by Noland J.L., Atkinson R.H., Schuller M.P. (1991), Gregorczyk and Lourenço (2000); Parivallal et al. (2011); and Simões (2012).

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The in-situ deformability test is moderately destructive as it requires the removal of horizontal mortar joints (bed-joint) for the insertion of the two flat jacks (Figure 10A.2a). The horizontal slots are separated by at least five courses of brickwork, but the separation distance should not exceed 1.5 times the flat jack length. A pressure controlled hydraulic pump is used to inflate the flat jacks, applying vertical confinement pressure to the masonry between the two jacks. To monitor displacement, typically three measurement devices are attached between the two flat jacks (Figure 10A.2b). These flat jacks need to be calibrated, following ASTM C 1197 - 04 (ASTM 2004).

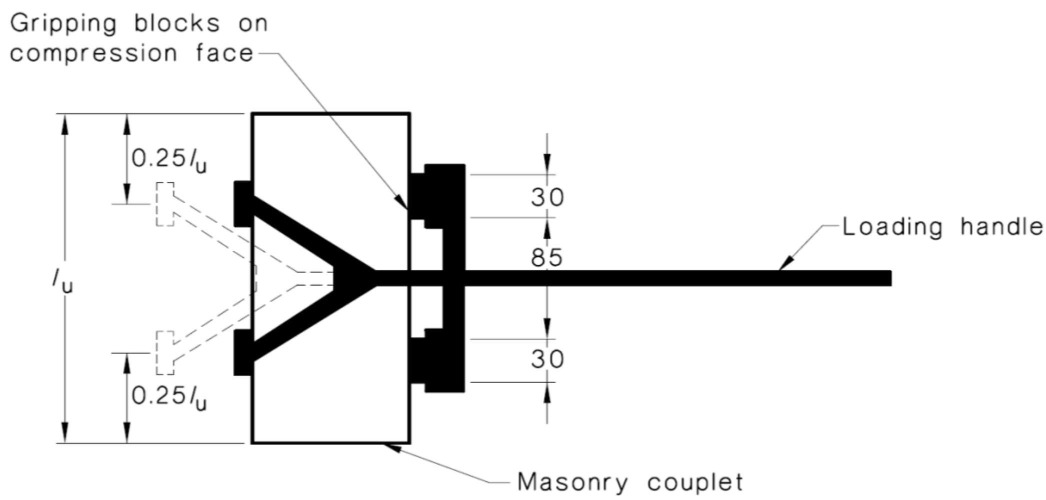
(a) Cutting mortar bed-joints and insertion of flat jacks into clay brick masonry

(b) In-situ deformability test set-up under preparation in clay brick masonry

Figure 10A.2: In situ deformability test preparation (EQ STRUC Ltd)

10A.2.3 Masonry flexural bond strength

Extract masonry prisms two bricks high and a single brick wide, and subject these to the flexural bond test of AS 3700-2001 (Australian Standards, 2001). Remove any rendering plaster from the sides of the sample before performing this test. Cut any samples that are two leafs thick or more into single leaf masonry prism samples. Alternatively, you may conduct the flexural bond test in situ if this is more practical.



(a) Plan view

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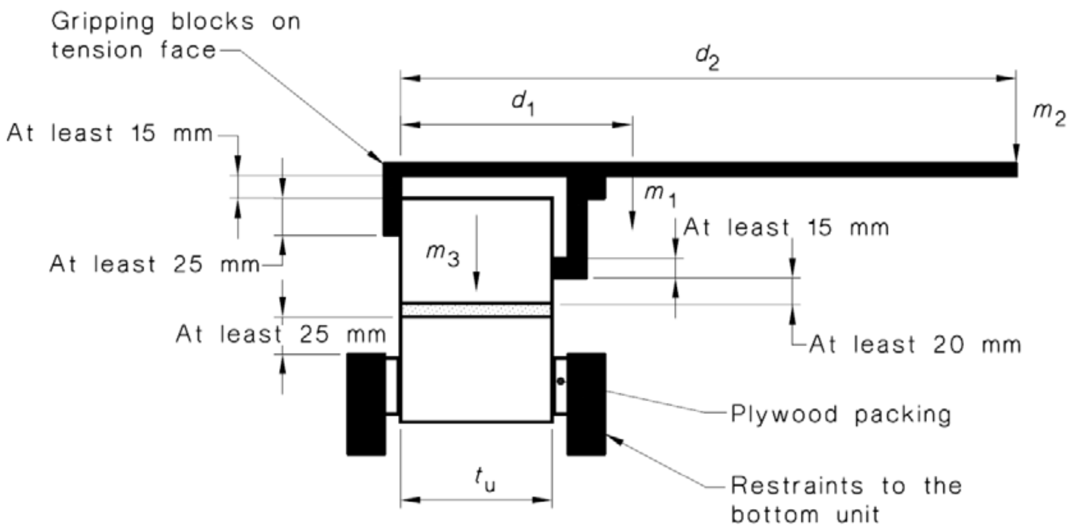

(b) Elevation view

Figure 10A.3: Flexural bond test-set-up (AS 3700-2001)

10A.2.4 Masonry bed-joint shear strength

Conduct the ASTM C 1531-03 (ASTM 2003b) in-situ bed-joint shear test to determine masonry bed-joint properties. This type of test is moderately destructive as it requires the removal of at least one brick on one side of the test specimen to allow for insertion of a hydraulic jack, as well as the removal of a vertical mortar joint on the opposite side to allow horizontal bed joint movement to occur. The hydraulic jack is then loaded, using a pressure controlled hydraulic pump, until visible bed-joint sliding failure occurred. You can then derive the bed-joint shear strength from the peak pressure records.

Alternatively, extract three brick high masonry prisms for laboratory testing following the triplet shear test BS EN 1052-3 (BSI 2002). This test should be conducted while applying axial compression loads of approximately 0.2 MPa, 0.4 MPa and 0.6 MPa. At least three masonry prism samples should be tested at each level of axial compression. Remove any rendering plaster from both sides of the sample before testing. Cut any masonry samples that are two leafs thick or more into single leaf samples. Bed-joint shear tests performed in the laboratory and in situ are shown in Figure 10A.4.

(a) Laboratory shear triplet test

(b) In situ shear test without flat jacks (EQ STRUC Ltd)

Figure 10A.4: In situ and laboratory bed- joint shear test

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The in situ bed-joint shear test is limited to tests of the masonry face leaf. When the masonry unit is pushed in a direction parallel to the bed joint, shear resistance is provided across not only the bed-joint shear planes but also the collar joint shear plane. Because seismic shear is not transferred across the collar joint in a multi-leaf masonry wall, the estimated shear resistance of the collar joint must be deducted from the test values. This reduction is achieved by including a 0.75 reduction factor in Equation 10.33, which is the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar joints for a typical clay masonry unit.

The term P in Equation 10.33 represents the axial overburden acting on the bed joints. This value multiplied by the bed-joint coefficient of friction, (μ_f), allows estimation of the frictional component contributing to the recorded bed-joint stress. Due to the typical large variation of results obtained from individual bed-joint shear strength tests, the equation conservatively assumes $\mu_f = 1.0$ for the purposes of determining cohesion, c. Therefore, for simplicity, the μ_f term has been omitted from the equation.

10A.3 Constituent Material Properties

10A.3.1 Brick compressive strength

Extract individual brick units for the ASTM C 67-03a (ASTM 2003a) half brick compression test. Cut these brick units into halves and cap them using gypsum plaster before compression testing (Figure 10A.5). Note that it is possible to obtain half brick units from the residual samples of the Modulus of Rupture test described in Section 10A.3.2.

Figure 10A.5: Brick and mortar sample and compression test set-up (EQ STRUC Ltd)

10A.3.2 Brick modulus of rupture

Extract individual brick units from the building and subject these to the modulus of rupture (MoR) test ASTM C 67-03a (ASTM 2003a). The tested brick specimens from the MoR test may be subjected to the half brick compression test ASTM C 67-03a (ASTM 2003a) in order to obtain a direct relationship between the brick MoR and compressive strength, f'_b. Previous experimental investigation has confirmed that the brick unit MoR can be approximated to equal $0.12f'_b$.

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10A.3.3 Mortar compressive strength

Extract irregular mortar samples for laboratory testing. As it is common for URM walls to have eroded mortar joints that were later repaired using stronger mortar, take care when selecting the location for mortar sample extraction to ensure that your samples are representative.

The method to determine mortar compressive strength is detailed in ASTM C 109-08 (ASTM 2008). This method involves testing of 50 mm cube mortar samples, which generally are not attainable in existing buildings as most mortar joints are only 10 to 18 mm thick. Therefore, cut the irregular mortar samples into approximately cubical sizes with two parallel sides (top and bottom). The height of the mortar samples should exceed 15 mm in order to satisfactorily maintain the proportion between sample size and the maximum aggregate size. Cap the prepared samples using gypsum plaster to ensure a uniform stress distribution and testing in compression (Valek and Veiga, 2005): see Figure 10A.6 for examples.

Measure the height to minimum lateral dimension (h/t) ratio of the mortar samples and use this to determine the mortar compressive strength correction factors. Divide the compression test result by the corresponding correction factors in Equation 10A.1. The average corrected strength is equal to the average mortar compressive strength, f'_j.

$$f'_j = \alpha_{tl}\alpha_{ht}f'_{ji} \qquad \text{…10A.1}$$

where:

f'_{j}	=	normalised mortar compressive strength
α_{tl}	=	t/l ratio correction factor
α_{ht}	=	t/l ratio correction factor
f'_{ji}	=	measured irregular mortar compressive strength.

Equation 10A.1 normalises the measured compressive strength of irregular mortar samples to the compressive strength of a 50 mm cube mortar. Factors α_{tl} and α_{ht} are calculated as per Equations 10A.2 and 10A.3 (where $M.F$ should be calculated as per Equation 10A.4) respectively. Factor α_{tl} is required in order to normalise the sample t/l ratio to 1.0, while factor α_{ht} is required in order to normalise the sample h/t ratio to 1.0, corresponding to a cubic mortar sample that is comparable to a 50 mm cube. These factors were derived based on the study detailed in Lumantarna (2012).

$$\alpha_{\mathrm{tl}} = 0.42\frac{t}{l} + 0.58 \qquad \text{…10A.2}$$

$$\alpha_{\mathrm{ht}} = \frac{1}{M.F} \qquad \text{…10A.3}$$

$$M.F = 2.4(\frac{h}{t})^2 - 5.7(\frac{h}{t}) + 4.3 \qquad \text{…10A.4}$$

When conducting tests on laboratory manufactured samples make 50 mm mortar cubes, leave these to cure under room temperature (±20 °C) for 28 days, and test them in compression following the mortar cube compression test ASTM C 109-08 (ASTM 2008).

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(a) Example of typical extracted mortar samples **(b) Example of typical mortar sample preparations** **(c) Example of typical test set-up**

Figure 10A.6: Determination of mortar compression strength (EQ STRUC Limited)

10A.4 Proof Testing of Anchor Connections

An epoxied or grouted anchorage system is a typical method of connecting the floor and roof diaphragms of the building to masonry walls. Reliable anchor pull-out and shear strength is important for assessment or design of anchors and the specification of anchor spacing. Standard installation procedures of embedded anchors involve drilling the masonry wall, cleaning the drilled hole and epoxying or grouting threaded steel bars to the specified embedment depth, typically 50 mm less than the wall thickness. Two-part epoxy or high strength grouts are typically used with surface preparation conducted in accordance with the manufacturer's specifications.

On-site quality control and proof testing should be undertaken on at least 15% of all installed adhesive anchors, of which 5% should be tested prior to the installation of more than 20% of all anchors. Testing is required to confirm workmanship (particularly the mixing of epoxy and cleaning of holes) and anchor capacity against load requirements. If more than 10% of the tested anchors fail below a test load of 75% of the nominated probable capacity, discount the failed anchors from the total number of anchors tested as part of the quality assurance test. Test additional anchors to meet the 15% threshold requirements. Failures that cannot be attributed to workmanship issues are likely to be indicative of an overestimation of the available capacity and a reassessment of the available probable capacity is likely to be required.

10A.4.1 Anchors loaded in tension

Once the adhesive is cured (typically over 24 hours), the steel anchors can be loaded in tension using a hydraulic jack until ultimate carrying capacity is reached (ASTM, 2003) or when the load exceeds two times the specified load. The typical test set-up is shown in Figure 10A.7. A 600 mm clear span of reaction frame allows testing of up to 300 mm embedment depth without exerting any confining pressures onto the test area, as the reaction frame supports are outside the general zone of influence. On completing the test, the anchor stud is typically cut flush with the wall surface.

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(a) Typical anchor pull test set up

(b) Close up of the typical test set-up with an alternative test frame

Figure 10A.7: Typical anchor pull-out test set-up (EQ STRUC Ltd)

10A.4.2 Anchors loaded in shear

The test set-up that could be adopted for in situ testing of anchors loaded in shear is shown in Figure 10A.8. Monotonic shear loading can be applied by using a single acting hydraulic actuator, with the external diameter of the actuator selected to be as small as possible. The bracket arrangements should minimise the tension loads in the anchors. The aim is to determine the shear capacity in the absence of tension.

(a) Typical anchor shear tests set-up (push cycle)

(b) Typical anchor shear tests set-up (pull cycle)

Figure 10A.8: Shear tests set-up used (EQ STRUC Ltd)

10A.5 Investigation of Collar Joints and Wall Cavities

Investigation of collar joints quality and wall cavities can be undertaken using a Ground Penetrating Radar (GPR) structural scanner (Figure 10A.9a). The scanner is capable of accurately determining the member thickness, metallic objects, voids and other

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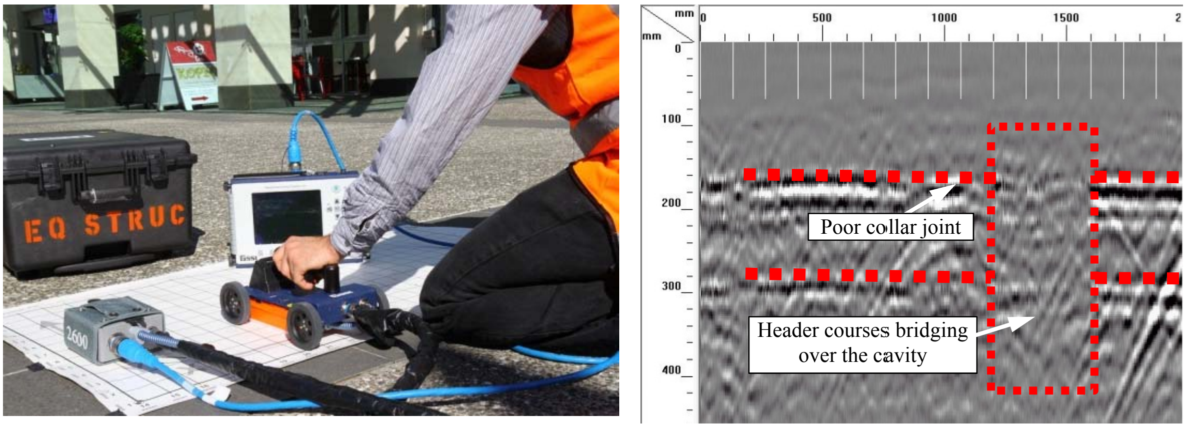
information. An example of the information provided by GPR scanning is presented in Figure 10A.9b.



(a) Ground Penetrating Radar (GPR) scanner **(b) Typical results output**

Figure 10A.9: Example of non-invasive scanning using Ground Penetrating Radar (GPR) scanner technology (EQ STRUC Ltd)

10A.6 Cavity Tie Examination

The main focus of the cavity tie examination is to identify the condition and frequency of the cavity ties embedded between the leaves of the cavity URM walls. A borescope inspection camera can be used to inspect the air cavity through a void left from a removed brick or an air vent (Figure 10A.10).

(a) Borescope inspection camera **(b) Typical example of cavity observations**

Figure 10A.10: Borescope inspection camera (EQ STRUC Ltd)

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Appendix 10B: Derivation of Instability Deflection and Fundamental Period for Face-Loaded Masonry Walls

10B.1 General considerations and approximations

There are many variations that need to be taken into account when considering a general formulation for URM walls that might fail out-of-plane. These include:

- Walls will not usually be of a constant thickness in a building, or even within a storey.
- Walls will have embellishments, appendages and ornamentation that may lead to eccentricity of masses with respect to supports.
- Walls may have openings for windows or doors.
- Support conditions will vary.
- Existing buildings may be rather flexible, leading to possibly large inter-storey displacements that may adversely affect the performance of face-loaded walls.

You can use the following approximations to simplify your analysis while still accounting for some the key important factors.

1 Deformations due to distortions (straining) in the wall can be ignored. Assume deflections to be entirely due to rigid body motion.

> *Note:*
>
> *This is equivalent to saying that the change in potential energy from a disturbance of the wall from its initial position is mostly due to the movement of the masses of the elements comprising the wall and the movements of the masses tributary to the wall. Strain energy contributes less to the change in potential energy.*

2 Assume that potential rocking occurs at the support lines (e.g. at roof or floor levels) and, for walls that are supported at the top and bottom of a storey, at the mid-height. The mid-height rocking position divides the wall into two parts of equal height: a bottom part (subscript *b*) and a top part (subscript *t*). The masses of each part are not necessarily equal.

> *Note:*
>
> *It is implicit within this assumption and (1) above that the two parts of the wall remain undistorted when the wall deflects. For walls constructed of softer mortars or walls with little vertical pre-stress from storeys above, this is not actually what occurs: the wall takes up a curved shape, particularly in the upper part. Nevertheless, errors occurring from the use of the stated assumptions have been found to be small and you will still obtain acceptably accurate results.*

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3 Assume the thickness to be small relative to the height of the wall. Assume the slope, *A*, of both halves of the wall to be small; in the sense that $cos(A) \approx 1$ and $sin(A) \approx A$.

Note:

The approximations for slope are likely to be sufficiently accurate for reasonably thin walls. For thick walls where the height to thickness ratio is smaller, the formulations in this appendix are likely to provide less accurate results and force-based approaches provide an alternative.

4 Inter-storey slopes due to deflection of the building are assumed to be small.

Note:

Approximate corrections for this effect are noted in the method.

5 In dynamic analyses, the moment of inertia is assumed constant and equal to that applying when the wall is in its undisturbed position, whatever the axes of rotation.

Note:

The moment of inertia is dependent on the axes of rotation. During excitation, these axes continually change position. Assuming that the inertia is constant is reasonable within the context of the other approximations employed.

6 Damping is assumed at the default value in NZS 1170.5:2004, which is 5% of critical.

Note:

For the aspect ratio of walls of interest, additional effective damping due to loss of energy on impact is small. Furthermore, it has been found that the surfaces at rocking (or hinge) lines tend to fold onto each other rather than experience the full impact that is theoretically possible, reducing the amount of equivalent damping that might be expected. However, for in-plane analysis of buildings constructed largely of URM, adopting a damping ratio that is significantly greater than 5% is appropriate.

7 Assume that all walls in storeys above and below the wall under study move "in phase" with the subject wall.

Note:

Analytical studies have found this to be the case. One reason for this is that the effective stiffness of a wall as it moves close to its limit deflection (e.g. as measured by its period) becomes very low, affecting its resistance to further deflection caused by accelerations transmitted to the walls through the supports. This assumption means that upper walls, for example, will tend to restrain the subject wall by exerting restraining moments.

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10B.2 Vertically spanning walls

10B.2.1 General formulation

Figures 10B.1 and 10B.2 show the configuration of a wall panel within a storey at two stages of deflection. The wall is intended to be quite general. Simplifications to the general solutions for walls that are simpler (e.g. of uniform thickness) are made in a later section.

Figure 10B.1 shows the configuration at incipient rocking. Figure 10B.2 shows the configuration after significant rocking has occurred, with the wall having rotated through an angle A and with mid-height deflection, Δ, where $\Delta = Ah/2$.

In Figure 10B.1 the dimensions e_b and e_t relate to the mass centroids of the upper and lower parts of the panel. The dimension e_p relates to the position of the line of action of weights from upper storeys (walls, floors and roofs) relative to the centroid of the upper part of the panel. The arrows on the associated dimensioning lines indicate the positive direction of these dimensions for the assumed direction of motion (angle A at the bottom of the wall is positive in the anti-clockwise sense). Under some circumstances the signs of the eccentricities may be negative; for example for e_p when an upper storey wall is much thinner than the upper storey wall represented here, particularly where the thickness steps on one face. When the lines of axial force from diaphragm and walls from above are different, the resultant force should be calculated.

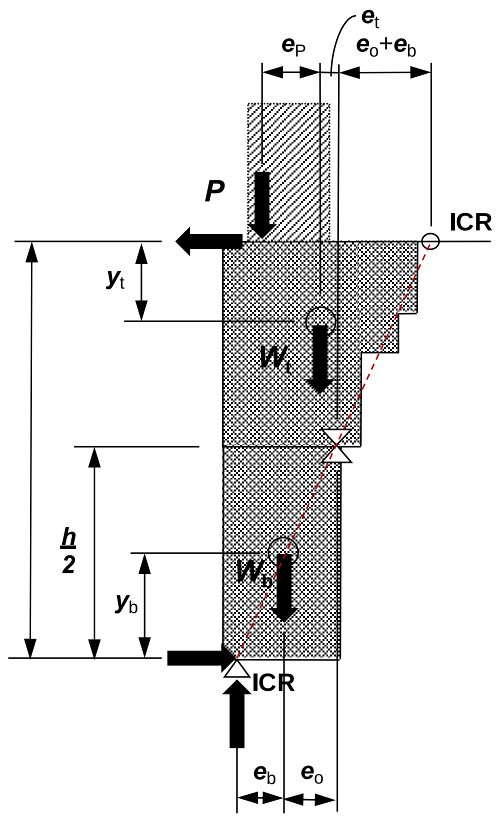

Figure 10B.1: Configuration at incipient rocking

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The instantaneous centres of rotation (ICR) are also marked on these figures. These are useful in deriving virtual work expressions.

10B.2.2 Limiting deflection for static instability

You can write the equation of equilibrium directly by referring to Figure 10B.2 and using virtual work expressions. For static conditions this is given by:

$$W_{\mathrm{b}}(e_{\mathrm{b}} - Ay_{\mathrm{b}}) + W_{\mathrm{t}}\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} - A(h - y_{\mathrm{t}})\big) + P\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}} - Ah\big) - \Psi(W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}y_{\mathrm{t}}) = 0 \qquad \ldots 10\mathrm{B}.1$$

The final term represents the effect of any inter-storey drift. In the derivation presented, the total deformation has been assumed to be that resulting from the summation of the drift and the rocking wall.

