DESIGN GUIDE

(2007)

SLENDER PRECAST CONCRETE PANELS WITH LOW AXIAL LOAD

G J Beattie

ISSN: 0113 3675

Preface

This document arose from the identified need to provide engineering design guidance for slender reinforced concrete wall panels that had dimensional details that placed them outside the empirical design procedures of the 1995 issue of the Concrete Structures Standard NZS 3101. Time overtook the development of this Design Guide, and as a consequence NZS 3101 was re-issued as NZS 3101:2006 before this Design Guide was completed. The Design Guide has been re-worked to align with the revised Concrete Structures Standard.

Acknowledgments

This project was funded by the Research for Industry Fund of the Foundation for Research, Science and Technology, the Building Research Levy and the Cement and Concrete Association of New Zealand.

Contribution to and review of the Design Guide by Mr Barry Brown of Fraser Thomas Ltd, Mr Dene Cook of the Cement and Concrete Association, Professor John Mander of the University of Canterbury, Dr Barry Davidson of the University of Auckland and Dr Nigel Priestley of the ROSE School, Italy, is gratefully acknowledged.

Note

This Design Guide is intended for engineers involved in the design of slender reinforced concrete wall panels that are typically used as the exterior walls of warehouses and industrial buildings. The panels typically have no connection to adjacent panels, except that they share common supports at their bases and possibly the tops.

Disclaimer

Contents **Page**

Appendices

Tables

Figures **Page**

1. INTRODUCTION

This Design Guide is intended to provide engineers with the tools to make decisions on the design philosophy to be followed and the associated design and construction details for a range of applications of slender precast panels.

There are many variations in the use of slender wall panels. The panels may be:

- designed as load-bearing wall panels in buildings of several storeys (designed as main structural elements)
- fixed to a steel supporting structure as cladding elements only, and not participating in the overall structural system (panels designed as face-loaded parts resisting seismic self-weight and wind actions)
- required to carry structural actions such as in-plane shear or face loads to the foundation (designed as main structural elements)
- cast in long sections which, when in place, span in both the vertical direction either between the foundation and an eaves beam and the horizontal direction between columns or as three side-supported elements with no eaves beam (designed as main structural elements using plate and/or yield line theory).

Occasionally, slender wall panels may be required to operate as simple cantilevers, providing the primary resistance to structural loads at right angles to their faces.

1.1 Wall panels that are covered in this Design Guide

The use of this Guide is currently limited to narrow wall panels of low rise industrial type buildings where the axial loads are low ($\leq 0.015 f'_c.L.t$), where f'_c is the compressive strength of the concrete (MPa), L is the length of the wall (mm) and t is the thickness of the wall (mm). Additionally, the Guide focuses on wall panels that have no support provided along their vertical edges, nor any connection to the adjacent panels. However, wall panels that are supported against out-of-plane displacement at discreet locations along their vertical edges and designed in accordance with this Guide will yield a conservative result.

1.2 Wall panels that are not covered in this Design Guide

The Guide *does not* address wall panels that support flooring, either intermediate over the height of the panel or at the top of the panel, as may be found in multi-storey construction.

The Guide *does not* address wall panels with continuous or discrete vertical edge supports, provided by either cast in-situ columns, concrete buttresses or by steel portal legs. The design philosophy in that case is limited by in-plane shear strength and self-weight and is thus very different from the discrete unsupported panels which are the subject of this Guide.

2. DEFINITION

A slender precast concrete wall is one that has certain physical characteristics which make the type a unique subset of walls.

The predominant characteristic is that it contains only a single layer of reinforcement. It is a requirement of the New Zealand Concrete Structures Standard [32] that for a wall to contain only one layer of reinforcement it must not be thicker than 200 mm (an exception is basement walls 250 mm maximum) (clause 11.3.3)[1] and if it contains a single layer of reinforcement it must not be designed for a ductility factor, μ, greater than 3 (Table 2). Furthermore, clause 11.4.4 limits the sectional curvature ductility at the ultimate limit state to a value less than those given in Table 2.4 of NZS 3101 for a limited ductile plastic hinge region.

The Standard further requires (clause 11.3.11.5) that walls with a vertical reinforcement ratio greater than 0.01 must have the steel enclosed by lateral ties. Slender precast walls are generally designed with a vertical steel percentage less than 1% so that confinement is not required.

The unsupported height-to-thickness ratio is large. This ratio can reach 75 for carefully designed and detailed wall systems, but is more often in the range of 20 to 60.