Writing:

$$a = W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}(h - y_{\mathrm{t}}) + Ph \qquad \ldots 10\mathrm{B}.2$$

and:

$$b = W_{\mathrm{b}}e_{\mathrm{b}} + W_{\mathrm{t}}(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}}) + P\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}}\big) - \Psi(W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}y_{\mathrm{t}}) \qquad \ldots 10\mathrm{B}.3$$

and collecting terms in *A,* the equation of equilibrium is rewritten as:

$$-aA + b = 0 \qquad \ldots 10\mathrm{B}.4$$

from which:

$$A = \frac{b}{a} \qquad \ldots 10\mathrm{B}.5$$

when the wall becomes unstable.

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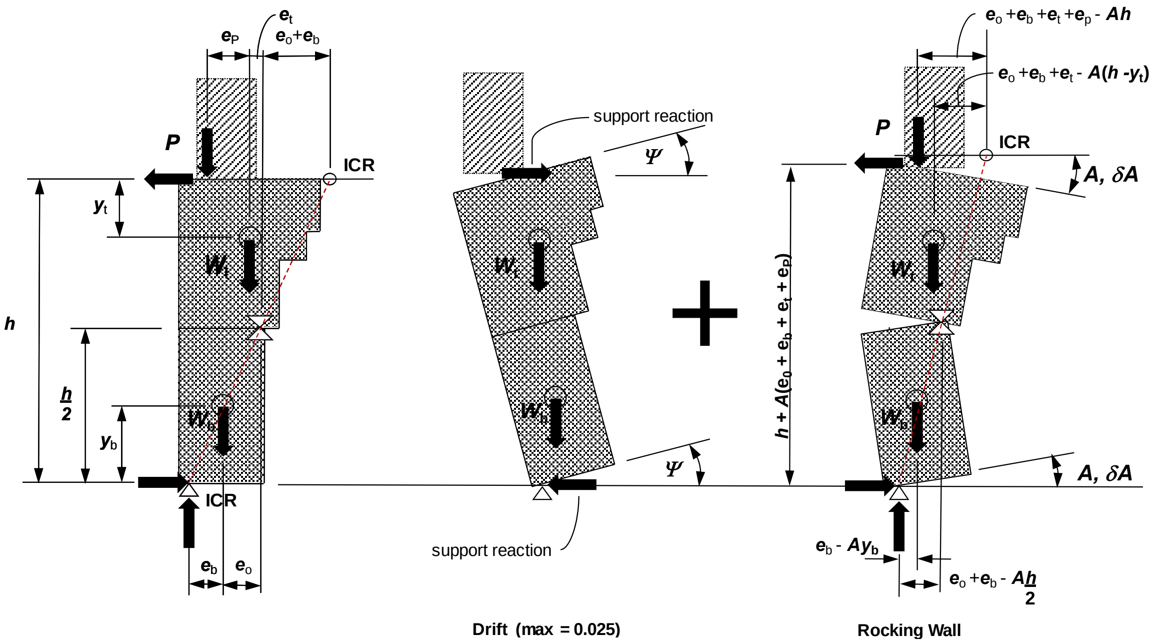

Figure 10B.2: Configuration when rotations have become significant and there is inter-storey drift

Therefore, the critical value of the deflection at mid-height of the panel, at which the panel will be unstable, is:

$$\Delta_i = A\frac{h}{2} = \frac{bh}{2a} \qquad \ldots 10B.6$$

It is assumed that Δ_m, a fraction of this deflection, is the maximum useful deflection. Experimental and analytic studies indicate that this fraction might be assumed to be about 0.6. At larger displacements than $0.6\Delta_i$, analysis reveals an undue sensitivity to earthquake spectral content and a wide scatter in results.

10B.2.3 Equation of motion for free vibration

When conditions are not static, the virtual work expression on the left-hand side in the equation above is unchanged, but the zero on the right-hand side of the equation is replaced by mass x acceleration, in accordance with Newton's law. This gives:

$$-aA + b = -J\ddot{A} \qquad \ldots 10B.7$$

This uses the usual notation for acceleration (a double dot to denote the second derivative with respect to time; in this case indicating angular acceleration), and J as the rotational inertia.

The rotational inertia can be written directly from Figures 10B.1 and 10B.2, noting that the centroids undergo accelerations vertically and horizontally as well as rotationally, and these accelerations relate to the angular acceleration in the same way as the displacements relate to the angular displacement. While the rotational inertia is dependent on the

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displacements, the effects of this variation are ignored. Therefore, the rotational inertia is taken as that when no displacement has occurred. This gives the following expression for rotational inertia.

$$J = J_{\mathrm{bo}} + J_{\mathrm{to}} + \frac{1}{\mathrm{g}}\left\{W_{\mathrm{b}}[e_{\mathrm{b}}^2 + y_{\mathrm{b}}^2] + W_{\mathrm{t}}[(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}})^2 + y_{\mathrm{t}}^2] + P\left[\left(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}}\right)^2\right]\right\} + J_{\mathrm{anc}} \quad \text{...10B.8}$$

where J_{bo} and J_{to} are the mass moments of inertia of the bottom and top parts respectively about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but contribute to its inertia.

For a wall with unit length, held at the top and bottom, and rocking crack at mid-height, with a density of ρ per unit volume, the mass moment of inertia about the horizontal axis through the centroid is given by:

$$I_{\mathrm{xx}}(\mathrm{kgm}^2) = \rho\frac{t_{\mathrm{Gross}}\left(\frac{h}{2}\right)^3}{12} \quad \text{...10B.9}$$

The corresponding mass moment of inertia about the vertical axis through the centroid is:

$$I_{\mathrm{yy}}\ (\mathrm{kgm}^2) = \rho\frac{\left(\frac{h}{2}\right)t_{\mathrm{Gross}}^3}{12} \quad \text{...10B.10}$$

The polar moment of inertia through the centroid is the sum of these, or:

$$J_{\mathrm{bo}}(\ \mathrm{kgm}^2) = J_{\mathrm{to}} = I_{\mathrm{xx}} + I_{\mathrm{yy}} = \rho t_{\mathrm{Gross}}\left(\frac{h}{2}\right)\frac{\left[t_{\mathrm{Gross}}^2+\left(\frac{h}{2}\right)^2\right]}{12} = \frac{m}{2}\frac{\left[t_{\mathrm{Gross}}^2+\left(\frac{h}{2}\right)^2\right]}{12} = \frac{W}{2g}\frac{\left[t_{\mathrm{Gross}}^2+\left(\frac{h}{2}\right)^2\right]}{12} \quad \text{...10B.11}$$

where m is the mass (kg) and W (N) is the weight of the whole wall panel and g is the acceleration of gravity.

Note that in this equation the expressions in square brackets are the squares of the radii from the instantaneous centres of rotation to the mass centroids, where the locations of the instantaneous centres of rotation are those when there is no displacement. Some CAD programs have functions that will assist in determining the inertia about an arbitrary point (or locus), such as about the ICR shown in Figure 10B.2.

Collecting terms and normalising the equation so that the coefficient of the acceleration term is unity gives the following differential equation of free vibration:

$$\ddot{A} - \frac{a}{J}A = -\frac{b}{j} \quad \text{...10B.12}$$

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10B.2.4 Period of free vibration

The solution of the equation for free vibration derived in the previous section is:

$$A = C_1 sinh\left(\sqrt{\frac{a}{J}}\tau\right) + C_2 cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{…10B.13}$$

The time, τ, is taken as zero when the wall has its maximum rotation, $A\ (=\Delta/2h)$. Using this condition and the condition that the rotational velocity is zero when the time $\tau = 0$, the solution becomes:

$$A = \left(\frac{2\Delta}{h} - \frac{b}{a}\right) cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{….10B.14}$$

Take the period of the "part", T_{p}, as four times the duration for the wall to move from its position at maximum deflection to the vertical. Then the period is given by:

$$T_{\mathrm{p}} = 4\sqrt{\frac{J}{a}} cosh^{-1}\left(\frac{\frac{b}{a}}{\frac{b}{a} - \frac{2\Delta}{h}}\right) \qquad \text{…10B.15}$$

This can be simplified further by substituting the term for Δ_{i} found from the static analysis and putting the value of Δ used for the calculation of period as Δ_{t} to give:

$$T_{\mathrm{p}} = 4\sqrt{\frac{J}{a}} cosh^{-1}\left(\frac{1}{1 - \frac{\Delta_{\mathrm{t}}}{\Delta_{\mathrm{i}}}}\right) \qquad \text{…10B.16}$$

If we accept that the deflection ratio of interest is 0.6 (i.e. ${}^{\Delta_{\mathrm{m}}}/{}_{\Delta_{\mathrm{i}}} = 0.6$), then this becomes:

$$T_{\mathrm{p}} = 6.27\sqrt{\frac{J}{a}} \qquad \text{…10B.17}$$

as in the 2006 guidelines. However, research (Derakhshan et al, (2014a)) indicates that the resulting period and responding displacement demand is too large if a spectrum derived from linear elastic assumptions is used. Rather, this research suggests that an effective period calculated from an assumed displacement of 60% of the assumed displacement capacity should be used. Therefore, the period is based on $\Delta_{\mathrm{t}} = 0.36\Delta_{\mathrm{i}}$ so that:

$$T_{\mathrm{p}} = 4.07\sqrt{\frac{J}{a}} \qquad \text{…10B.18}$$

10B.2.5 Maximum acceleration

The acceleration required to start rocking of the wall occurs when the wall is in its initial (undisturbed) state. This can be determined from the virtual work equations by assuming that $A=0$. Accordingly:

$$\ddot{A}_{\max} = \frac{b}{J} \qquad \text{…10B.19}$$

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However, a more cautious appraisal assumes that the acceleration is influenced primarily by the instantaneous acceleration of the supports, transmitted to the wall masses, without relief by wall rocking. Accordingly:

$$C_m = \frac{b}{(W_b y_b + W_t y_t)} \qquad \text{…10B.20}$$

where C_m is the acceleration *coefficient* to just initiate rocking.

10B.2.6 Participation factor

The participation factor can be determined in the usual way by normalising the original form of the differential equation for free vibration, modified by adding the ground acceleration term. For the original form of the equation, the ground acceleration term is added to the RHS. Written in terms of a unit rotation, this term is ($W_b y_b$ + $W_t y_t$) times the ground acceleration. The equation is normalised by dividing through by J, and then multiplied by h/2 to convert it to one involving displacement instead of rotation. The participation factor is then the coefficient of the ground acceleration. That is:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \qquad \text{…10B.21}$$

10B.2.7 Simplifications for regular walls

You can make simplifications where the thickness of a wall within a storey is constant, there are no openings, and there are no ancillary masses. Further approximations can then be applied:

- The weight of each part (top and bottom) is half the total weight, W.
- $y_b = y_t = h/4$
- The moment of inertia of the whole wall is further approximated by assuming that all e are very small relative to the height (or, for the same result, by ignoring the shift of the ICR from the mid-line of the wall), giving $J = Wh^2/12g$. Alternatively, use the simplified expressions for J given in Table 10B.1.

10B.2.7.1 Approximate displacements for static instability

Table 10B.1 gives values for a and b and the resulting mid-height deflection to cause static instability when e_b and/or e_p are either zero or half of the effective thickness of the wall, t. In this table e_o and e_t are both assumed equal half the effective wall thickness. While these values of the eccentricities are reasonably common, they are not the only values that will occur in practice.

The effective thickness may be assumed as follows:

$$t = \left(0.975 - 0.025\frac{P}{W}\right) t_{\text{nom}} \qquad \text{…10B.22}$$

where t_{nom} is the nominal thickness of the wall.

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Experiments show that this is a reasonable approximation, even for walls with soft mortar. In that case, there is greater damping and that reduces response, which compensates for errors in the expression for effective thickness.

10B.2.7.2 Approximate expression for period of vibration

Noting that:

$$a = \left(\frac{W}{2} + P\right) h \qquad \text{...10B.23}$$

and using the approximation for *J* relevant to a wall with large aspect ratio, the expression for the period is given by:

$$T_{\mathrm{p}} = 4.07\sqrt{\frac{2Wh}{12g(W+2P)}} \qquad \text{...10B.24}$$

where it should be noted that the period is independent of the restraint conditions at the top and bottom of the wall (i.e. independent of both e_{b} and e_{p}).

If the height is expressed in metres, this expression simplifies to:

$$T_{\mathrm{p}} = \sqrt{\frac{0.28h}{(1+2P/W)}} \qquad \text{...10B.25}$$

It should be appreciated that periods may be rather long.

This approximation errs on the low side, which leads to an underestimate of displacement demand and therefore to slightly incautious results. The fuller formulation is therefore preferred.

10B.2.7.3 Participation factor

Suitable approximations can be made for the participation factor. This could be taken at the maximum value of 1.5. Alternatively, the numerator can be simplified as provided in the following expression, and the simplified value of *J* shown in Table 10B.1 can be used.

10B.2.7.4 Maximum acceleration

By making the same simplifications as above, the maximum acceleration is given by:

$$\ddot{A}_{\mathrm{max}} = \frac{b}{J} = \frac{12bg}{Wh^2} \qquad \text{...10B.26}$$

Or, more cautiously, the *acceleration coefficient*, C_{m}, is given in Table 10B.1 for the common cases regularly encountered.

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10B.2.8 Adjustments required when inter-storey displacement is large

Using the common limit on ψ of 0.025, and substituting for $W_b = W_t = W/2$ and $y_b = y_t = h/4$, δb is found to be $Wh/160$. Taking h/t = 25, in the absence of any surcharge, the percentage reduction in the instability deflection for each case shown in Table 10B.1 is 31% for Cases 0 and 2, and 16% for Cases 1 and 3. These are not insignificant, and these affects should be assessed especially in buildings with flexible principal framing such as steel moment-resisting frames.

Table 10B.1: Static instability defection for uniform walls, various boundary conditions

Boundary condition number	**0**	**1**	**2**	**3**
				
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	t/2
b	$(W/2+P)$t	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)$h	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)\text{t}}{(2W+4P)}$	T
J	$\{(W/12)[h^2 +7t^2] +Pt^2\}/g$	$\{(W/12)[h^2+16t^2] +9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2] +9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2] +4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

Note:
1. The boundary conditions of the piers shown above are for clockwise potential rocking.
2. The top eccentricity, e_t is not related to a boundary condition, hence is not included in the table. The top eccentricity, e_t, is the horizontal distance from the central pivot point to the centre of mass of the top block which is not related to a boundary condition.
3. The eccentricities shown in the sketches are for the positive sense. Where the top eccentricity is in the other sense e_p should be entered as a negative number.

10B.3 Vertical cantilevers

10B.3.1 General formulation

Figure 10B.3 shows a general arrangement of a cantilever. The wall illustrated has an overburden load at the top, but this load will commonly be zero, as in a parapet. Where a load does exist it is important to realise that the mass associated with that load can move horizontally. As a result the inertia of the wall is affected by the overburden to a greater extent than if the wall was supported horizontally at the top. If the top load is supported on the wall in such a way that its point of application can change, as is the case if it is through a continuous beam or slab that crosses the wall, there will be an eccentricity of the point of application of P.

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Sometimes several walls will be linked; for example, when a series of face-loaded walls provide the lateral resistance to a single-storey building. This case can be solved by methods derived from the general formulation, but express formulations for it are not provided here.

For the single wall illustrated, it is assumed that P is applied eccentric to the centre of the wall at the top and that point of application remains constant. It is straightforward to obtain the following parameters:

$$a = Wy_{\mathrm{b}} + Ph \qquad \ldots 10B.27$$

$$b = We_{\mathrm{b}} + P(e_{\mathrm{b}} + e_{\mathrm{p}}) \qquad \ldots 10B.28$$

$$J = \frac{W}{12g}(h^2 + t_{\mathrm{nom}}^2) + \frac{W}{g}(y_{\mathrm{b}}^2 + e_{\mathrm{b}}^2) + \frac{P}{g}\left[h^2 + \left(e_{\mathrm{b}} + e_{\mathrm{p}}\right)^2\right] \qquad \ldots 10B.29$$

Note that in these equations e_{p} is taken as positive in the sense shown in Figure 10B.3.

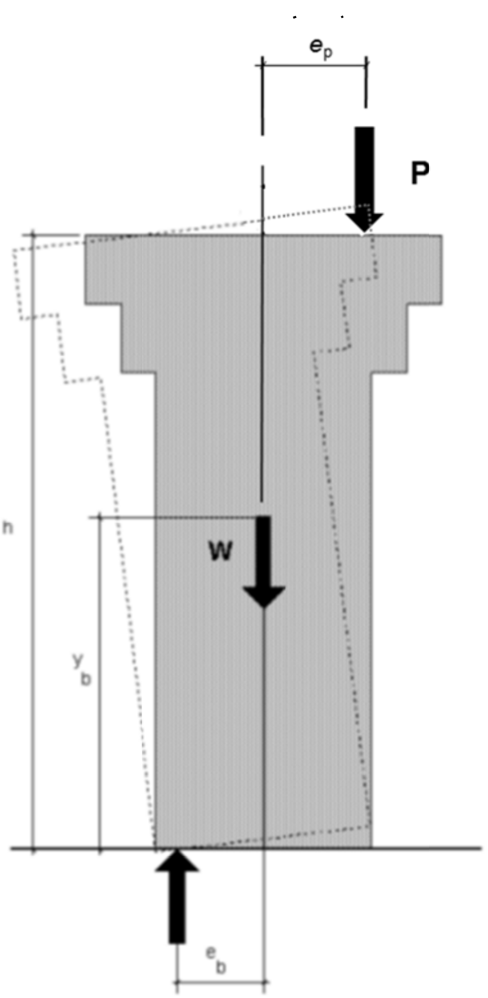

Figure 10B.3: Single cantilever

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10B.3.2 Limiting deflection for static instability

When the wall just becomes unstable, the relationship for *A* remains the same as before but the deflection is *Ah*. Thus, the limiting deflection is given by:

$$\Delta_{\mathrm{i}} = Ah = \frac{bh}{a} = \frac{[We_{\mathrm{b}} + P(e_{\mathrm{b}} + e_{\mathrm{p}})]h}{Wy_{\mathrm{b}} + Ph} \qquad \text{...10B.30}$$

For the case where P=0 and $y_{\mathrm{b}}=h/2$ this reduces to $\Delta_{\mathrm{i}} = 2e_{\mathrm{b}} = t$.

10B.3.3 Period of vibration

If $\Delta_{\mathrm{t}} = 0.36\Delta_{\mathrm{i}}$ as for the simple case, the general expression for period would remain valid. However, cantilevers are much more susceptible to instability under real earthquake stimulation than wall panels that are supported both top and bottom. Therefore, the maximum useable displacement for calculation of capacity, Δ_{m}, is reduced from $0.6\Delta_{\mathrm{i}}$ to $0.3\Delta_{\mathrm{i}}$ and the displacement for calculation of period changes from $0.6\Delta_{\mathrm{m}}$ to $0.8\Delta_{\mathrm{m}} = 0.24\Delta_{\mathrm{i}}$ so that:

$$T_{\mathrm{p}} = 3.1\sqrt{\frac{J}{a}} \qquad \text{…10B.31}$$

Where P=0, $e_{\mathrm{b}}=t/2$, $y_{\mathrm{b}}=h/2$, approximating $t=t_{\mathrm{nom}}$ and expressing h in metres, the period of vibration is given by:

$$T_{\mathrm{p}} = \sqrt{0.65h\left[1 + \left(\frac{t}{h}\right)^2\right]} \qquad \text{…10B.32}$$

Note that *P*, whether eccentric or not, will not affect the static instability displacement, and therefore neither the displacement demand (by affecting the period), nor the displacement capacity.

10B.3.4 Participation factor

The expression for the participation factor remains unaffected; that is, $\gamma = Wh^2/2J$. This may be simplified for uniform walls with P=0 (no added load at the top) by inserting the specific expression for *J*. This gives:

$$\gamma = \frac{3}{2\left(1 + \left(\frac{t}{h}\right)^2\right)} \qquad \text{…10B.33}$$

10B.3.5 Maximum acceleration

Using the same simplifications as above:

$$C = \frac{t}{h} \qquad \text{…10B.34}$$

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Appendix 10C: Charts for Assessment of Out-of-Plane Walls

10C.1 General

This appendix presents simplified ready-to-use charts for estimation of *%NBS* for face-loaded unreinforced masonry walls with uniform thickness. The charts have been developed for walls with various slenderness ratios (wall height/thickness) vs Basic Performance Ratio (BPR). The BPR can be converted to *%NBS* after dividing it by the product of the appropriate spectral shape factor ($C_h(0)$, required to evaluate $C(0)$ for parts), return period factor (R), hazard factor (Z), near-fault factor ($N(T, D)$), and part risk factor (R_p) which have been assigned unit values for developing the charts. The charts are presented for various boundary conditions and ratio of load on the wall to self-weight of the wall.

Refer to Section 10 and Appendix 10B for symbols and sign conventions.

This appendix includes charts for the following cases:

- one-way vertically spanning walls laterally supported both at the bottom and the top with no inter-storey drift
- one-way vertically spanning walls laterally supported at the top and the bottom with inter-storey drift of 0.025
- vertical cantilever walls.

The following section presents how these charts should be used.

10C.2 One-way Vertically Spanning Face-Loaded Walls

Charts for one-way vertically spanning walls are presented in Figures 10C.1a-f, 10C.2a-f and 10C.3a-f for 110 mm, 230 mm and 350 mm thick walls respectively for inter-storey drift of 0.00. Similarly, charts for an inter-storey drift of 0.025 are presented in Figures 10C.4a-f, 10C.5a-f and 10C.6a-f for 110 mm, 230 mm and 350 mm thick walls respectively. The charts have been developed for $e_t = e_o = t/2$ and various values for e_p.

Follow the following steps for estimation of *%NBS* for a vertically spanning face-loaded wall:

- Identify thickness, t_{Gross} and height, h of the wall.
- Calculate slenderness ratio of the wall (h/t_{Gross}).
- Calculate the total self-weight, W of the wall.
- Calculate vertical load, P on the wall. This should include all the dead load and appropriate live loads on the wall from above.
- Calculate P/W.
- Calculate eccentricities (e_b and e_p). e_b could be $t/2$ or 0, whereas e_p could be $\pm t/2$ or 0. To assign appropriate values, check the base boundary condition and location of P on the wall. Calculation of effective thickness, t is not required.

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- Refer to the appropriate charts (for appropriate e_b and e_p, P/W and inter-storey drift).
- Estimate Basic Performance Ratio (BPR) from the charts. Linear interpolation between plots may be used as necessary for inter-storey drifts between 0 and 0.025.
- Refer NZS 1170.5 for $C_h(0)$ required to evaluate $C(0)$ for parts, R, Z, $N(T, D)$, C_{Hi} and R_p. For estimation of C_{Hi}, h_i is height of the mid-height of the wall from the ground.
- $\%NBS = \frac{Basic\ Performance\ Ratio\ from\ charts\ for\ h/t}{C_h(0)RZN(T,D)C_{Hi}R_P}$

10C.3 Vertical Cantilevers

Charts for one-way vertically spanning walls are presented in Figures 10C.7a-c, 10C.8a-c and 10C.9a-c for 110 mm, 230 mm and 350 mm thick walls respectively.

Follow the following steps for estimation of *%NBS* of a face-loaded cantilever wall:

- Identify thickness, t_{Gross} and height, h of the wall.
- Calculate slenderness ratio of the wall (h/t_{Gross}).
- Calculate total self-weight, W of the wall above the level of cantilevering plane.
- Calculate vertical load, P on the wall, if any. This should include all the dead load and appropriate live loads on the wall from above.
- Calculate P/W.
- Calculate eccentricity, e_p, for loading $P(e_p)$. e_p could be ±t/2 or 0, which depends upon location of P on the wall. Calculation of effective thickness, t is not required.
- Refer to the appropriate charts (for appropriate e_p, and P/W).
- Estimate Basic Performance Ratio (BPR) from the charts. Interpolation between plots may be used as necessary.
- Refer NZS 1170.5 for $C_h(0)$ required to evaluate $C(0)$ for parts, R, Z, $N(T, D)$, C_{Hi} and R_p. For estimation of C_{Hi}, h_i shall be taken as height of the base of the cantilever wall.
- $\%NBS = \frac{Basic\ Performance\ Ratio\ from\ charts\ for\ h/t}{C_h(0)RZN(T,D)C_{Hi}R_P}$

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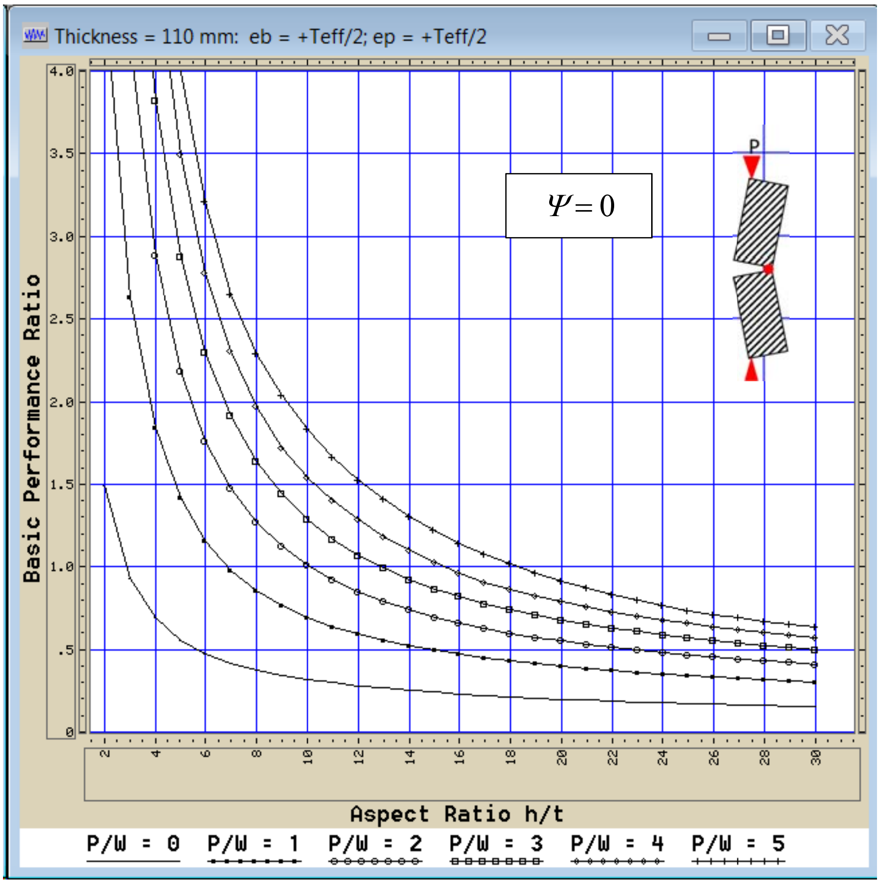
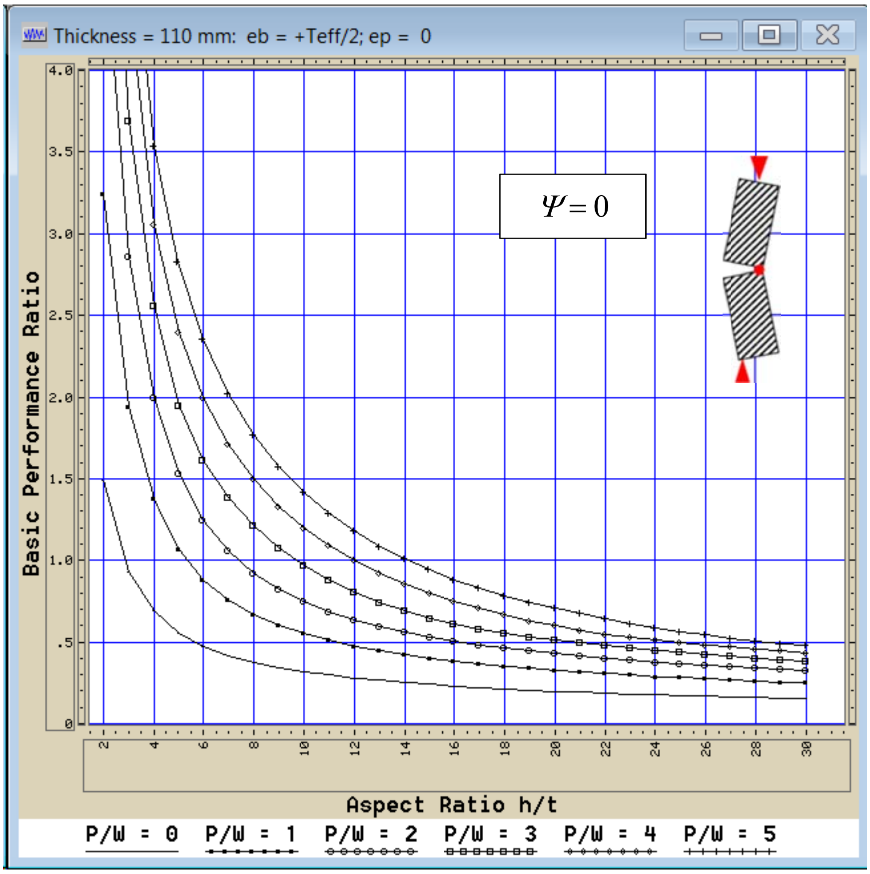


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} =110 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 110 mm)

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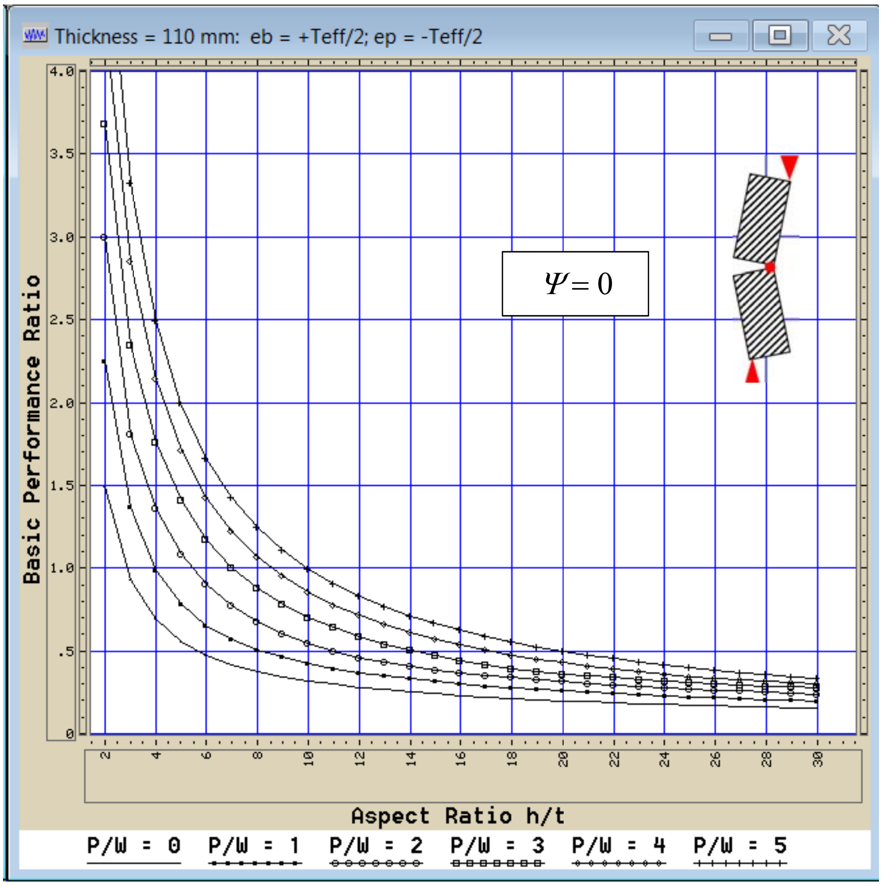
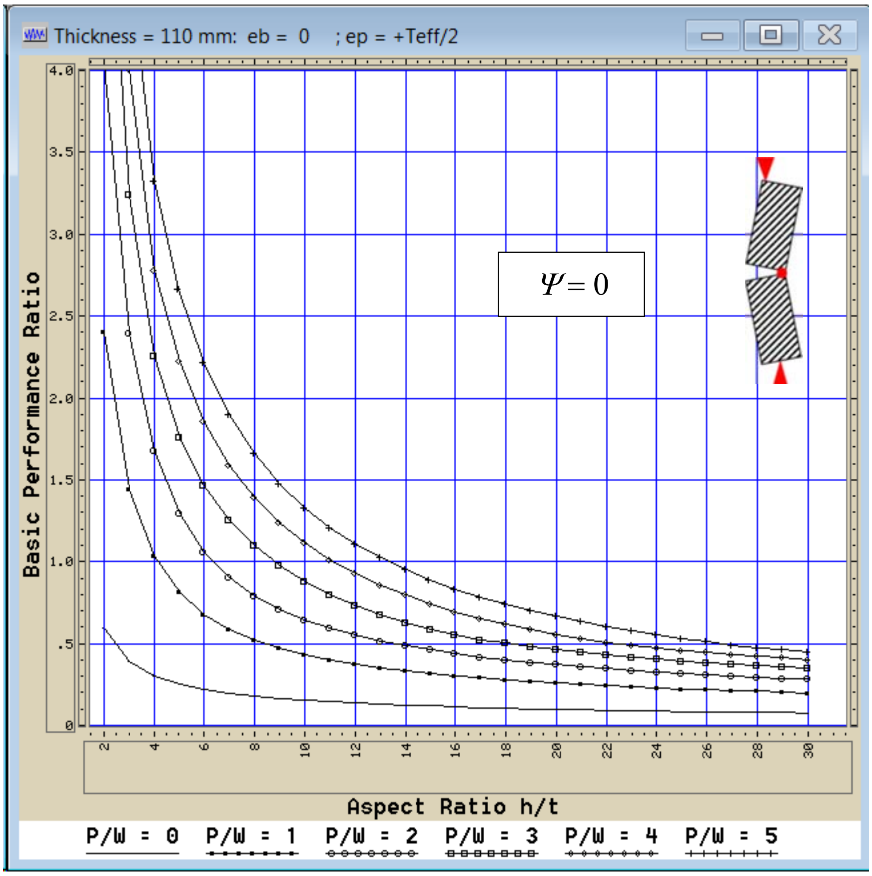


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 110 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 110 mm)

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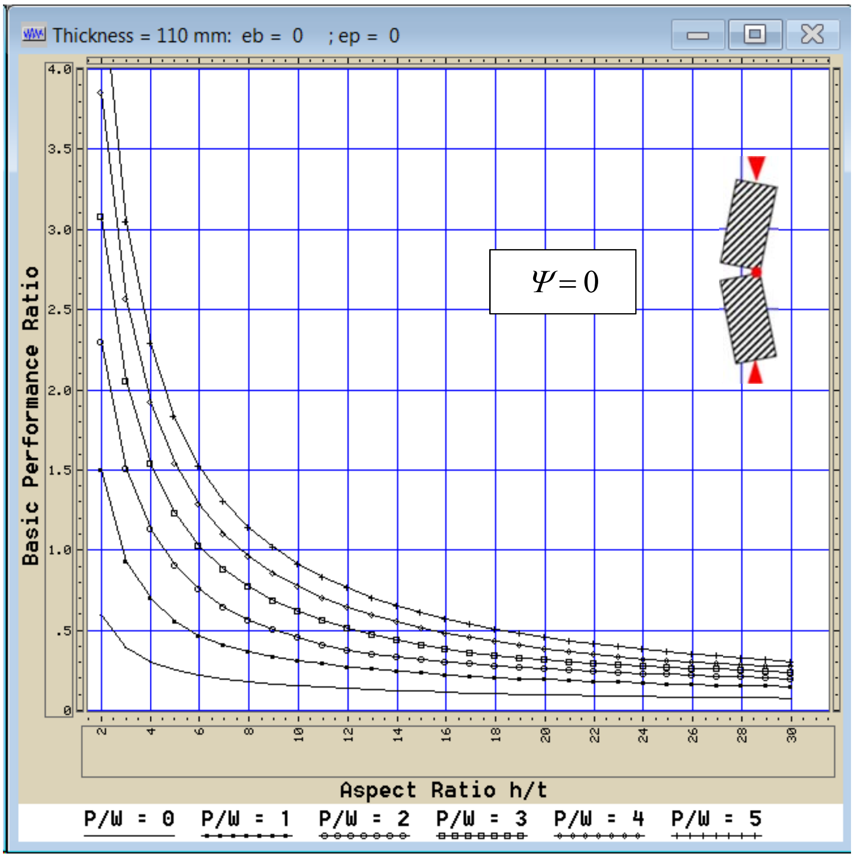
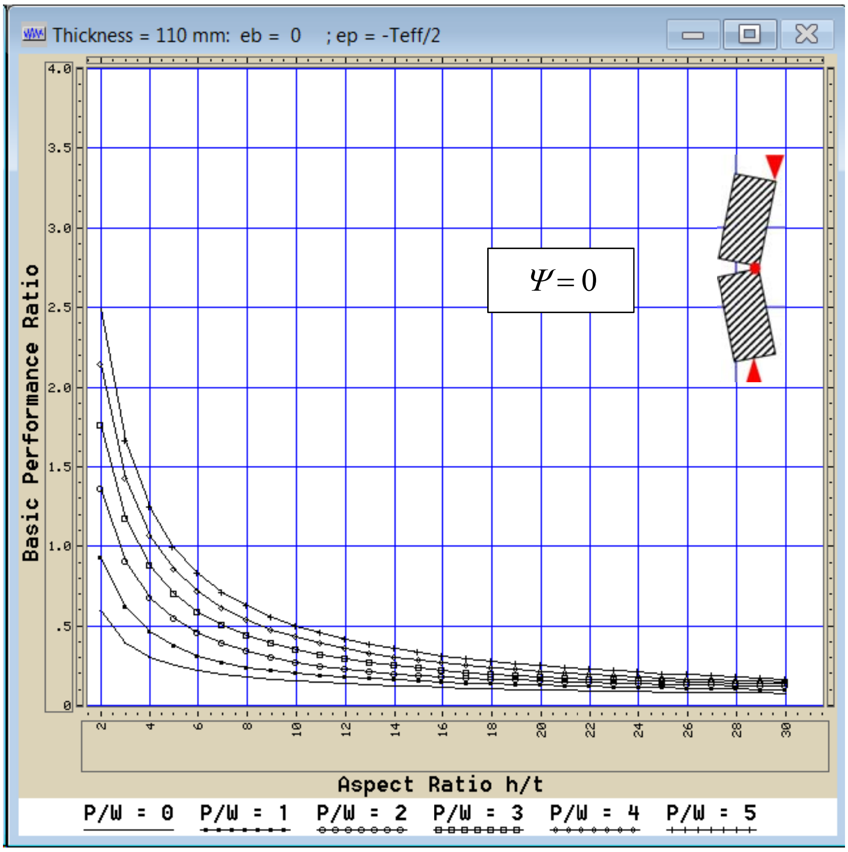


e) For e_b = 0 and e_p = 0 (t_{Gross} = 110 mm)



f) For e_b = 0 and e_p = -*t*/2 (t_{Gross} = 110 mm)

Figure 10C.1: 110 mm thick one-way vertically spanning face-loaded walls (Ψ = 0)

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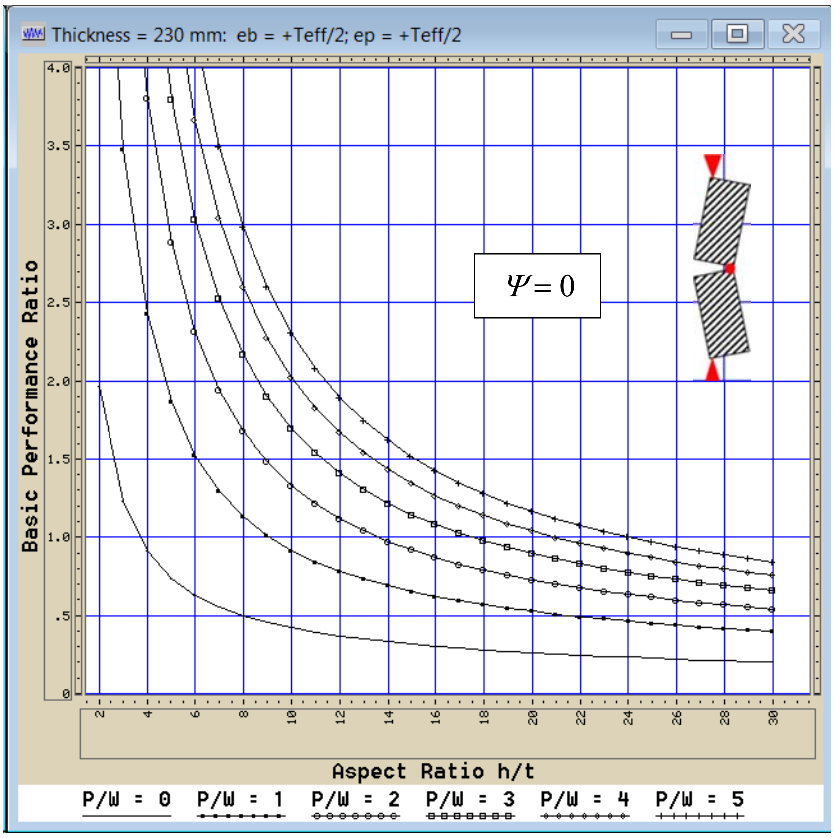
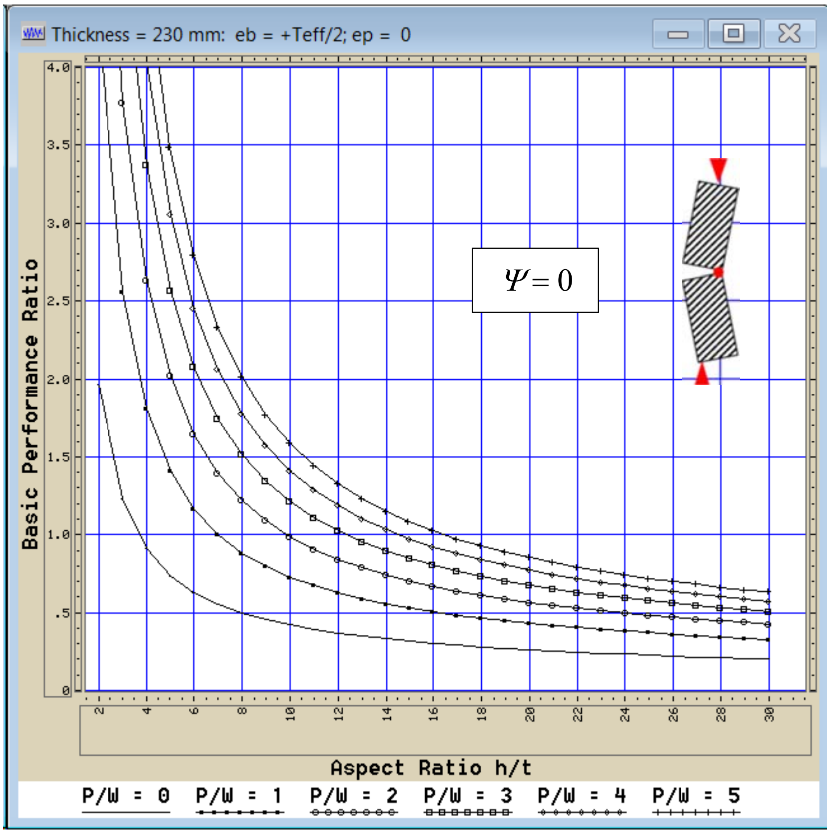


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} =230 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} =230 mm)