The panel is not cast in place but is either cast on the adjacent floor slab of the building and lifted into its final position once it has reached an acceptable strength level or, alternatively, the panels are cast on flat beds in precasting yards and transported to the site before being lifted into position. The panel widths can be up to 3.2 m and still be carried vertically on a truck without an over-height permit. By supporting the panels on an angle, the manageable width can be increased further.

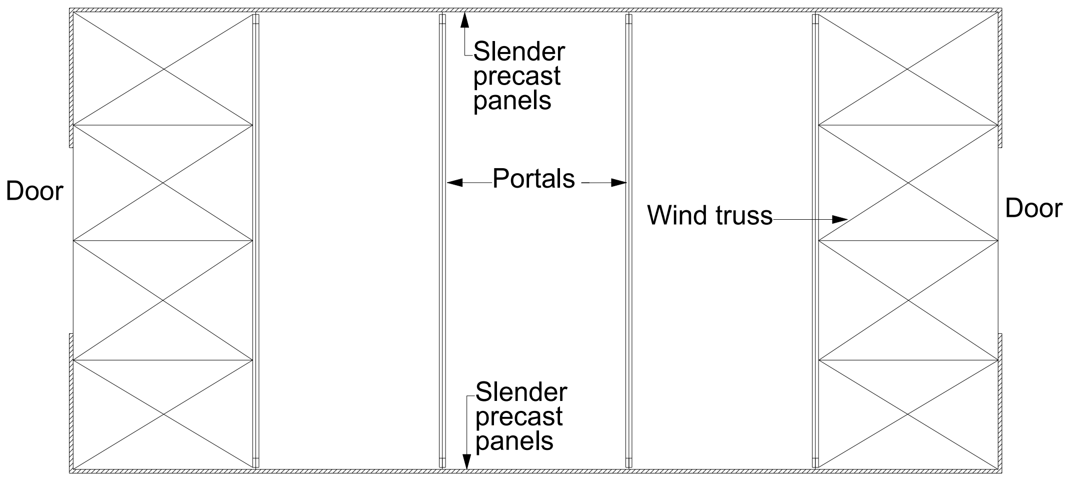
3. SITUATIONS WHERE SLENDER PRECAST PANELS ARE USED

The major use of slender precast panels is for boundary walls in warehouse/factory structures where the eaves height is high (between 6 and 10 m) to provide uninterrupted internal space for palletised storage. The panel shapes will vary from designer to designer depending on the area constraints of the site and the required speed of construction. Often the panels are only required to support their own self-weight or a small portion of the roof structure because the major roof support is provided by portal frames. In these instances, the panels are likely to be required to resist the spread of fire to adjoining properties, provide a weather barrier and resist in-plane shear loads from wind and seismic actions. They will also be required to resist out-of-plane actions from their self-weight. Typical arrangements are presented in Figure 1. A wind truss is generally included in the last bay between the portal and the end wall and this serves to transfer longitudinal loads from the roof plane into the side walls.

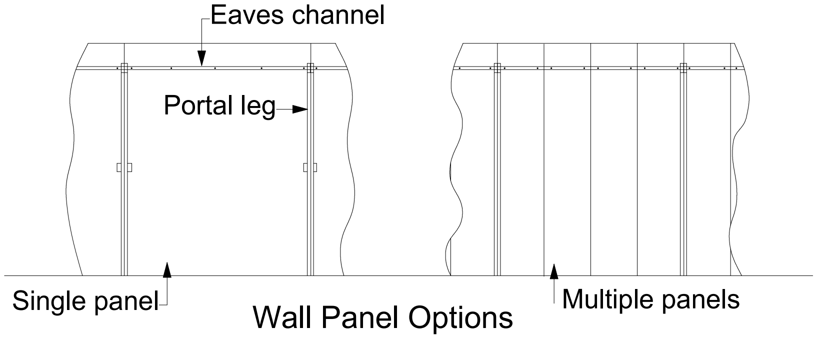
In other cases the panels are required to support the weight of the roof directly as the portals are replaced with rafters that are fixed to the inside face of the panels. The panels are still likely to have a fire-resisting and a weather-resisting function. The wind truss at the eaves will transfer lateral seismic and wind forces to the end walls but the flexibility of the truss will determine how much displacement will occur at the top of the wall panels. End bay wind trusses are still used for transfer of the lateral loads to the side walls. Typical arrangements are presented in Figure 2.

Occasionally, situations occur where one end of the building is open with no opportunity for bracing within that plane. The side wall elements will be required to resist lateral loads in action perpendicular to their faces as cantilevers (Figure 3).

[1] Note that in this Design Guide any references to clauses contained in brackets are referring to clauses in NZS 3101.