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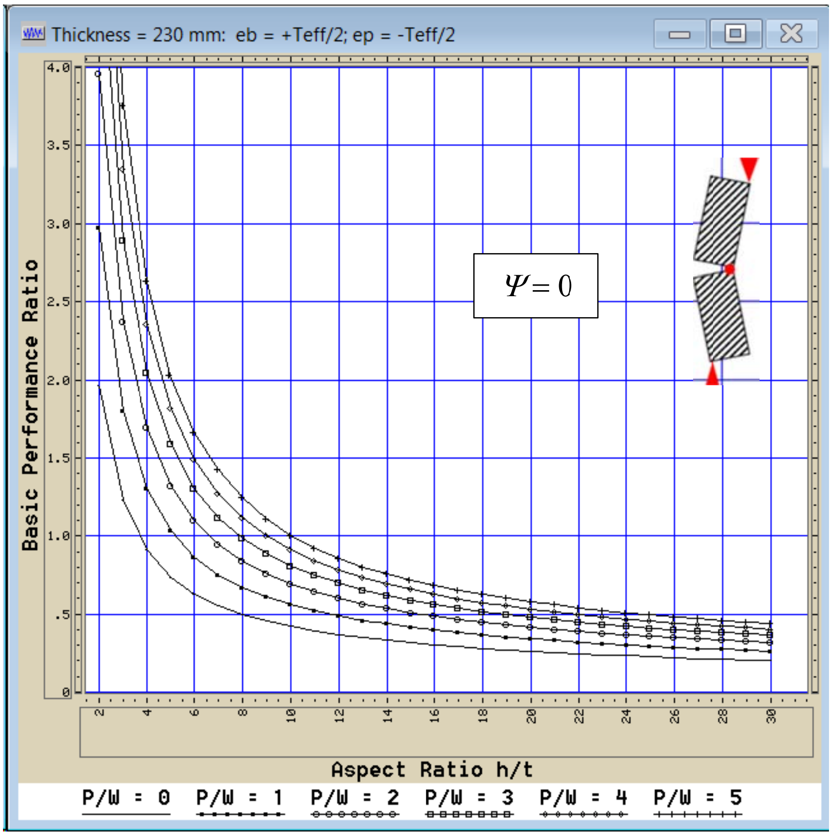
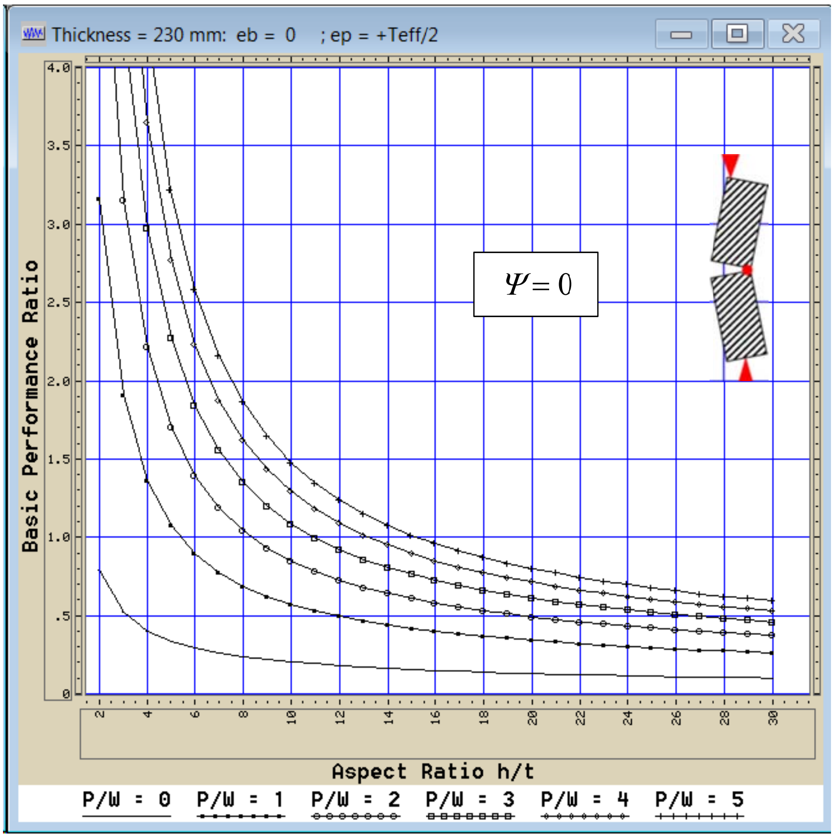


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 230 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 230 mm)

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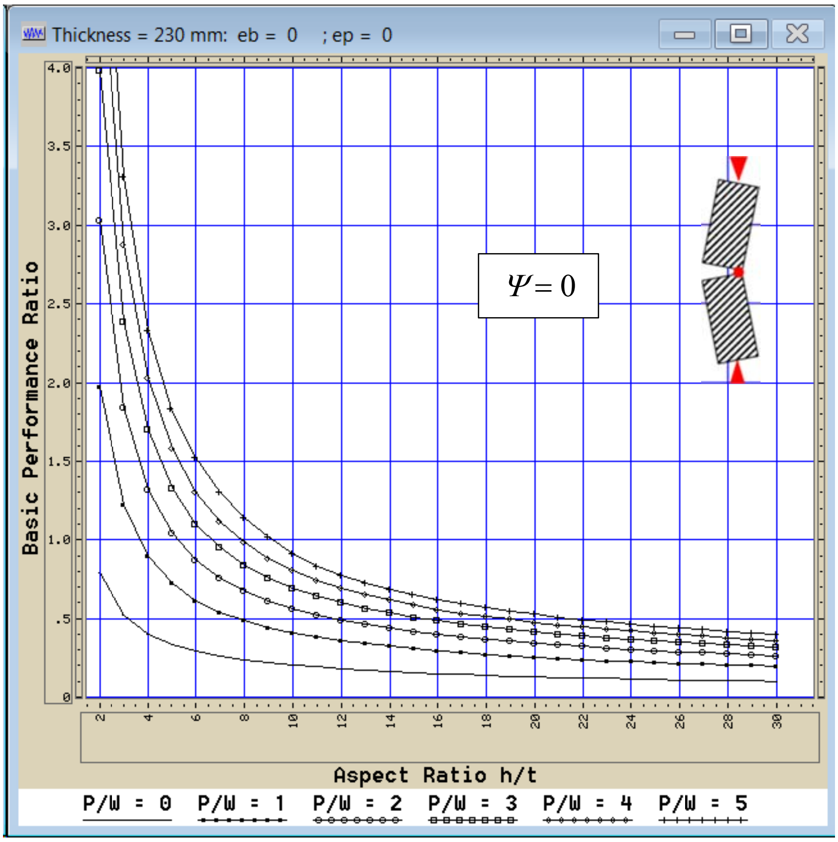
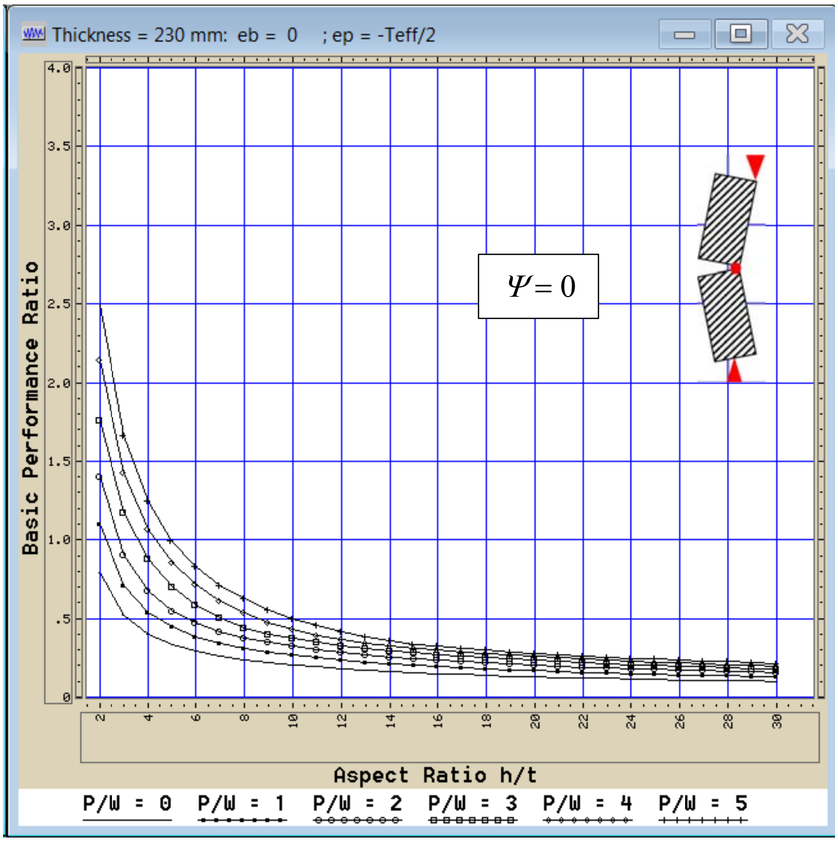


e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 230$ mm)



f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 230$ mm)

Figure 10C.2: 230 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0$)

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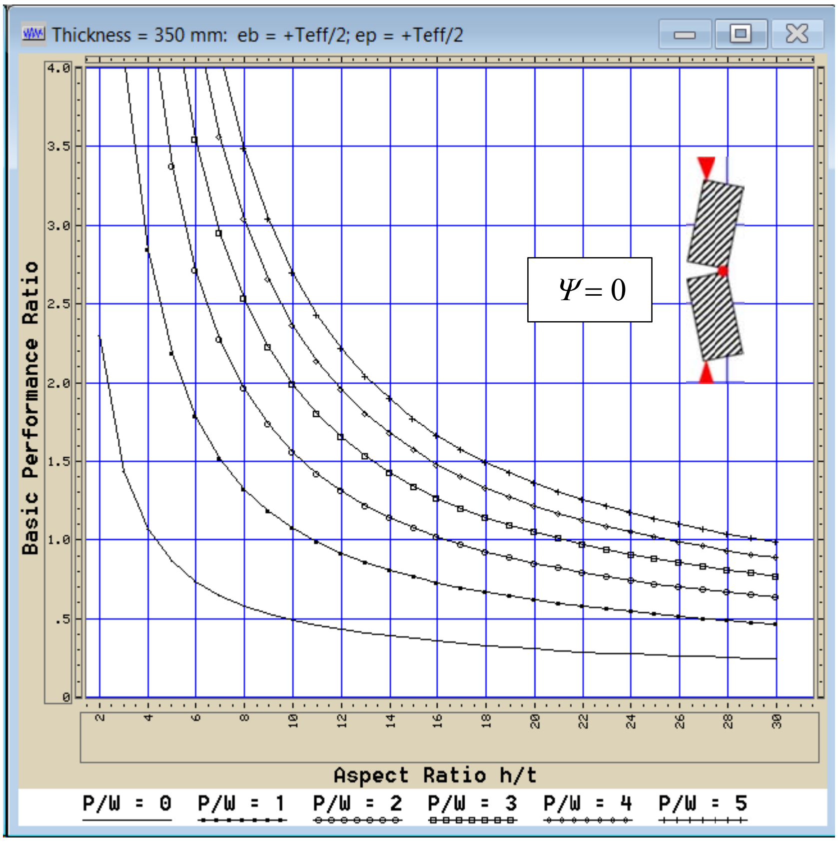
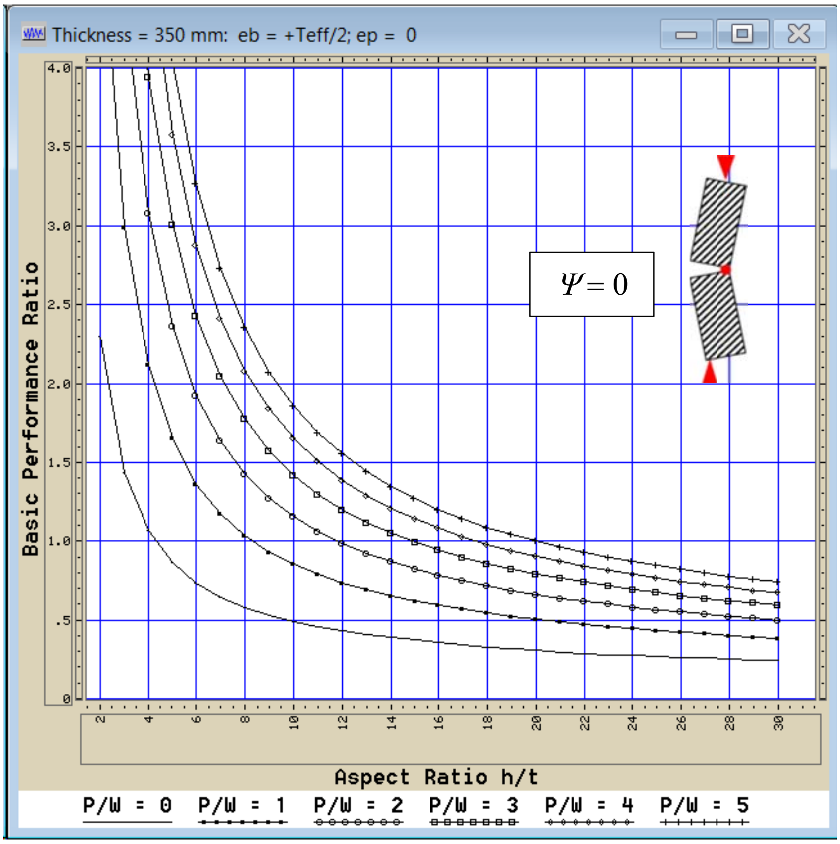


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} = 350 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 350 mm)

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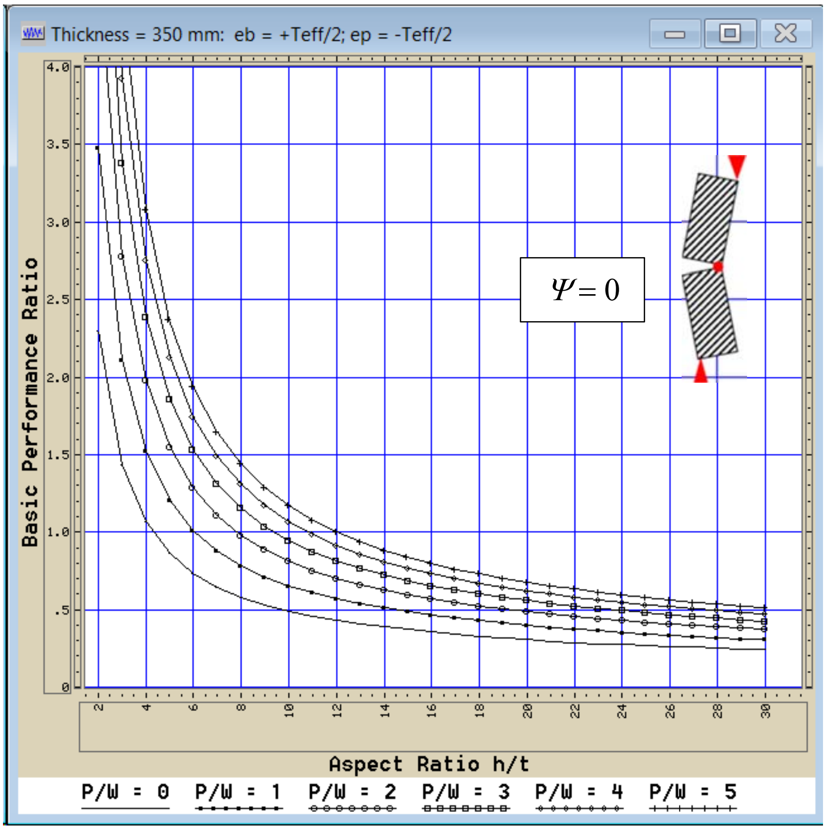
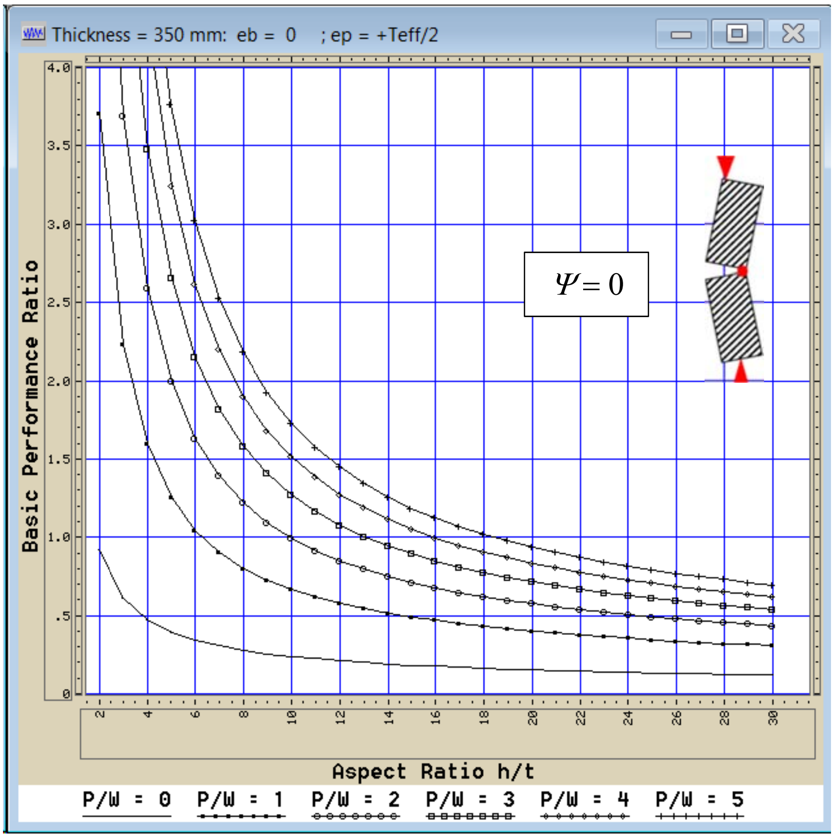


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 350 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 350 mm)

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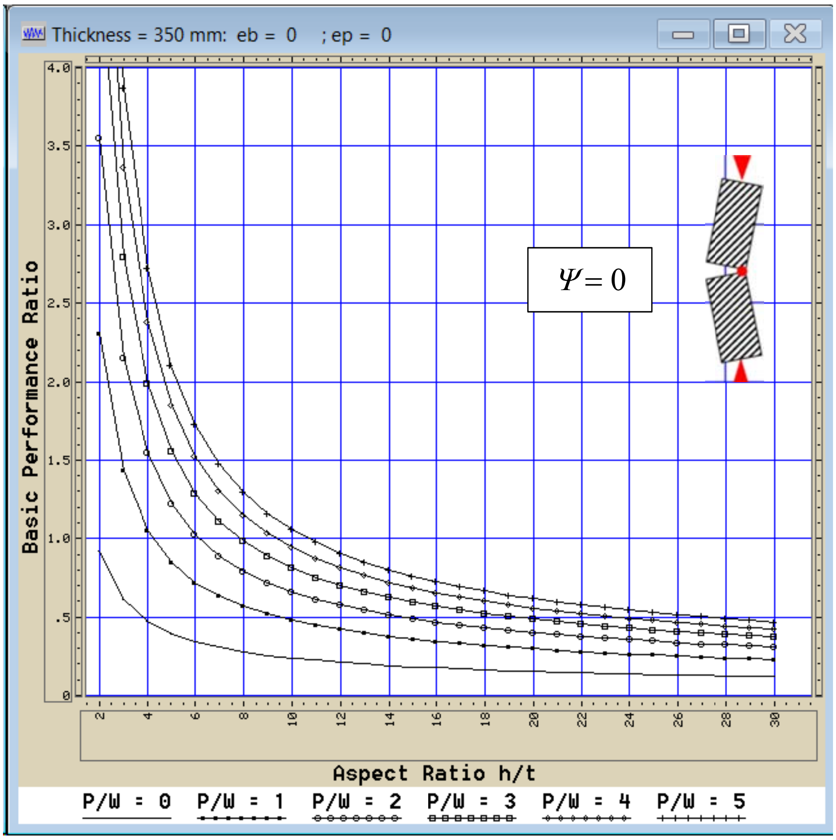
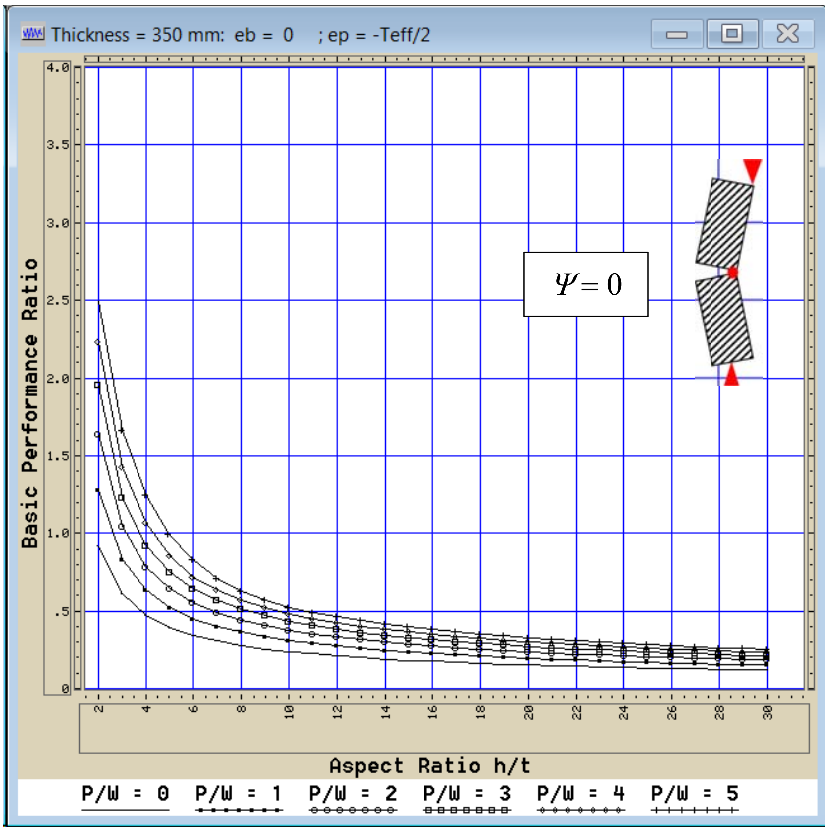


e) For e_b = 0 and e_p = 0 (t_{Gross} = 350 mm)



f) For e_b = 0 and e_p = -*t*/2 (t_{Gross} = 350 mm)

Figure 10C.3: 350 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0$)

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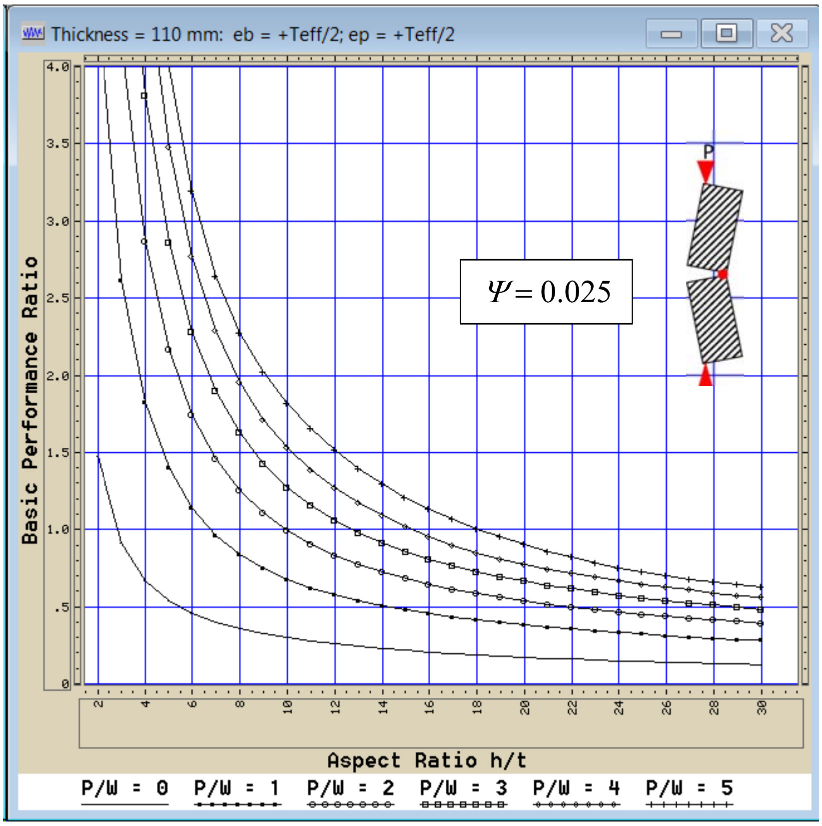
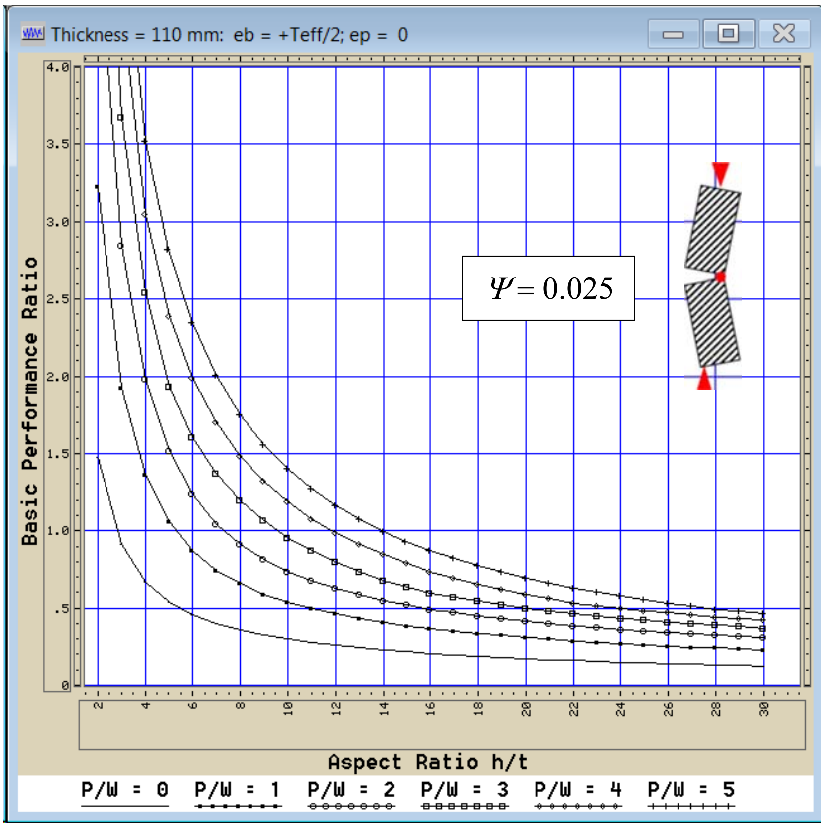


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} = 110 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 110 mm)

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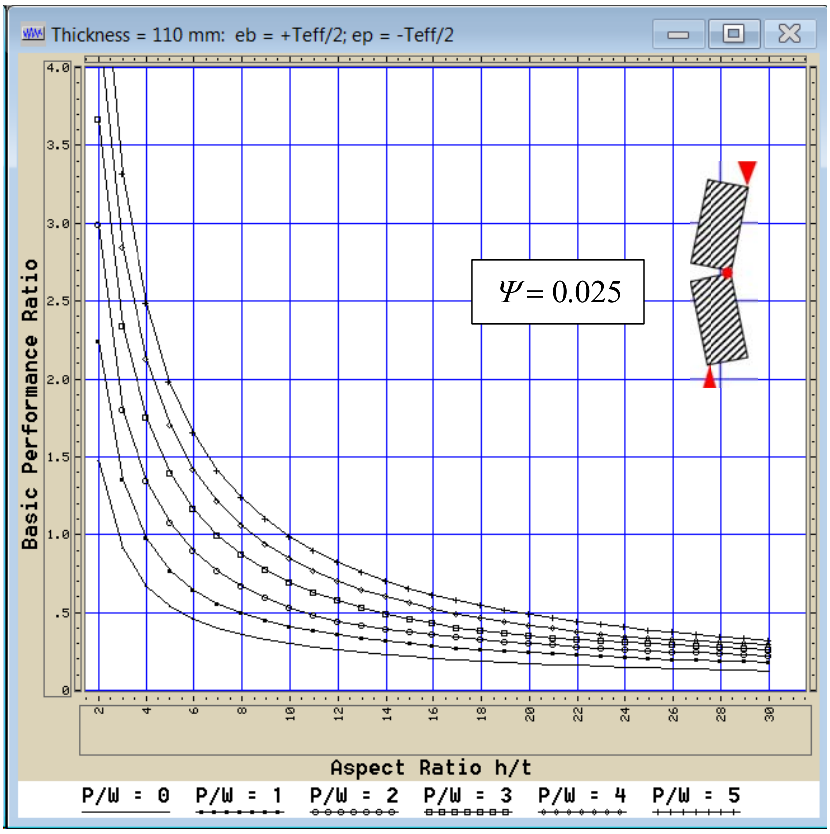
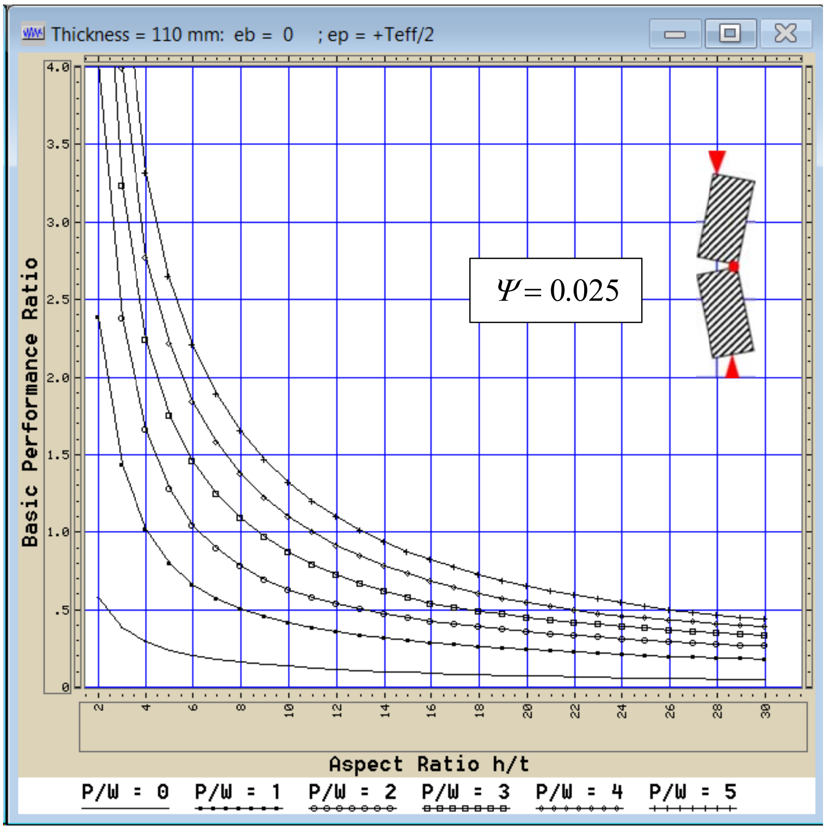


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 110 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 110 mm)

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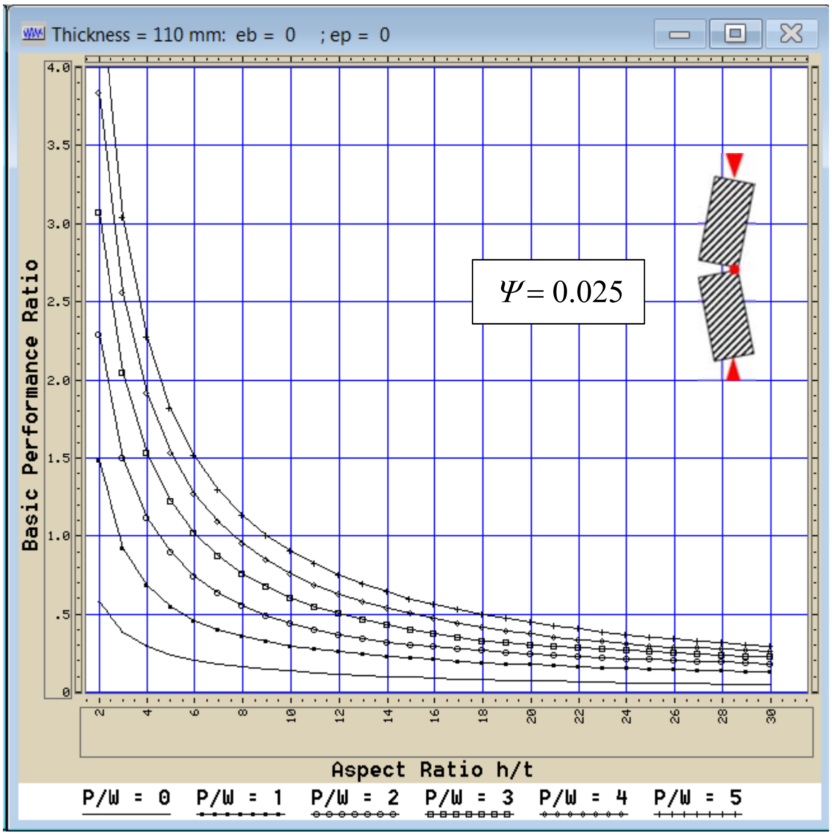
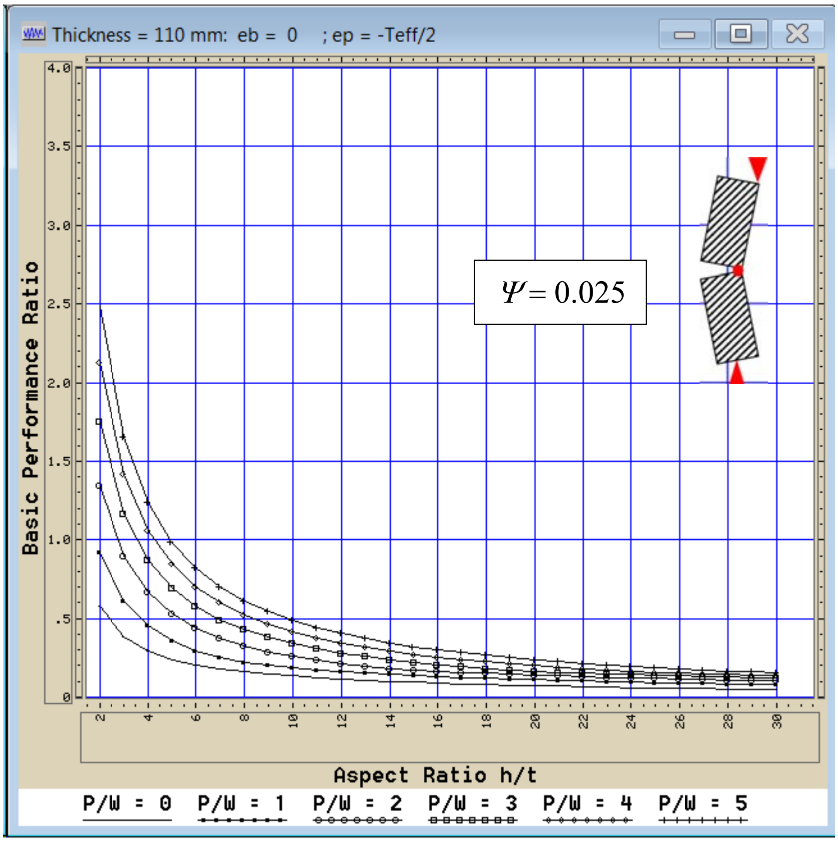


e) For e_b = 0 and e_p = 0 (t_{Gross} = 110 mm)



f) For e_b = 0 and e_p = -t/2 (t_{Gross} = 110 mm)

Figure 10C.4: 110 mm thick one-way vertically spanning face-loaded walls (Ψ = 0.025)

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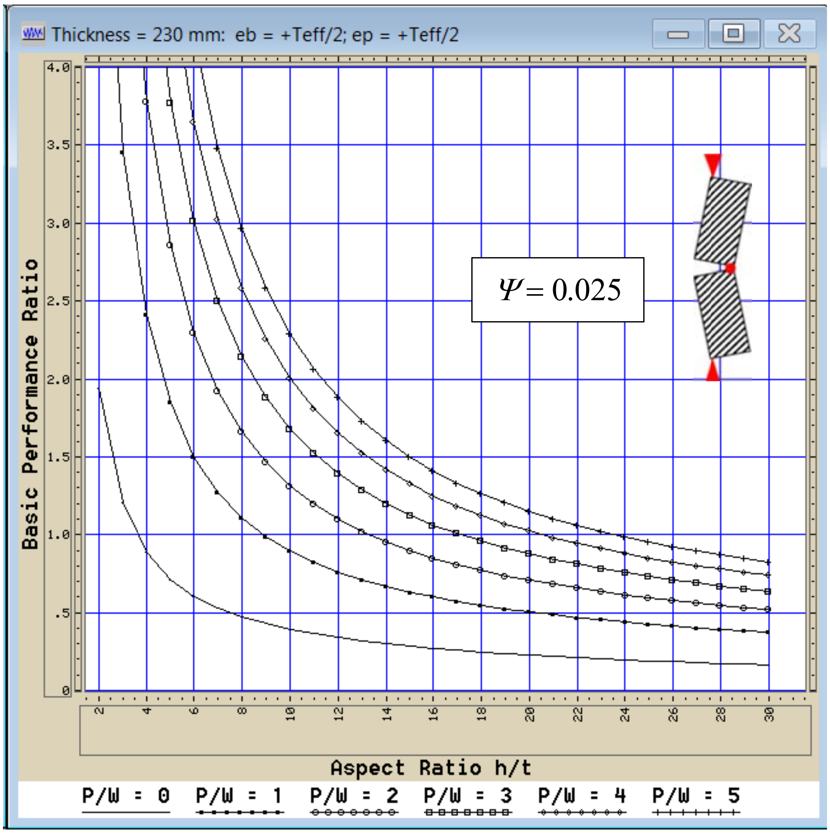
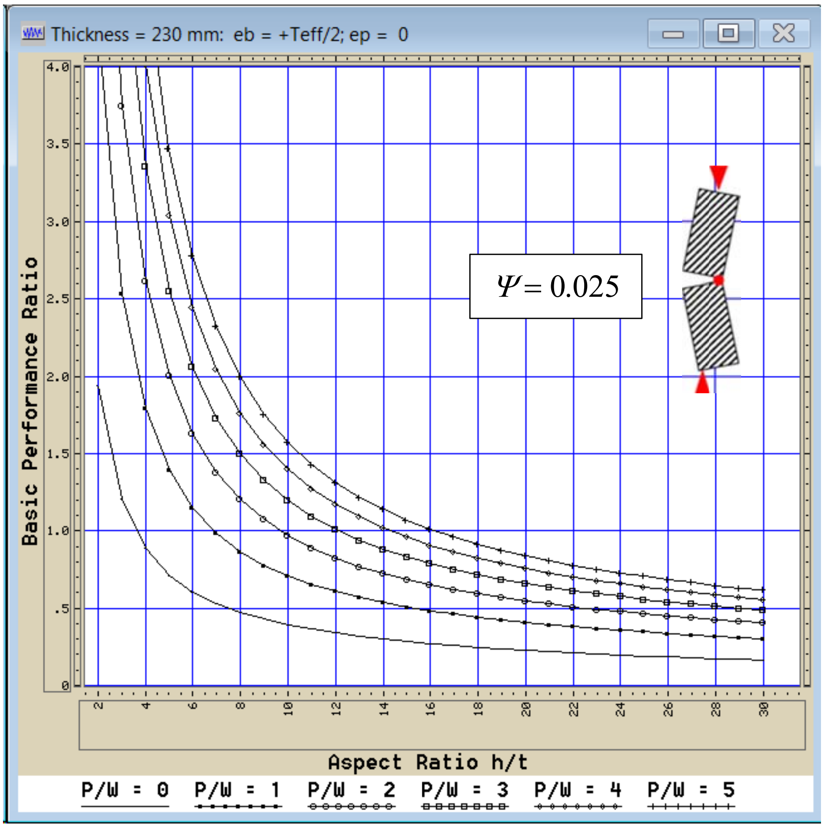


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} =230 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} =230 mm)

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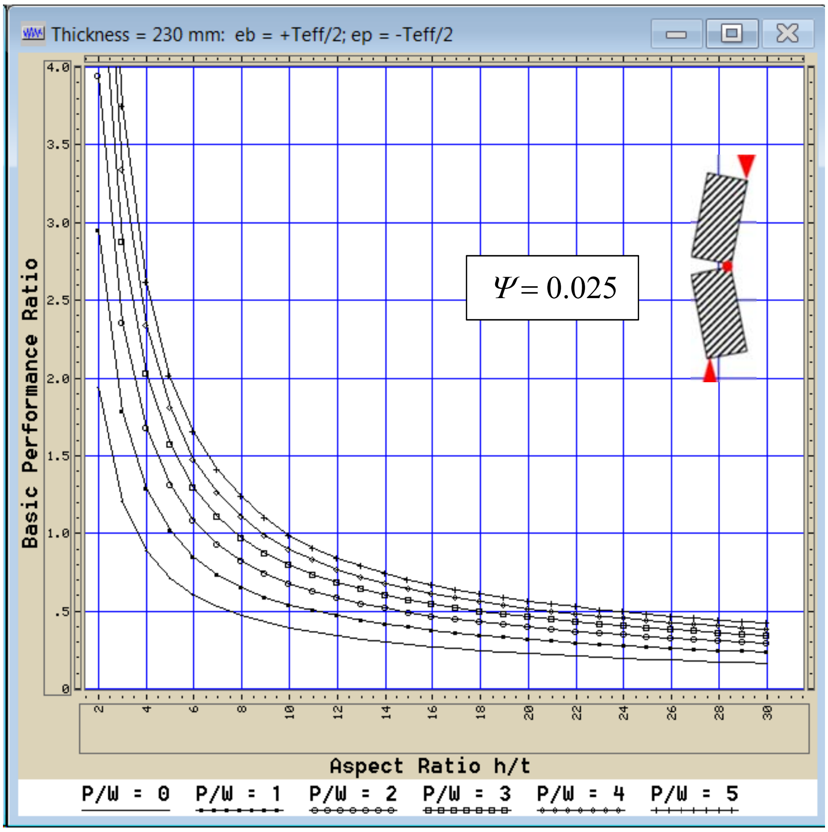
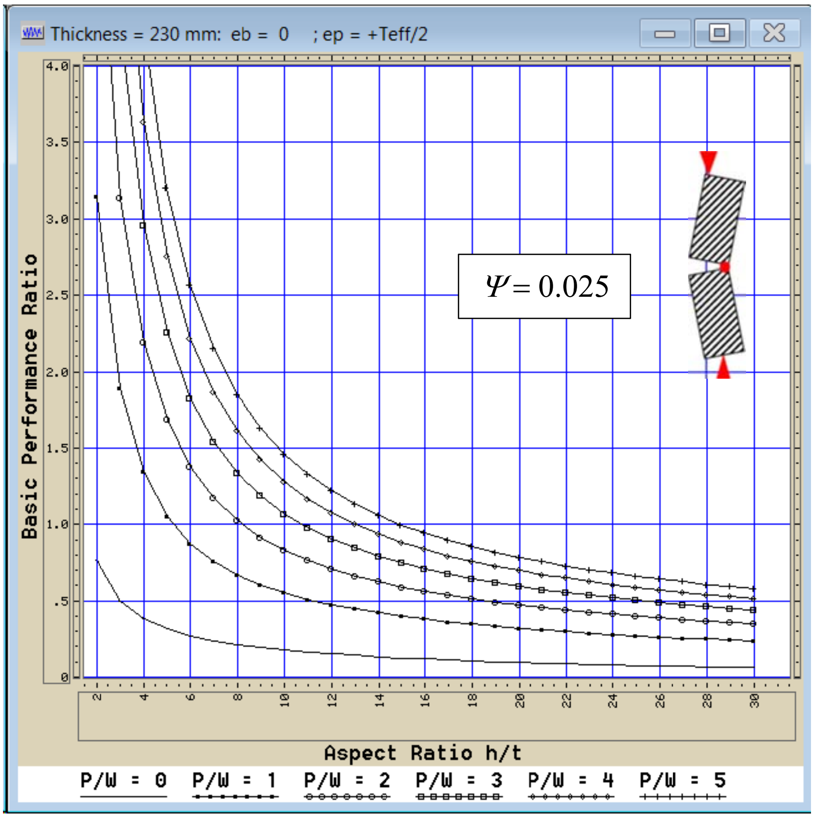


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 230 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 230 mm)