Plan of Building with Slender Precast Panels and Portal Frames



Wall Panel Options

Single panel between portals fixed to an eaves beam and possibly to the portal leg. Alternatively, the panel spans horizontally between the portal legs with no eaves connection. Foundation connection may or may not be out-of-plane moment resisting.

Multiple panels spanning between the foundation and the eaves beam (no connection to portal legs). Foundation connection likely to be out-of-plane moment resisting.
Generally no physical connnection between panels.

Elevations of Side Walls

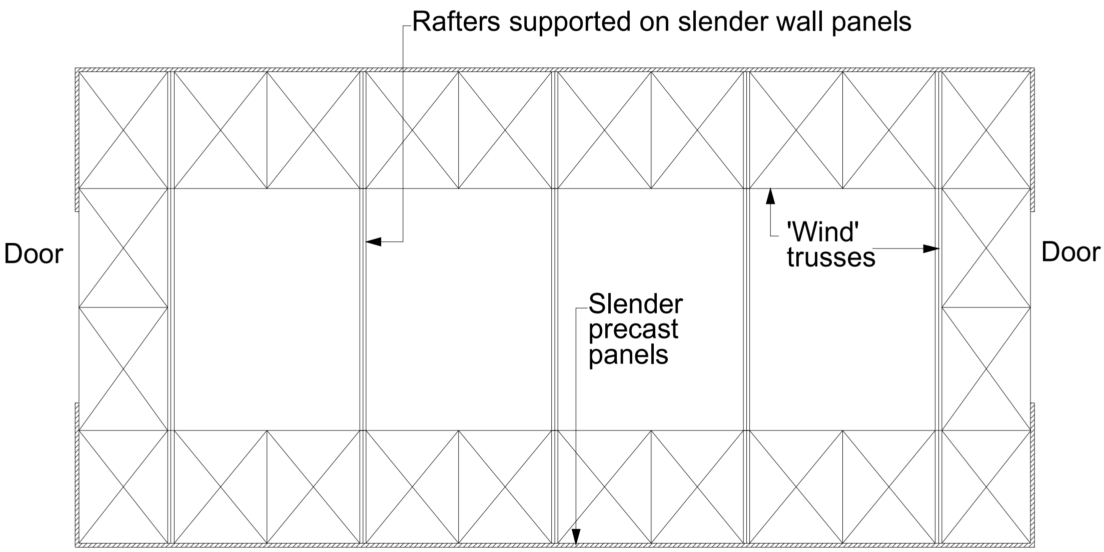
Expected Actions:

1. Transverse wind and seismic forces resisted by portal action, end wall panels resist local forces by in-plane shear/flexure.

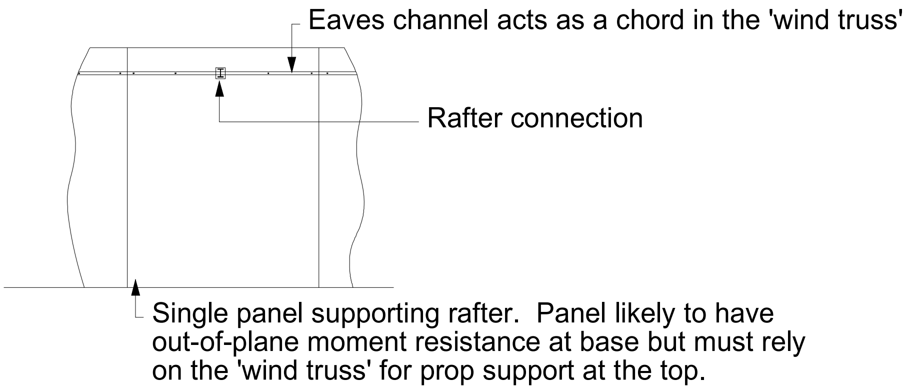
2. Longitudinal wind and seismic forces from the roof and top half of the end walls transferred by 'wind trusses' to the side walls which resist the transferred forces by in-plane shear.