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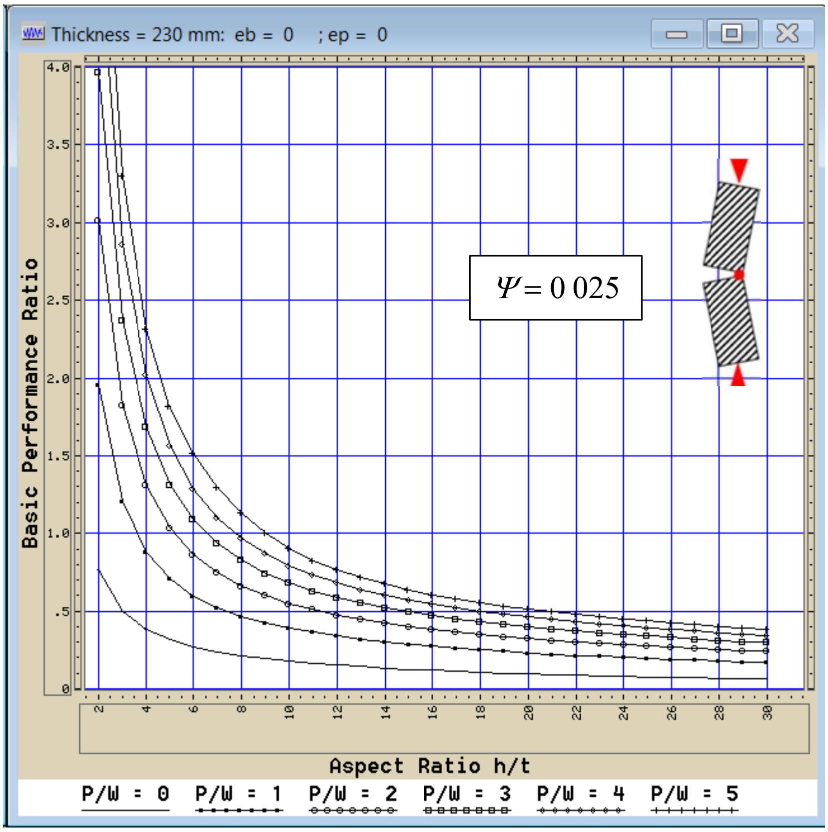
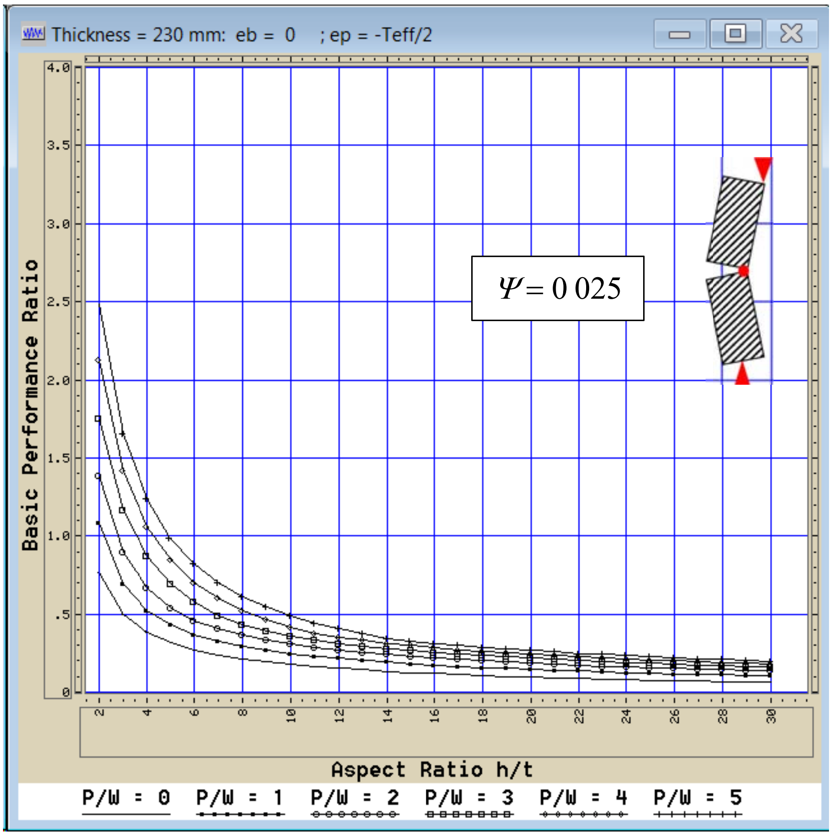


e) For e_b = 0 and e_p = 0 (t_{Gross} = 230 mm)



f) For e_b = 0 and e_p = -t/2 (t_{Gross} = 230 mm)

Figure 10C.5: 230 mm thick one-way vertically spanning face-loaded walls (Ψ = 0.025)

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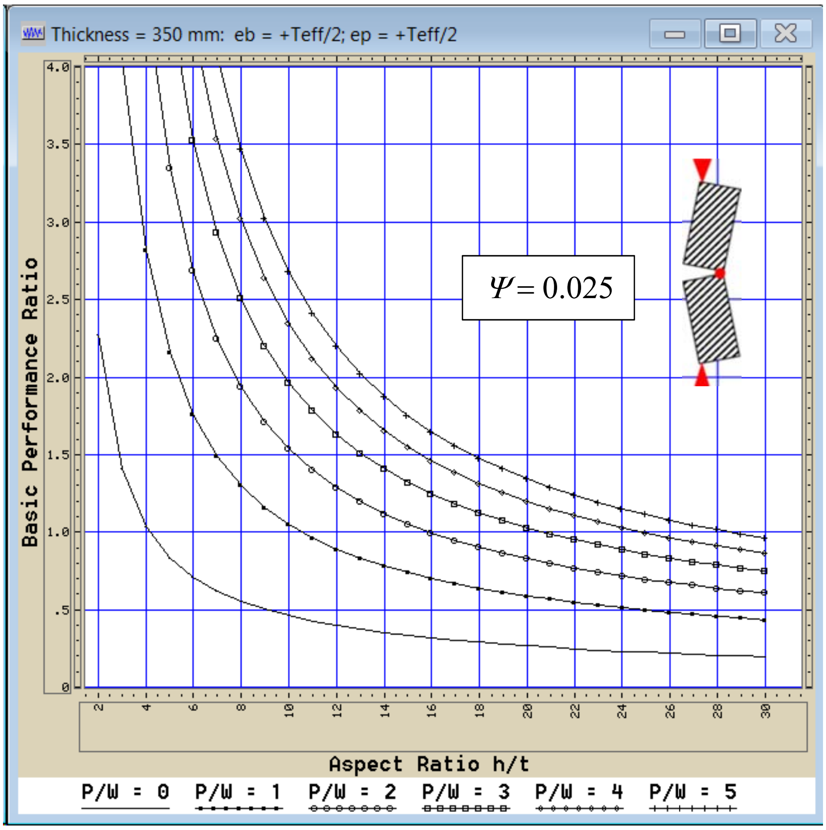
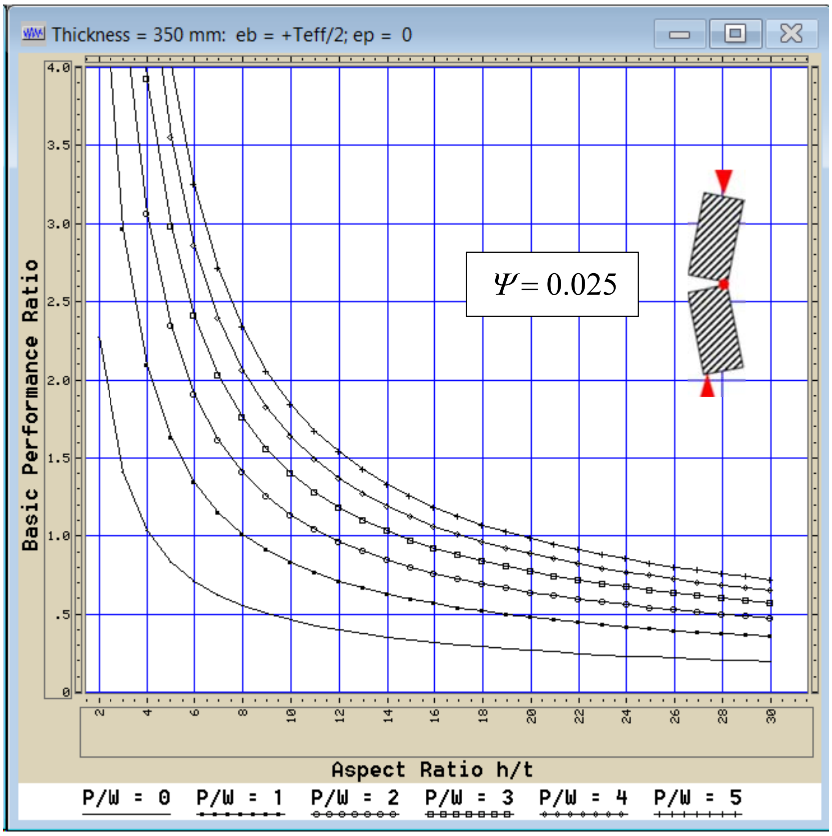


a) For e_b = + *t*/2 and e_p = + *t*/2 (t_{Gross} = 350 mm)



b) For e_b = + *t*/2 and e_p = 0 (t_{Gross} = 350 mm)

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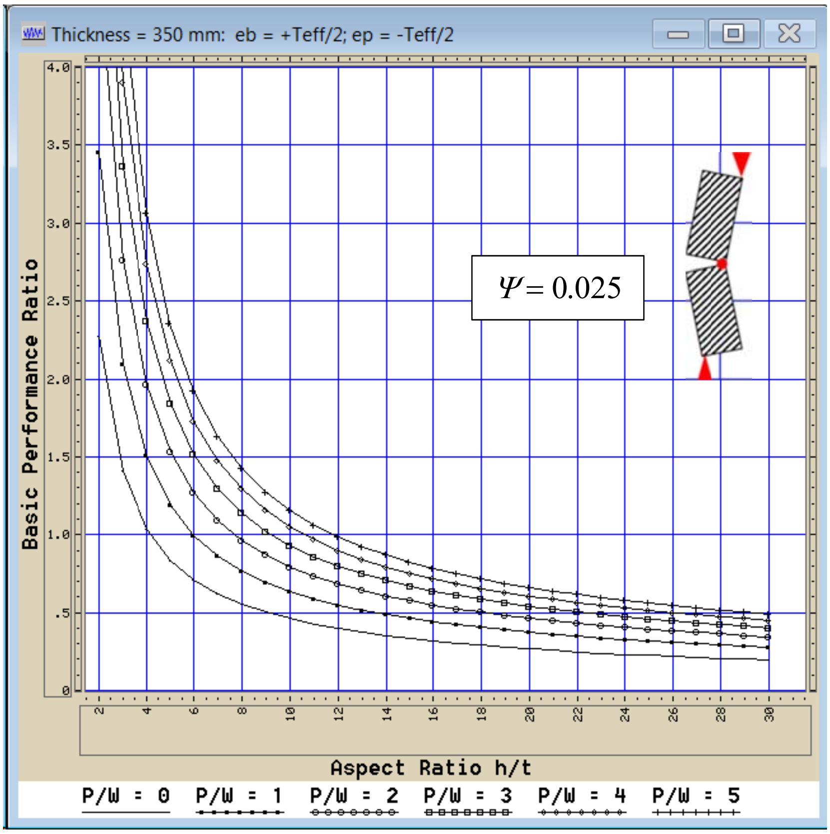
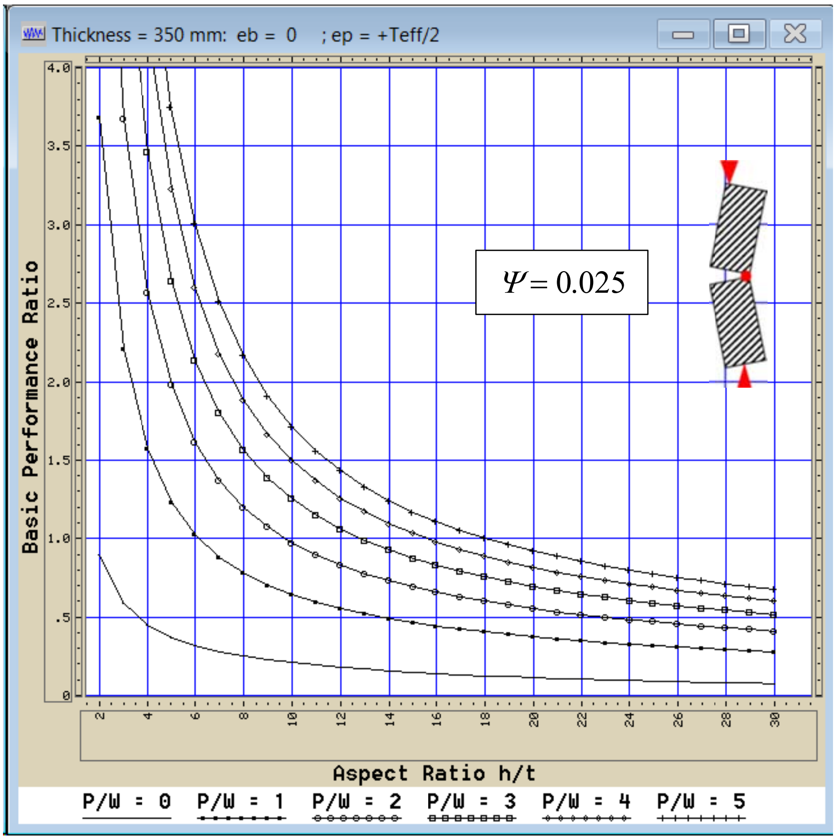


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 350 mm)



d) For e_b = 0 and e_p = t/2 (t_{Gross} = 350 mm)

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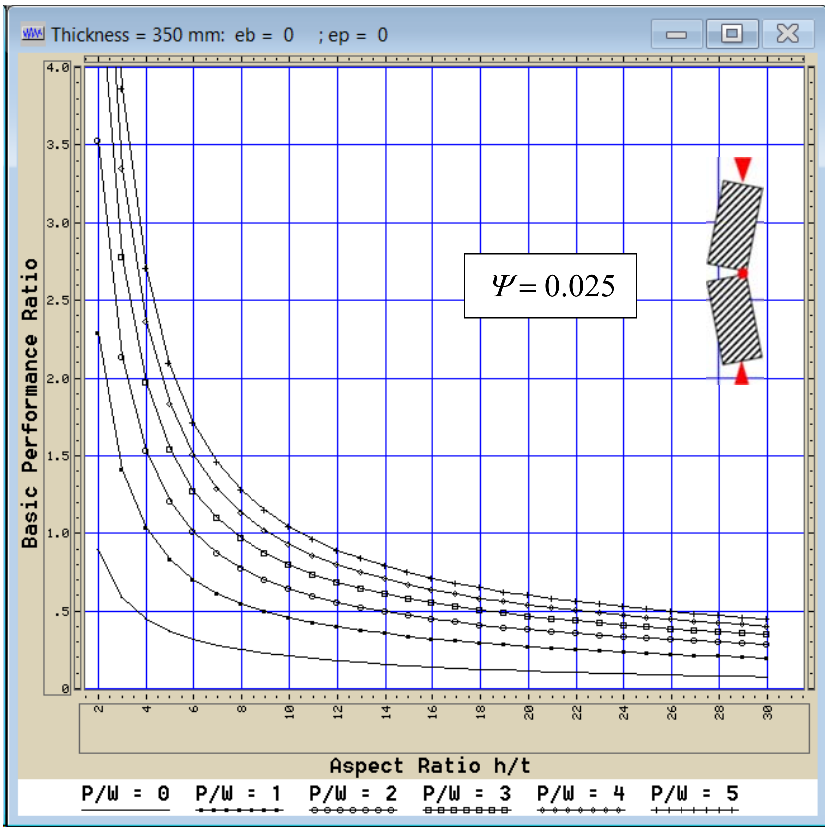
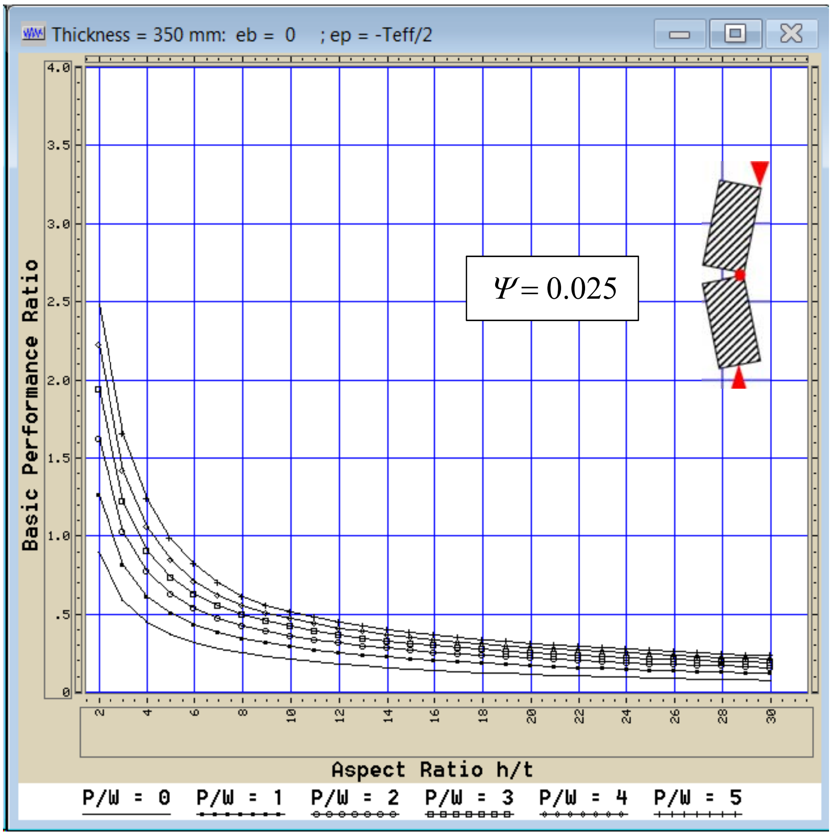


e) For e_b = 0 and e_p = 0 (t_{Gross} = 350 mm)



f) For e_b = 0 and e_p = $-t/2$ (t_{Gross} = 350 mm)

Figure 10C.6: 350 mm thick one-way vertically spanning face-loaded walls (Ψ = 0.025)

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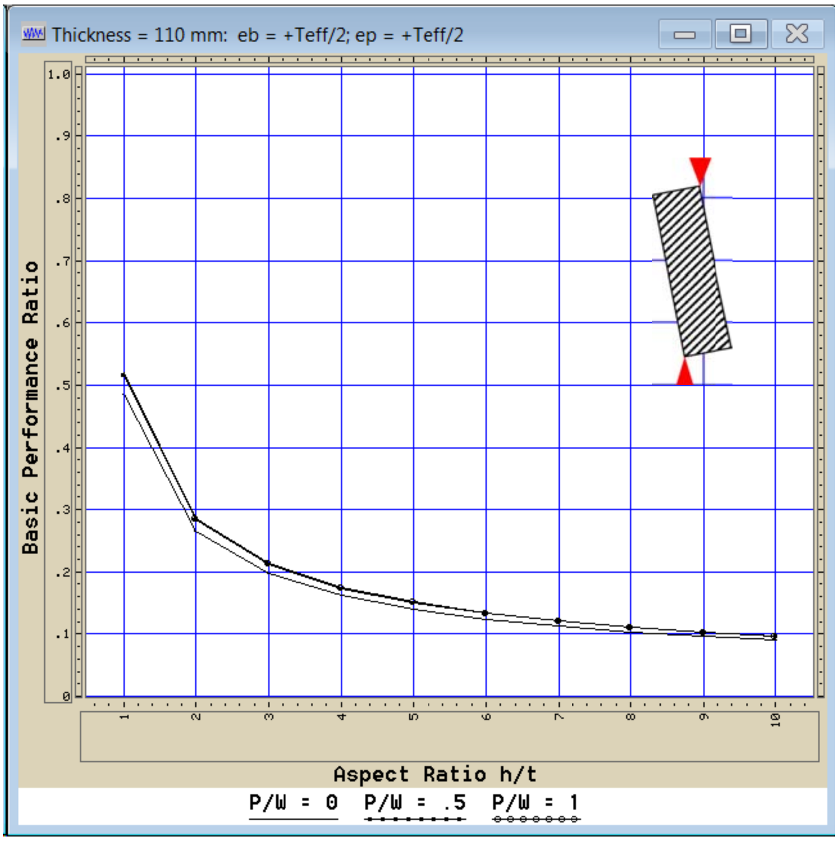
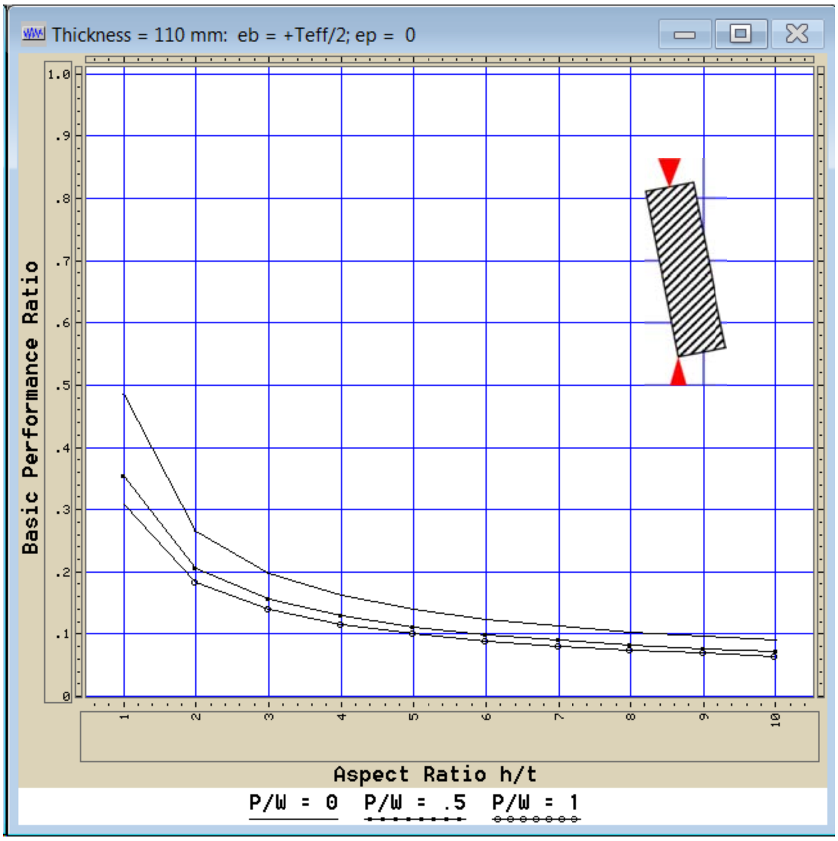


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} = 110 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 110 mm)