3. Wind truss likely to be flexible if spanning large distances (>20m).

Figure 1. Buildings incorporating slender wall panels and portal frames



Plan of Building with Slender Precast Panels and Rafters



Side Wall Panel

Expected Actions:

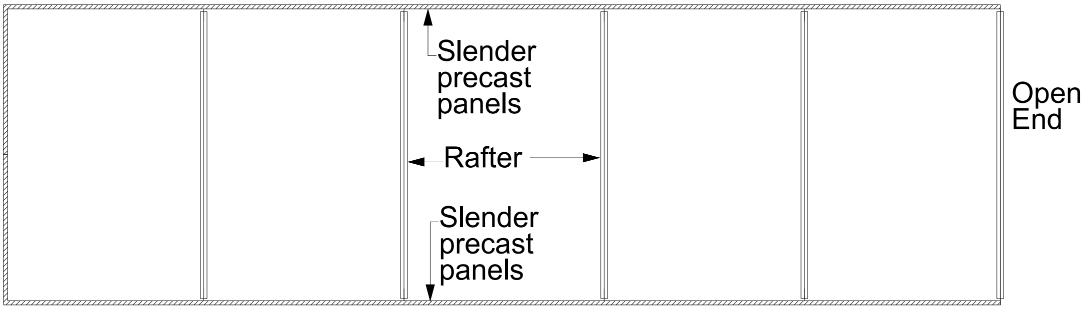
1. Transverse wind and seismic forces on upper half of sidewall panels and roof transferred to longitudinal 'wind truss'.

2. Longitudinal 'wind truss' supported only by end wall panels in in-plane shear/flexure.

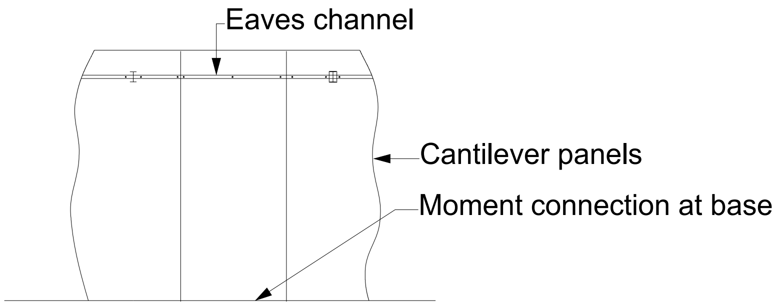
3. Longitudinal wind and seismic forces from the roof and top half of the end walls transferred by transverse 'wind trusses' to the side walls which resist the transferred forces by in-plane shear/flexure.

4. Roof gravity loads supported by side wall panels.

Figure 2. Buildings incorporating slender wall panels which support the roof



Plan of Building with Cantilevered Slender Precast Panels



Elevation of Wall Panels

Elevations of Side Walls

Expected Actions:

1. Transverse wind and seismic forces resisted by cantilever action, left end wall panels may resist local forces by in-plane shear/flexure.

2. Longitudinal wind and seismic forces from the roof and top half of the end wall transferred to the side walls via roof. Side walls resist the transferred forces by in-plane shear.

Figure 3. Buildings with an open end and out-of-plane loaded cantilever panels

4. STRUCTURAL LIMITS FOR SLENDER PANELS

The limits on the use of slender precast panels are based on the design philosophy being employed. Essentially, as the assumed ductility of the system increases then so does the need for better detailing. Table 2.5 of NZS 3101 limits the design of walls with a single layer of reinforcement loaded in-plane to a ductility of 3. If singly reinforced walls are to be designed for ductility in earthquakes, limits are placed on the sectional curvature ductility (clause 11.4.4). These are given in Table 2.4 of the Standard for limited ductile plastic regions as $15\gamma\alpha_{fy}$, where γ is the nominal first yield curvature and α_{fy} is a multiplier based on the yield strength of the steel.

Many individual parameter limits cannot be treated in isolation in the determination of the strength of the panel. For example, the axial load alone cannot be used without allowance for its position with respect to the centreline of the panel.

4.1 Panel thickness

NZS 3101 does not allow walls to be less than 100 mm thick (clause 11.3.2) and, unless two layers of reinforcement are used, the maximum thickness is 200 mm (250 mm for basement walls) (clause 11.3.3). This Guide is limited to walls with a single layer of reinforcement.

Under the additional design requirements for earthquake effects in NZS 3101, clause 11.4.2.1 contains requirements to safeguard against premature out-of-plane buckling in potential plastic hinge regions of thin walls more than two storeys high. Equation 11-20 of NZS 3101 is used to determine the minimum thickness for both doubly and singly reinforced walls. The α_r factor in the equation effectively increases the minimum thickness for singly reinforced panels by 25% over that required for doubly reinforced panels. For singly reinforced panels designed for a ductility up to 3 and with a maximum steel percentage of 1%, the minimum thickness can vary from as low as 100 mm for a 5 m high wall with a height-to-length (*H/L*) ratio of 1 to 200 mm for a 15 m high wall that is 5 m long.

4.2 Concrete strength

The 28 day minimum concrete compressive strength specified in clause 5.2.1 of NZS 3101 is 25 MPa and the maximum is 100 MPa, although ductile and limited ductile elements shall not have a strength greater than 70 MPa (clause 5.2.1). However, other provisions of the Standard may well govern the minimum required strength. For example, exposure classification B2, which incorporates large areas near the sea in main centres (see NZS 3101 for exact locations), will require a minimum specified compressive strength of 30 MPa (Table 3.6) combined with a minimum cover of 45 mm. In exposure classification B1, the minimum specified compressive strength may be 25 MPa and the cover may reduce to 40 mm.

The minimums specified in the Standard are never likely to be tested because construction requirements such as early age stripping and lifting will dictate that the strength must be greater than the minimums in the Standard. Advice from the precasting industry [39] indicates that for panels that are to be cast in a yard and shipped to site the generally specified strength is 45 MPa to allow early stripping, but for panels that are cast on site and tilted into position the commonly specified strength is 30 MPa.

4.3 Reinforcing steel strength

Two main reinforcement characteristic strengths are available – 500 MPa and 300 MPa. Grade 500E and 300E bars may be used. NZS 3101 does **NOT** support the use of "L" grade bars, and "N" grade bars may **ONLY** be used in non-seismic situations. The greater characteristic strength has the benefit that a lesser amount of steel is required to achieve the desired section strength, but with this benefit comes the disadvantage that lap and development lengths must be longer because the lap length is related to the yield strength of the steel. This is particularly important at the base of the wall where starter bars are sometimes grouted into ducts in the panel.

Grade 300E or 500E reinforcement may be successfully bent once as long as the bars are bent around the correctly sized mandrel. It is **NOT** acceptable to bend bars by threading a steel pipe over the end of the bar and bending with no former. For bars up to 20 mm diameter, the minimum bend radius is 2.5 times the diameter of the bar. Bars should not be re-bent if at all possible and designers must take account of this when planning the steel detailing for construction in their panel designs. NZS 3109 [51] places limits on which bar types can be re-bent. Grade 300E bars may be re-bent once. Grade 500 quenched and self-tempered bars must **NOT** be straightened or re-bent. Grade 500E micro-alloyed bars may be re-bent, but only when heated in accordance with NZS 3109. If re-bending appears to be unavoidable, consideration should be given to the use of proprietary threaded anchors so that steel can be installed once the panel is in its final position.

4.4 Reinforcing bar size, bar spacing and reinforcement ratio

NZS 3101 requires the minimum bar size for walls to be 10 mm and the maximum diameter is limited to 1/7 of the wall thickness (clause 11.3.11.2). However, in potential plastic hinge regions, this upper limit is reduced to 1/8 (clause 11.4.5). In both directions, the bar spacing must be a maximum of three times the thickness of the wall or 450 mm, whichever is the smaller.

NZS 3101 places limits on the reinforcement ratio for walls as follows:

4.4.1 Horizontal reinforcement

The minimum requirements for horizontal shear steel are contained in Equation 11.19 of NZS 3101. Manipulation of this equation results in the minimum horizontal steel percentage being $0.7/f_y$, where f_y is the nominal yield strength of the horizontal bars.

4.4.2 Vertical reinforcement

The vertical steel percentage is determined by the specified concrete strength and the bar strength. The minimum percentages for combinations of concrete and steel strength are given in Table 1.

The minimum percentage is calculated from $\sqrt{f'_c}\ /4f_y$ and the maximum from $16/f_y$ (clause 11.3.11.3). For actions causing bending about the weak axis of the panel, the area of reinforcement is required to be such that at every section the distance from the extreme compression fibre to the neutral axis is less than 0.75 times the distance in the balanced strain condition.

Clause 11.3.10.3.8 (d) requires the minimum vertical reinforcement ratio to be greater than $0.7/f_y$, where f_y is the nominal yield strength of the vertical bars. However, this requirement will always be overridden by those in Table 1.

Clause 11.3.11.5 of NZS 3101 requires the vertical reinforcement to be enclosed with lateral ties if the percentage is greater than 1%. Because of the difficulty involved in providing confinement to a single layer of steel, designers are very likely to elect to keep the vertical steel content to less than 1%, which will mean that steel yielding will occur under face loading well before any concrete compression failure.

Table 1 Minimum vertical steel percentages

Concrete strength (MPa)	Steel strength (MPa)	Minimum percentage	Maximum percentage
25	300	0.41	5.3
30	300	0.46	5.3
35	300	0.49	5.3
45	300	0.56	5.3
25	500	0.25	3.2
30	500	0.27	3.2
35	500	0.30	3.2
45	500	0.34	3.2

4.5 Development of horizontal reinforcement

Horizontal steel in walls expected to be subjected to earthquake effects, which covers all slender precast walls which are required to resist seismic in-plane loads, is required to be adequately anchored at the wall edges and this may be achieved with standard hooks as close to the end of the wall as possible (NZS 3101 clause 11.4.6.2).