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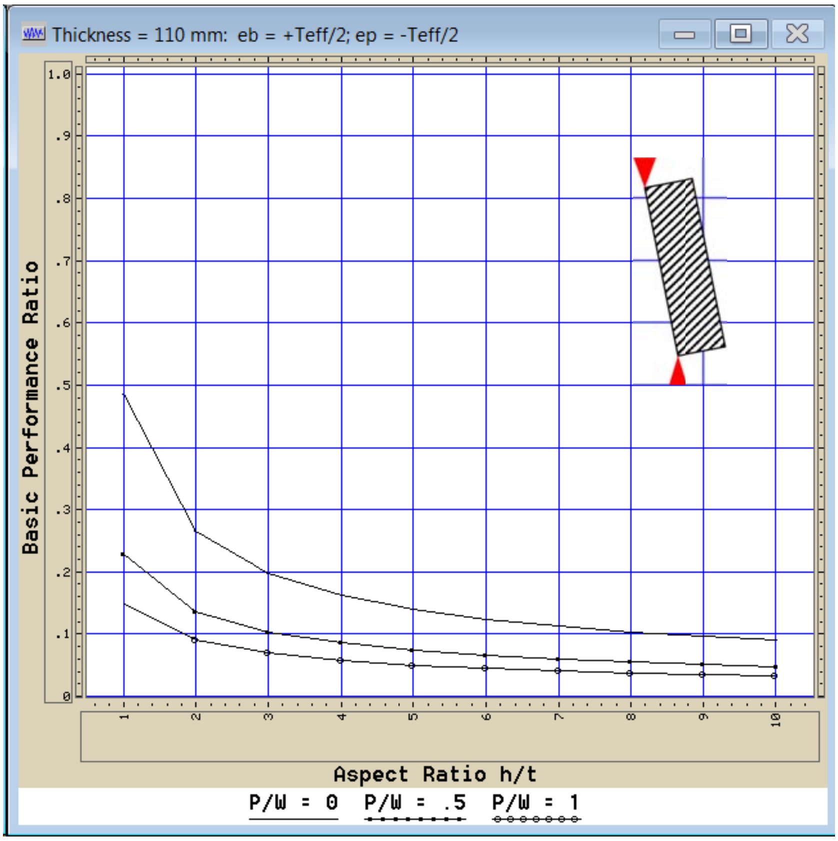
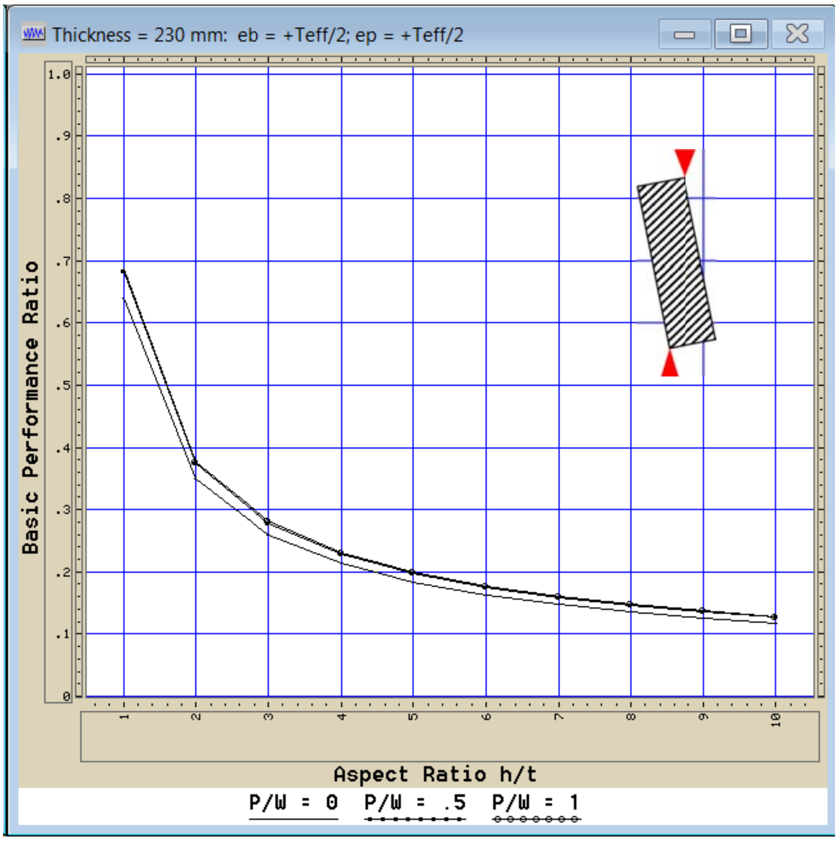


c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 110 mm)

Figure 10C.7: 110 mm thick cantilever wall



a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} = 230 mm)

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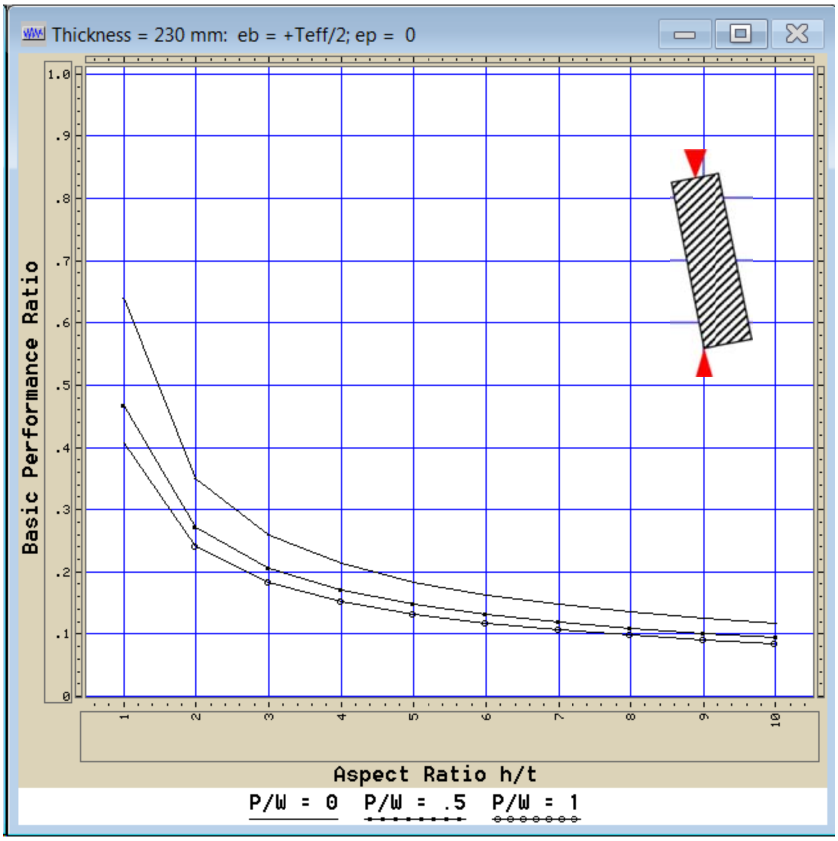
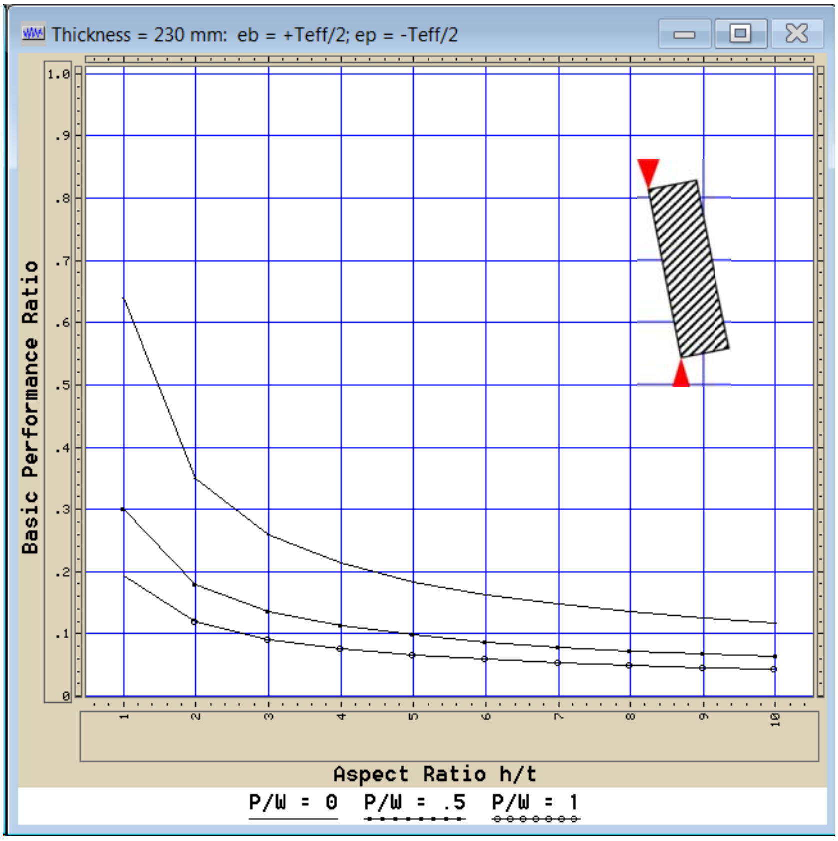


b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 230 mm)



c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 230 mm)

Figure 10C.8: 230 mm thick cantilever wall

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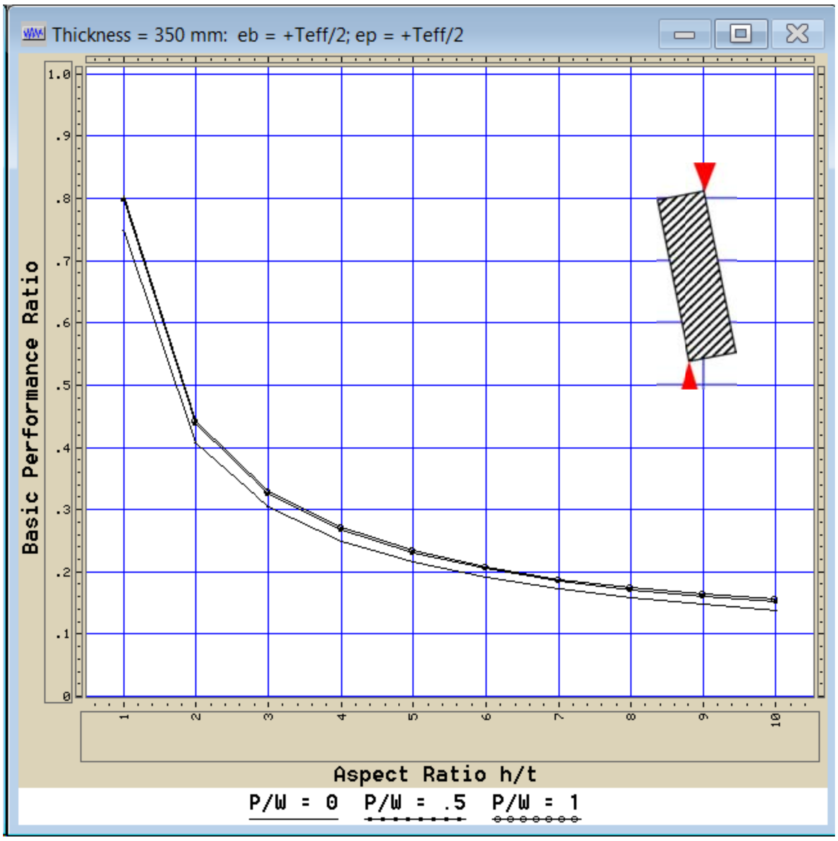
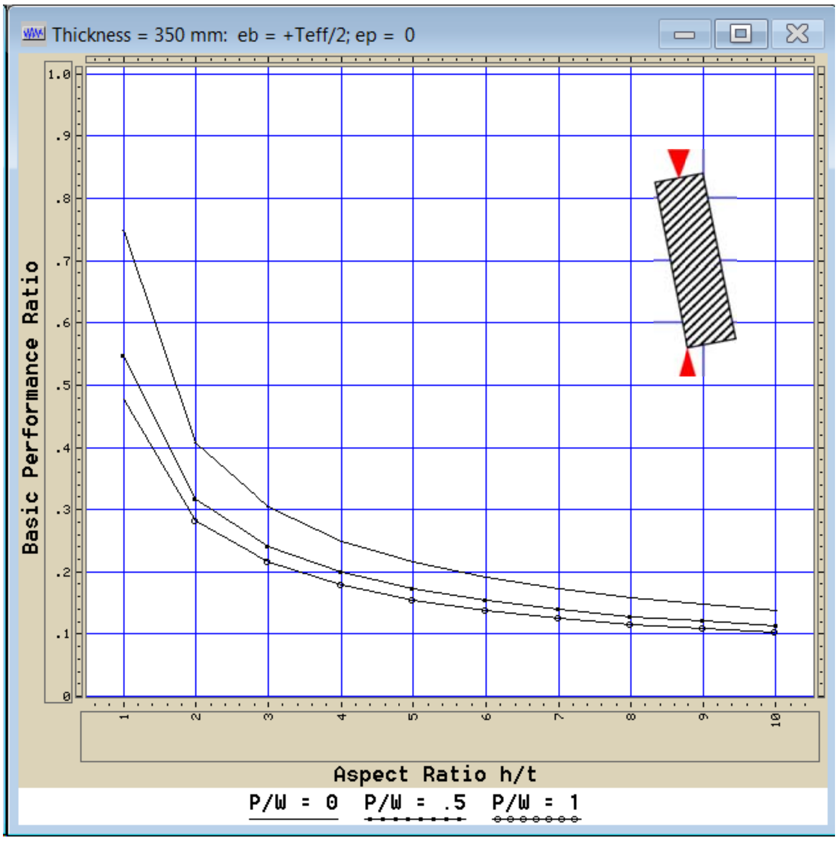


a) For e_b = + t/2 and e_p = + t/2 (t_{Gross} = 350 mm)



b) For e_b = + t/2 and e_p = 0 (t_{Gross} = 350 mm)

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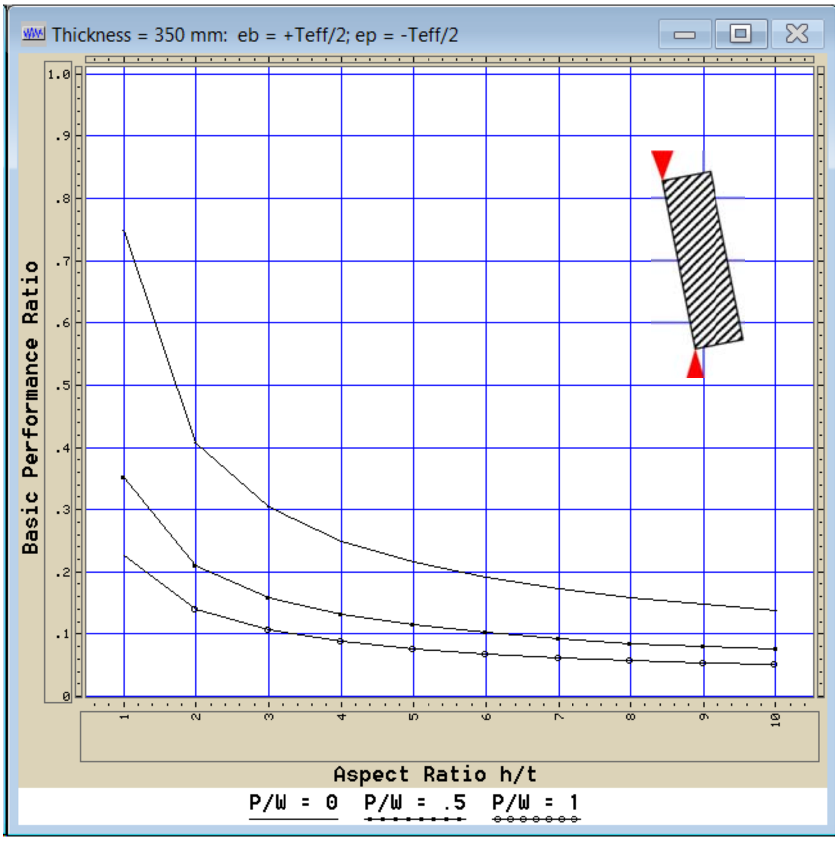

c) For e_b = + t/2 and e_p = -t/2 (t_{Gross} = 350 mm)

Figure 10C.9: 350 mm thick cantilever wall

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New Zealand Society For Earthquake Engineering

Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

Section 14 New Section Geotechnical Considerations

Recommendations of a NZSEE Project Technical Group
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Any comments will be gratefully received.

Please forward any comments to NZSEE Executive Officer at exec@nzsee.org.nz

April 2015

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Section 14 - Geotechnical Considerations

14.1 Introduction

This section of the Guidance provides a commentary on the geotechnical considerations that should be borne in mind in the assessment of a structure's vulnerability to earthquakes. A brief overview of the main topics of interest is provided. A fuller guidance document that expands on the main topics is in preparation (due 2015).

Depending on the site ground conditions and the level of detail of the assessment, the geotechnical input to an assessment may be limited with only the site subsoil class being material to the assessment. This could be the case for ISAs[1].

In many instances, however, the ground and its interactions with the structure, at increasing levels of shaking intensity, can be complex and non-linear, requiring careful consideration, specialist geotechnical advice and close collaboration between the structural and geotechnical engineer during the entire assessment process.

In some cases the structural performance may be driven by geotechnical considerations. Some projects may warrant special studies, including a site-specific seismic hazard assessment.

The early decisions regarding the complexity of the geotechnical assessment that is warranted will be under the influence of the assessing engineer, who will more than likely be a structural engineer. With this comes the responsibility to identify:

- the possible geotechnical issues (and the uncertainties in the assumed geotechnical conditions);
- their potential influence on the seismic performance of the building
- when it is appropriate, if not essential, to seek specialist assistance from an appropriately experienced geotechnical engineer.

Geotechnical hazards that have the potential to significantly affect the performance of a structure that might not be readily apparent to a non-geotechnical engineer include:

- <u>Loss of ground strength and stiffness</u> – liquefaction (sandy soils), cyclic softening (clayey soils)
- <u>Land instability</u> – lateral spreading, rockfall, slope instability (above or below the structure), instability of retaining works
- <u>Other geohazards</u> – e.g. fault rupture, complexities of near-fault effects, tsunami, tectonic movement leading to flood inundation.

Some of these geohazards may propagate from outside the building footprint necessitating a wider view of potential hazard sources, which is essential for a holistic assessment. Such hazards will not affect the *%NBS* score based on the current needs for assessing the structure's rating. However, if a hazard is present it should be reported.

[1] Refer to Section 3 Table IEP-2

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Note:

All structural assessments should include consideration of the influences the ground behaviour can have on structural performance. The level of consideration will be a function of the detail of the assessment and the likely sensitivity of the seismic performance of the building to the geotechnical conditions.

The assessor should recognise that geotechnical support may be required during the ISA for some projects.

Effective assessment of structures starts with effective communication between the client/owner/tenant, the structural engineer and the geotechnical engineer.

A collaborative approach between all parties is essential if the final assessment is to be appropriate for the purpose to which it is to be put.

14.2 Scope

The geotechnical guidance (in preparation) will provide the means for the practitioner to identify the level of influence that ground behaviour may play in a structure's performance during earthquake shaking, and, where possible, to quantify these effects. The guidance will assist in the understanding of the complexity, resources, time and cost that a particular assessment warrants.

It is anticipated the main topics covered will include:

- Interaction between the geotechnical engineer and the structural engineer
- The roles of the geotechnical engineer
- Timing of input from the geotechnical engineer
- Identification of common geohazards
- How to screen geohazards to target the influences that are material to the structural assessment
- Selection of geotechnical design parameters and strength reduction factors
- Assessment and mitigation of geohazards
- Geotechnical reporting
- How to prepare a brief for a geotechnical engineer
- Case studies.

Previous versions of the NZSEE Guidance considered only a few of the potentially significant geotechnical considerations:

- Near fault factor, hazard scaling factor and site subsoil class (Table IEP-2)
- Site characteristics (Section 3.5.6(n) & Table 3A.4).

The guidance given in this section can be considered interim pending completion of a complete section covering the assessment of geohazards and including this in the seismic assessment of a building.

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14.3 The Holistic Engineering System

The soil, foundations and superstructure should be considered as a *holistic engineering system* with each component having the potential to influence the earthquake response and deformation of the other, often adversely and sometimes significantly so. The effects can sometimes be beneficial but any such effects should be cautiously appraised.

Note:

The assessment may identify effects (e.g. kinematic interaction and damping) that could potentially reduce the shaking input to the structure relative to the free-field motion.

However, such effects should be considered with caution. They are subject to on-going research and are not routinely accepted in New Zealand practice as they may lead to non-conservative predictions of structural behaviour. Refer also Section 14.10.

This holistic approach, where we examine and characterise the three distinct, interlinked elements is encompassed in the assessment of *soil-foundation-structure interaction* (SFSI), also referred to as *soil-structure interaction* (SSI), refer to Figure 14.1.

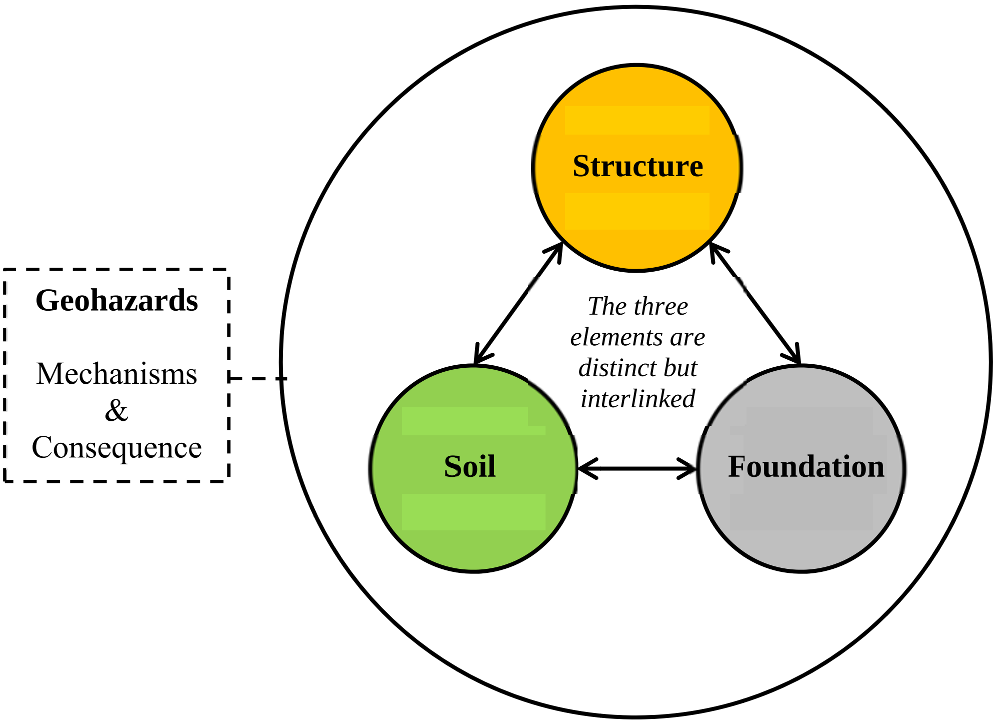

Figure 14.1: Soil-Foundation-Structure Interaction

This simple diagram illustrates:

- that a structural assessment should examine all three primary elements that make up the engineering system
- that the SFSI system is within the sphere of influence of geohazards (their various mechanisms and consequences)

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- that the potential influence of geohazards should be considered in the planning and implementation of the assessment process (commencing in the ISA) - not as later refinements to the commonly used fixed-base structural model
- the interaction needed between engineering disciplines.

It is important to consider SFSI as not just relating to the dynamic interaction of the soil and the foundation with the structure, but also wider aspects such as the response of the system to land instability issues (e.g. lateral spread, slope deformation, etc.).

In this context the term “soil” means the *ground model*. The ground model is a composite of the geological model, the regional seismic hazard model, the geotechnical model, the groundwater regime and the terrain at and around the structure.

14.4 Roles and Responsibilities for Geotechnical Inputs

14.4.1 General

Effective assessment of structures starts with effective communication between the client/owner/tenant, the structural engineer and the geotechnical engineer. A collaborative approach between all parties is essential if the final assessment is to be appropriate for the purpose to which it is to be put.

It is important that there is a common understanding of the expectations, roles and requirements of each team member at the outset of an assessment. Developing an appropriate geotechnical brief in collaboration with the geotechnical engineer is an important step in the assessment process.