4.6 Axial load in combination with out-of-plane moment (due to face load)

When portal frames are used in conjunction with the slender wall panels, the panels will essentially have only their self-weight to support, in combination with any face load moments. The axial load is taken as the applied axial load plus the weight of the panel above the mid-point between its supports.

NZS 3101 (clause 11.3.5.1.2) provides an equation for adjusting the design moment, M^*, at the mid-point of the panel between its supports and checking this against the strength of the section, φM_n (i.e. $\varphi M_n > M^*$). This equation is repeated as equation 1 but with terms consistent with this Guide and modification to match typically reinforced slender panels.

$$M^* = \frac{M_a^*}{1 - \left(\frac{5PH^2}{(0.75)48 E_c I_{cr}} \right)}$$ **Equation 1**

where
- P = design axial load on the wall at the ultimate limit state
- E_c = modulus of elasticity of the concrete
- E_s = modulus of elasticity of the steel
- H = the height of the panel between supports
- M_a^* = moment at the mid-height of the section due to the applied loads
- I_{cr} = cracked section moment of inertia of the panel about its weak axis
- = $nA_{se}(d-kd)2 + Lkd^3/3$
- n = the modular ratio, E_s/E_c
- A_{se} = $(P + A_s f_y)/f_y$
- d = the distance from the compression edge to the reinforcing steel
- L = the length of the wall panel

k is determined by elastic theory

Occasionally, walls will be expected to behave as pure cantilevers, as in buildings with one or two wide-end openings. In this situation, the effective height will be twice the height of the panel and in Equation 1 the "*H*" term will be twice the actual height of the wall.

5. NON-STRUCTURAL LIMITS ON THE USE OF SLENDER PANELS

5.1 Fire-resisting requirements

5.1.1 Existing Building Code and Standards provisions

Clause C of the New Zealand Building Code (NZBC) [41] requires that adjacent household units and other property be protected from fire spread by thermal radiation or structural collapse and that fire-fighting personnel are protected. In industrial buildings, the concrete wall panels must not collapse outwards, thus endangering the lives of fire fighters. On the property boundary, they also must not collapse outwards and damage adjacent property.

The commentary to NZS 3101 [36] notes that while the requirement for external free-standing cantilever walls having to resist 0.5 kPa face load has been removed from the Loadings Standard [53], it has been included in NZS 3101 for the design of connections to ensure a degree of robustness for these types of buildings, both during and after the fire. It further notes that walls connected to a very weak or flexible roof structure will need to be designed as cantilevers from the base during a fire because the weak roof will not have sufficient strength to prevent outward movement of the wall panel. In lieu of undertaking calculations to determine the effects of thermal bowing, the designer may elect to design the wall to resist a face loading of 0.5 kPa.

NZS 3101 contains specific requirements for the connections between the wall panels and the roof structure to prevent outward collapse in fire (clause 4.8). The connections must be able to resist the largest of:

(a) for all walls, the force resulting from a face load of 0.5 kPa

(b) for walls connected to a flexible unprotected steel structure, the force required to develop the ultimate bending moment of the wall at its base.

To allow for the reduced capacity in fire conditions, it further states that the fixings in the panel must be designed using 30% of the yield strength of the steel in ambient conditions, components made from other types of steel must be designed using the mechanical properties of the steel at 680°C, and proprietary inserts must be designed for a minimum fire-resistance rating of at least 60 minutes for unsprinklered buildings and 30 minutes for sprinklered buildings.

Because of the essential continued performance of the connections, limits have been placed on the types of inserts used. Proprietary cast-in or drilled-in inserts must have an approved fire-resistance rating or otherwise be anchored into the wall by steel reinforcement (or be fixed to the reinforcement) and if adhesive anchors are used they are required to have an approved fire-resistance rating.

5.1.2 Portal structures

Warehouse structures with portal frames close to the boundary and supporting the roof will generally require fire rating of the column elements of the portal to prevent collapse within the designated fire-resistance period and to support the exterior wall elements once the portal rafter has collapsed under the heat of the fire. In these situations, the integrity of the connections between the panels and the eaves beams must remain intact if the wall panels are required to perform as either propped cantilevers or simply supported between the foundation and the eave. If the connection is not strong enough, the wall must be designed to ensure that it will not fail outwards in the fire.

Detailed procedures for the design of slender panels for fire are provided in reference [52]. This document considers the effects of fire on the curvature of the wall panels and includes for the P-Δ effects that result from either axial loads on curved panels and/or panel self-weight on curved cantilever panels.

5.1.3 Rafters supported by wall panels

When the only gravity support for the roof is provided by the wall panels, the panels must be fire rated as primary structural elements. Two fire behaviour scenarios are possible [46].

While the panels are propped by the braced roof in service, at the time of a fire, the bracing may well become ineffective and a base cantilever-resisting mechanism will be required to keep them upright. Depending on the size and location of the fire, as the steel roof collapses in the fire, significant flexural and axial loads may be introduced to the connection between the rafter and the panel (Figure 4):

- If the connection capacity is inadequate, the rafter may detach from the panel and the panel is required to support itself as a cantilever. In this situation, if the fire was particularly fierce and near to the panels, the thermal gradient through the panel would cause it to bow outwards and collapse could occur.

- If the connection has a capacity in excess of the design loads, the wall panel will be pulled into the building as the rafters sag in the fire. The position of the plastic yielding of the rafter under the heat of the fire will determine the level of flexural load introduced to the panel and this could be sufficient to damage the panel locally at the rafter connection.

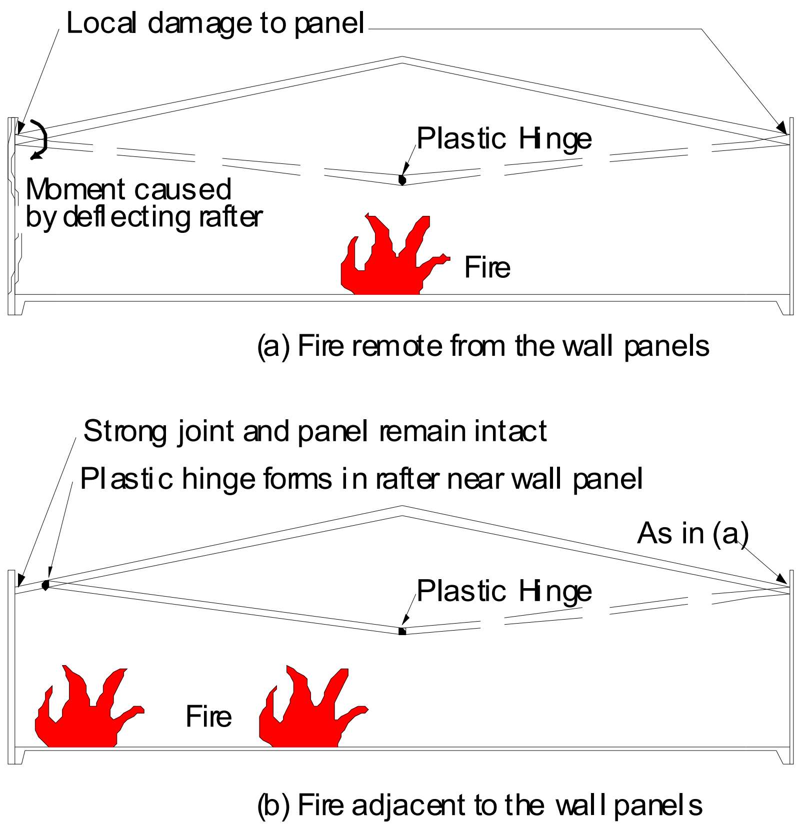


Figure 4. Effects of fire location on the possible behaviour of the rafter-panel connection

Further information on design for fire resistance may be found in reference [52].

5.2 Atmospheric temperature effects

It has been observed that slender walls can bow markedly under the effect of direct sun on the outside surface in combination with cooler inside temperatures. If one panel is subjected to a temperature differential caused by the sun, while an adjacent panel is shaded by other buildings, then differential movement between the panels can occur.

The coefficients of thermal expansion of concrete and steel are similar and about 0.0117 mm/m/°C. A temperature of 20° above ambient on the outside surface will cause the outside surface to lengthen by 0.23 mm/m. For a panel pinned at the top and bottom and subjected to a stable 20°C temperature differential through the wall thickness, this lengthening will cause the wall panel to curve outwards (Figure 5). The determination of the maximum outward displacement, Δ, is as follows:

The internal radius of curvature, $R = 1/\psi = (t / (\alpha_T * \Delta T)$
Therefore, $\psi = (\alpha_T * \Delta T)/t$

where	α_T	=	coefficient of thermal expansion
	ψ	=	curvature
	ΔT	=	temperature differential through the wall
	t	=	thickness of the wall

Therefore, $\Delta = \psi H^2/8$, where H is the height of the wall.