Note:

The scope of the geotechnical engineer’s brief should be prepared in active consultation between the geotechnical and structural engineer and be tailored to suit the project’s requirements.

There should be recognition of assessing the issues to the appropriate degree at different stages of a project – the main stages being (1) ISA, (2) DSA and (3) the mitigation/retrofit design phase, as appropriate for the particular project. Table 14.1 outlines the key steps in geohazard identification and at what stage in the assessment process they are likely to require consideration.

The prospect of SFSI assessment tends to immediately lead to expectations of project complexity and time-consuming, sophisticated analysis. It is important to start the assessment with a clear, simple expression of the issues.

The expected output from the definition stage would be a collaborative summary of the structure, foundation and ground vulnerabilities, the likelihood and consequences of their potential compounding interactions, and an outline of the scope of work likely to be needed to complete the assessment.

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At the outset of the project it is very important that the structural engineer is cognisant of potential geohazard influences and makes the client aware of the potential need for and value of geotechnical engineer input at various stages of the project.

An experienced structural engineer will know the process and how best to communicate to the client the staged approach involved and that a particular project may terminate at completion of the ISA, or advance into more detailed work, with or without geotechnical engineering input, depending on the client’s requirements, the characteristics of the structure and the ground conditions.

Table 14.1: Outline of key steps in geohazard identification and assessment

<table>
<tr><th colspan="4">Project phase</th><th rowspan="2">Topic</th></tr>
<tr><th>ISA[1]</th><th colspan="2">DSA</th><th>Mitigation</th></tr>
<tr><td>✓</td><td colspan="2"></td><td></td><td>Site subsoil class & near-fault factor (if relevant).</td></tr>
<tr><td>✓</td><td colspan="2"></td><td></td><td>Identify the geohazards that the site is vulnerable to.</td></tr>
<tr><td>✓</td><td>✓</td><td rowspan="5">Assess across a spectrum of earthquake demand</td><td></td><td>Assess the severity of the earthquake-induced degradation and/or the vulnerability of the site to any step change in its ability to provide foundation support.</td></tr>
<tr><td></td><td>✓</td><td></td><td>Determine the earthquake demand threshold at which degradation may become excessive.</td></tr>
<tr><td></td><td>✓</td><td></td><td>Ground shaking level at which step change may occur. This may include both the level and duration of shaking.</td></tr>
<tr><td></td><td>✓</td><td>✓</td><td>Magnitude of ground deformation (vertical and lateral).</td></tr>
<tr><td></td><td>✓</td><td>✓</td><td>Soil and/or foundation capacity (strength and stiffness) to enable structural modelling, as appropriate.</td></tr>
<tr><td></td><td colspan="2"></td><td>✓</td><td>Appropriate geohazard mitigation measures.</td></tr>
</table>

Note:
1. It is expected that the suitably competent structural assessor leading the project should be able to suitably characterise the geotechnical elements of an ISA without a geotechnical engineer’s input in many cases.

14.4.2 The Structural Engineer

The structural engineer will typically be the professional responsible for conducting a seismic assessment and therefore for recognising when specialist input from a geotechnical engineer is required. This requires a reasonable knowledge of the geotechnical factors likely to influence the seismic performance of the building and site and the ability to recognise when these are likely to be important considerations in the seismic assessment of the building. If the structural engineer is not comfortable that he/she is able to do this then the advice of a geotechnical engineer should be sought during an ISA, and especially for a DSA.

The structural engineer must identify the weaknesses and vulnerabilities of the structure, the load paths and magnitudes of foundation loads. The structural engineer must also

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identify when geotechnical issues are likely to be present that could significantly influence the seismic performance of the building.

The structural engineer will more than likely be the person who recommends when the engagement of a specialist geotechnical advisor is appropriate.

14.4.3 The Geotechnical Engineer

The geotechnical engineer must identify and advise on the degree to which any geohazards could impact directly on the structure's behaviour. In particular, it is important that "step change" behaviour of the ground and foundation support be identified and the consequence of this step-change behaviour be assessed (Clayton et. al. 2014).

Note:

Step change is an abrupt change in the soil's ability to provide foundation support. Some ground conditions can withstand an intensity of seismic shaking for which there is no material change in its ability to support a foundation, or the *degradation* is tolerable over the range of intensity of interest. However, some ground conditions have a threshold at which there is an abrupt (and often severe/intolerable) loss of foundation support, notably ground prone to liquefaction and slopes prone to failure.

Routine investigation methods and analytical tools can only give crude impressions of *where* the threshold may be. Prediction of the *magnitude* of ground and foundation displacement is typically only possible to +/- 100mm, and often with even less precision, particularly for high earthquake demand scenarios, i.e. greater than 500 year return period.

A step change in soil behaviour does not automatically lead to brittle behaviour of the soil-structure system or structural collapse. If there is a step change in soil behaviour leading to a brittle response in the structural performance, however, then it is crucial that the soil-structure interaction is adequately assessed as part of the seismic assessment.

It is often only possible to gain *impressions* of how the ground and foundation may behave, and not obtain absolute values of deformation severity. Accordingly, it will be more relevant to provide a summary of risks and consequences through the spectrum of earthquake demand, rather than attempt to provide quantitative predictions of deformation.

The scope of the geotechnical input required will vary from project to project, notably depending on the degree to which ground performance is likely to govern structural behaviour. The scope of geotechnical-related tasks include the identification and assessment of a suit of topics, some of which potentially have overlap to represent review and refinement as a project progresses.

14.4.4 Level of experience for Geotechnical Engineers

To provide the level of advice and judgments that will often be necessary will require knowledge of the earthquake behaviour of soils and geohazards and the way in which these are likely to interact with and influence the performance of structures.

It is very important, therefore that the advising CPEng geotechnical engineer has relevant experience in this field or has their work reviewed by a suitably experienced and qualified CPEng geotechnical engineer. The geotechnical professional must be competent with

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suitable relevant training and experience in foundation investigations and geotechnical earthquake engineering.

14.5 Assessment Principles

Important considerations for the assessment of the impact of geotechnical issues on an existing structure that may differ to that completed for a new structure include:

- In accordance with the NZ Building Act, the assessment of a building's seismic rating relates only to the geohazards that are present within the site boundary.

Note:

Significant geohazard sources that could impact on a building's seismic rating may be present outside the site boundary. Even though they may not affect the seismic rating for the building, this does not mean that identified damage potential resulting from ground performance or geohazards outside the site boundary should be ignored.

The holistic advice provided as the result of an assessment should also include comment on damage potential of geohazards that have been identified to have the potential to affect the structure but propagate outside the site boundaries and, therefore, do not affect the *%NBS* rating for the building.

- If the ISA identifies any potential geotechnical hazards then the involvement of a geotechnical engineer should be considered to be part of the detailed assessment scoping process.
- The assessment is primarily concerned with the *protection of life-safety* rather than *damage potential* (i.e. to understand the mechanisms that may lead to partial or full collapse of the structure, as it is generally the failure of the structure and/or its parts that will lead to casualties). Damage mitigation is considered to be at the discretion of the building owner and, therefore, does not affect the *%NBS* rating for the building.
- The geotechnical assessment considers the ground's behaviour across a *spectrum of earthquake demand*, not just at the design shaking level (i.e. 500 year return period shaking for importance level 2 buildings defined by NZS 1170.5).

Note:
The *spectrum of earthquake demand* could be modelled by assessing performance at increments of demand, for example as per the range of events given in Table 3.5; NZS 1170.5 *Return period factor*.

- The SFSI and dynamic response of existing structures is likely to be less well understood compared to newly designed buildings.
- Potentially, there may be constraints regarding the availability and/or accuracy of foundation details and subsurface information at and around the structure.
- Recognition that the reliability of the performance predictions of the ground in ever increasing levels of shaking must reduce. The degree of reliability in predicting behaviour of the site for a typical building for 2500 year return period shaking is not required to be the same as for 500 year return period shaking, for example.

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14.6 Managing Uncertainty

We can gain relatively reliable impressions of the behaviour of a structural system at low levels of seismic shaking for which the ground and structure are expected to behave for practical purpose in a more-or-less linear elastic manner. However, as the intensity increases (coupled with increased duration of shaking) we start to lose the ability to reliably model the true behaviour of the system, more so as the soil's non-linear characteristics become more prominent.

At the high end of the seismic demand spectrum where the risk to life is arguably of greatest interest, the complex non-linear behaviour of the soil and the complex interaction of the soil, foundations and structure precludes reliable prediction of the stability of buildings, so conservative assessment coupled with sound judgement are required. The consequences associated with the particular geohazard will typically influence how conservatively the uncertainties should be addressed. There must be a balance struck to avoid being overly-conservative and at the same time keep geotechnical investigation costs to an appropriate level.

It is important that "consistent crudeness" be applied to the modelling and assessment of the holistic engineering system (soil, foundations & structure). Parametric investigations to inform on the sensitivity of performance estimates to assumptions made are likely to be an essential part of the assessment.

Note:

The geotechnical assessment should make due allowance for the sensitivity in the performance estimates due to:

- Variation in the properties of materials (linear and non-linear);
- Variation in the characteristics of the site, and
- Accuracy limitations inherent in the methods used to predict the stability of the ground and foundations.

SFSI assessment often does not warrant in-depth analysis or allow for precision. There is great merit in starting with a simple sketch of the ground model, the foundations and the structural system (including the load paths). When the problem is understood the route to the solution is often a lot clearer.

Note:

It is important to consider SFSI as not just relating to the dynamic interaction of the soil and the foundation with the structure, but also wider aspects such as the response of the system to land instability issues (e.g. lateral spread, slope deformation, etc.).

For important buildings, consideration should be given to using site-specific seismic hazard analysis to better inform on the hazard at the site. The site-specific assessment might also allow consideration of the impact of the contribution of earthquakes of various magnitudes to the hazard where duration of shaking is also considered to be important. Care must be taken to recognise the inherent uncertainties associated with such analyses. Significant departures from Code defined shaking estimates should only be contemplated after careful consideration.

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Consideration should be given to when in the earthquake shaking sequence the structural and geotechnical influences may become significant, and whether or not these interact and compound the threat to the structure. For example, the effects of liquefaction may manifest late in the earthquake shaking sequence or even after shaking has ceased and therefore it may not be appropriate to consider simultaneous application of reduced soil resistance from liquefaction and the full shaking intensity on the building.

14.7 Geotechnical Performance Objectives

The assessment of an existing building for protection of life-safety warrants a different set of acceptable performance criteria and objectives compared to a newly designed building.

In new building design, it is appropriate to adopt a conservative interpretation of geotechnical parameters due to the inevitable uncertainties in ground characterisation and prediction of SFSI behaviour. Such an approach does not generally attract a high cost premium for new buildings. However, a "probable behaviour" mind-set is likely to be more appropriate in seismic assessment of existing buildings.

While non-linearity in foundation behaviour may not be desirable in new structure design where it is also appropriate to limit the risk of damage, some non-linearity in the foundation sub-structure may be acceptable from a life-safety preservation perspective and may be an acceptable mechanism to achieve energy dissipation in an existing building. The concept of geotechnical "failure" being when demand exceeds capacity should be reconsidered, and the concept of what constitutes capacity needs careful consideration.

Therefore, the commonly applied geotechnical performance objectives used in new design may need to be reconsidered. The assessment of an existing structure and its foundation often accepts some degree of nonlinearity, ground deformation and structural damage beyond that of the new design. For example, the assessor would need to ascertain whether a specific foundation behaviour mechanism (e.g. differential settlement or pad uplift) would result in structural instability or loss of gravity-load path in the structure.

The geotechnical assessment should consider both the strength aspects (earthquake loads, bearing strength, etc.) and also the displacement-based aspects (induced lateral displacement, foundation rotation, soil deformation, etc.).

Acceptable performance for geotechnical behaviour should be considered as a function of the *consequence* of the geotechnical induced deformation/loads on the foundation and structural performance, which in turn depends on the particular structure's vulnerabilities and the desired structural performance level. As such, the soil may undergo excessive deformation, but the behaviour of the ground may not necessarily be governing.

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14.8 Assessment of Site Subsoil Class

A crude assessment of site subsoil class can be made via reference to maps of depth to bedrock and the like if these are available. However, site-specific assessment is preferred as maps are typically prepared on a regional-scale basis and can be misleading as has been found in Wellington, for example.

NZS 1170.5 requires that one subsoil class be determined for a new building. By implication for sites where the subsoil varies this would be based on the profile that was likely to give the highest low amplitude site period. Whereas this approach may have little impact on the cost of a new building it can have a significant effect on the seismic rating for an existing building.

Changes to the way in which the influence of ground flexibility has been dealt with in successive codes over the last 20 years means that this influence can be significantly different from that assumed at the time the building was designed. This is especially the case if the subsoil is now characterised as being close to the boundary between previous classes.

Care must therefore be taken when assessing the site soil class for such sites and consideration also given to the way in which the building might respond given the varying ground conditions. In many cases it might be difficult to conceive that the building's response would be adversely affected if a small part of it extended over a softer soil profile especially if the foundation is quite stiff and has the ability to transfer seismic loads in plan. In other cases the lateral difference in soil stiffness might have the potential to result in an amplification of torsional effects in the building which should not be ignored.

Site subsoil classification solely by shear wave velocity for complicated subsoil profiles is problematic and may also miss potential topographic and basin edge amplification effects.

Provided that potential effects from such phenomena, and also the building location relative to the changes in soil stiffness have been accounted for, it is considered reasonable to base the subsoil classification for an existing building on the soil profile that applies for the majority of the site under the building footprint. For the lack of any better advice it is considered that the majority of the building footprint be defined as ≥80% of the footprint area.

This suggested departure from the application of NZS 1170.5 to account for lateral differences in soil stiffness can be considered as synonymous with the departures allowed in the structural assessment to reflect the difference in approach for assessment of existing buildings compared with the design of new buildings.

Assessors are also referred to Larkin & Van Houtte (2014) and Bradley (2015), which together provide an updated commentary on the assessment of site response.

Note:

The rationale for any departures from the simplistic NZS 1170.5 approach must be clearly articulated in the assessment, recognising always the accuracy implied and uncertainties present.

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14.9 Identification and Assessment of Geohazards

The geotechnical influences in a structural assessment can be wide ranging – from limited to complex and severe. The assessor should address the following questions initially via a screening process, conducted using existing information harvested from a site inspection and desk study.

- What are the potential geohazards the site could be impacted by?
- What is the likely severity of the impact if the geohazard occurred, on the building response and on the building as a whole?
- What are the effects of combinations of geohazards?
- Taking into account the initial knowledge of the *whole system* (ground, foundation, structure) what are its vulnerabilities?
- Is a step change in performance expected?
- Is there enough existing information available to satisfactorily answer the above questions?

If the influences are limited or not significant then the ground may be considered to be *competent* in that it provides satisfactory foundation support throughout the spectrum of earthquake demand being considered. Examples of competent ground include: strong rock, dense sand/gravel, and sites not vulnerable to slope instability (mass impact or underslip) or tsunami.

Non-competent ground may be ground that is prone to lose its ability to provide satisfactory foundation support over the spectrum of earthquake demand. Structures can also be vulnerable to loads (impact) from external sources. Loss of foundation support or impact can be due to a number of geohazard mechanisms as summarised in Table 14.2. Mixed foundations can exacerbate the foundation deformation response.

Table 14.2: Summary of potentially damaging mechanisms associated with geohazards

	Mechanism			
Geohazard	**Differential settlement**[1]	**Lateral extension**	**Direct impact of mass**	**Considered in ISA *%NBS* rating?**
Fault rupture[2]	✓	✓	-	Yes, if structure is within influence zone of the main rupture and associated shear zone.
Liquefaction or Cyclic softening (soft clay and plastic silt)	✓	✓	-	Yes
Settlement of non-liquefiable ground	✓	-	-	Check for influence, but unlikely to be an issue.
Low seismic bearing capacity	✓	-	-	Yes

Table 14.2 (cont…)

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Table 14.2 (cont…)

Geohazard	Mechanism			Considered in ISA *%NBS* rating?
	Differential settlement[1]	Lateral extension	Direct impact of mass	
Retaining wall instability[3]	✓	✓	✓	Possible. Depends on wall and site geometry.
Seismic slope instability (underslip)	✓	✓	-	Yes
Seismic slope instability (site inundation by soil/rock)	-	-	✓	No
Discrete rock fall impact (single or multiple boulders)	-	-	✓	No
Tsunami/Dam break (water & debris) including foundation scour	✓	-	✓	No

Note:
1. Including differential settlement and/or foundation rotation.
2. Vulnerability to earthquake-induced tectonic land subsidence and subsequent permanent inundation should also be considered in coastal areas
3. Consider effects of retaining walls upslope and downslope of the site.

Note:

If a hazard is known – report it. Although a geohazard may not influence the *%NBS* rating for an ISA a broader view of all relevant geohazards should be considered when providing advice to building owners.

Refer to ASCE 41 (2014) for a commentary on *Foundations and Geologic Site Hazards*.

14.10 Soil-Foundation-Structure Interaction

Structural engineers have typically adopted a fixed base model for the interface between the structure and the ground. This is based on an assumption that a fixed base translates to a lower first mode of vibration for the structure and a higher lateral load from design spectra than would be obtained if flexibility was introduced at the base. While this may be true in many cases it can lead to an invalid result in others.

For example, over estimating the restraint available at the base of a column founded on shallow pads may provide an erroneous idea of the bending moment profile in the column, and underestimate the deformations in a lateral load mechanism. Equally, assuming a rigid base under a wall may miss the potential for "foundation uplift/wall rocking" and the resulting effects of this.

Perhaps more significant, though, is the potential for the building response, as a whole, to be underestimated due to ignoring a possible resonance effect with the ground that is not sufficiently allowed for by the choice of the specified subsoil classification. Multi-storey buildings located on deep soil sites would be such a case.

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Allowing for soil-foundation-structure interaction can be very complicated and difficult to model. Precision should not be assumed in any assessment of the interaction, but the sensitivity to the expected response of the various assumptions should be understood. The process will require close collaboration between the structural engineer and the geotechnical engineer with each having an understanding of the issues faced by the other.

For assessments of earthquake performance of buildings both the structural and geotechnical engineer must recognise and accommodate the potential for non-linear behaviour of the structure, foundations and the ground. Principles to work by include:

- The ground's behaviour cannot be represented by unique parameter values with uniform distributions (e.g. linear springs).
- With close collaboration, the old fear of possible misinterpretations and abuse of numbers (e.g. spring stiffness, modulus of subgrade reaction) can be significantly reduced and possibly averted.
- An iterative process between structural and geotechnical designers has to be established as soil behaviour is non-linear, spring stiffness depends on load, and load depends on structure (including foundation) stiffness.
- Sensitivity to variations in assumptions should always be checked.

There can be some beneficial influence of soil-foundation structure interaction on a building's life-safety performance (e.g. elongation of building period, concentration of displacement demands in 'ductile' foundation rotation, damping resulting from plastic soil behaviour etc.). However, these beneficial influences are the subject of on-going research. The assessing engineer should be cautioned in adopting the various 'benefits' of SFSI if considering possible mechanisms that may significantly reduce the assumed seismic demands on the structure.

The following are considerations for SFSI modelling:

- Soil structure interaction is modelled directly by soil springs, because the structural model needs to be supported on *something*:
  - The behaviour of springs is predictable and easy to understand.
  - Springs are easy to incorporate into the software most structural engineers use.
  - In a lot of cases structure response is not that sensitive to the spring values used (sensitivity test – 50% to 200% x spring value). If insensitivity is confirmed, this in itself is a useful finding.
- Pinned or fixed supports are not necessarily realistic.
- Load transfer and shearing depends on relative stiffness of both structural elements and supporting ground.
- Multiple load cases to be considered (permanent, temporary, dynamic, different combinations, load factors etc.).
- Serviceability deflections are often critical for the design of new structures but not for the assessment of existing structures. Therefore, bearing capacities capped to limit settlements to meet serviceability conditions are not appropriate for assessments of structures for earthquake life-safety protection.
- Cost and time associated with more rigorous analysis methods.

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14.11 References

ASCE 41 (2014) Seismic evaluation and retrofit of existing buildings (Note: ASCE 41 provides guidance on site characterisation, geohazard mitigation, foundation strength & stiffness, SSI effects, seismic earth pressure and foundation retrofit. Care should be exercised to ensure advice taken from ASCE 41 is compatible with New Zealand practice).

Bradley (2015) Site-specific hazard analysis for geotechnical design in New Zealand. ANZ 2015 conference, Wellington, March 2015.

Clayton, Kam & Beer (2014) Interaction of geotechnical and structural engineering in the seismic assessment of existing buildings. 2014 NZSEE Conference. http://db.nzsee.org.nz/2014/oral/39_Kam.pdf

Larkin & Van Houtte (2014) Determination of site period for NZS 1170.5:2004. NZSEE Bull. Vol. 47, No. 1.

NZS 1170.5 (2004) Earthquake actions – New Zealand.

14.12 Suggested Reading

Boulanger & Idriss (2014). *CPT and SPT based liquefaction triggering procedures*. Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA.

Bray & Dashti (2014) *Liquefaction-induced building movements.* Bull. Earthquake Engineering (2014) 12:1129-1156.

Day (1996) Geotechnical Earthquake Engineering

Kramer (2002) Geotechnical Earthquake Engineering Handbook

NZS 1170.5 Supp 1 (2004) Earthquake actions – New Zealand - Commentary

NZGS Guidelines http://www.nzgs.org/publications/guidelines.htm; including:

- Why Buildings Respond Differently to Earthquakes
- Geotechnical Engineering Practice – Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards. [Under review. Revision due 2015].
- Geotechnical Engineering Practice – Module 2 – Guidelines for earthquake resistant foundation design. [In preparation – due 2015].
- Geotechnical Engineering Practice – Module 3 – Guidelines for seismic design of retaining structures [In preparation – due 2015].

Idriss and Boulanger (2008). *Soil liquefaction during earthquakes*. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA.

Pender (2014) *Integrated design of structure - foundation systems: the current situation and emerging challenges.* 2014 NZSEE Conference. http://db.nzsee.org.nz/2014/keynote/3_Pender.pdf

Refer to GNS Science (www.gns.cri.nz) and MfE (www.mfe.govt.nz) websites for information on active faults and tsunami hazard.

Refer to Christchurch City Council website (www.ccc.govt.nz and link below) for GNS Science reports covering mass movement, rockfall and cliff collapse hazard identification and risk management. The documents relate to the Christchurch Port Hills but provide commentaries on geohazard risk management that are relevant across the wider New Zealand context.

http://www.ccc.govt.nz/homeliving/civildefence/chchearthquake/porthillsgeotech/index.aspx

Note:

Web links and reference documents can change/evolve over time. Reference should always be made to the most recent documents.

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