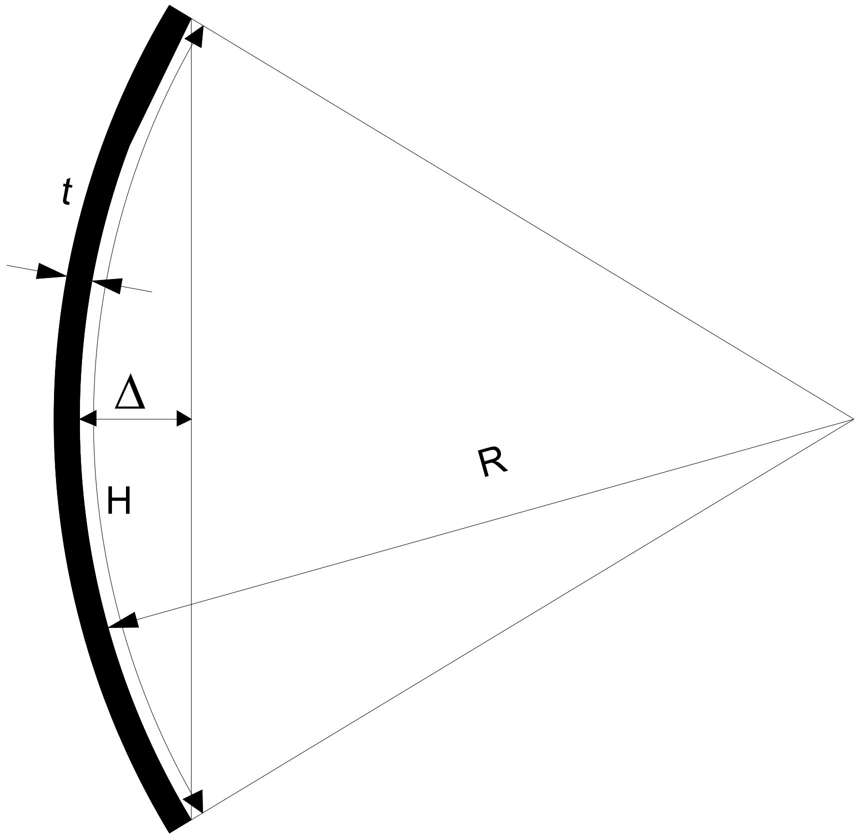


Figure 5. Deflection due to temperature differential across the section

The temperature gradient through the wall will not necessarily be linear when subjected to the direct heat of the sun. This is because there will be a lag as the concrete absorbs the heat on the outside faster than it will transfer it to the inner part of the wall. However, after several hours the gradient approaches a linear relationship.

Consider a panel with a height-to-thickness ratio of 60 and overall height of 9 m (thickness 150 mm) exposed to an outside air temperature of 30°C and with an inside air temperature of 20°C. After five hours the temperature gradient through the wall will be close to linear with an outside temperature of 28°C and 22°C on the inside face. Under these conditions the wall will deflect at its mid-height by about 5 mm. Direct sunlight on a panel will cause a greater curvature than just different inside and outside air temperatures. This may cause significantly different displacements between the two panels.

For panels that are not physically connected over their height, consideration must be given to the type of weatherproofing that is needed to accommodate the differential movement between panels. A simple backed sealant as shown in Figure 6 may not be able to accommodate the movement and the addition of a metal sliding flashing may be necessary. Welded plate connections between panels will constrain the panels to the same plane but this may introduce local secondary stresses that are sufficient to cause panel damage.

When the base of the panel is fixed against rotation, the panel will exhibit double curvature as it wants to curve outwards but is restrained from doing so. The deflected shape is as shown in Figure 7. The bending moments in the panel are determined by adding the restoring moments for a both-ends-fixed panel and the release moments for the pinned top (Figure 7).

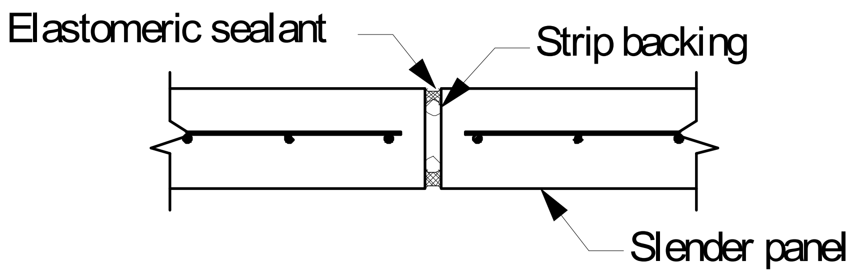


Figure 6. Flexible sealant joint between panels

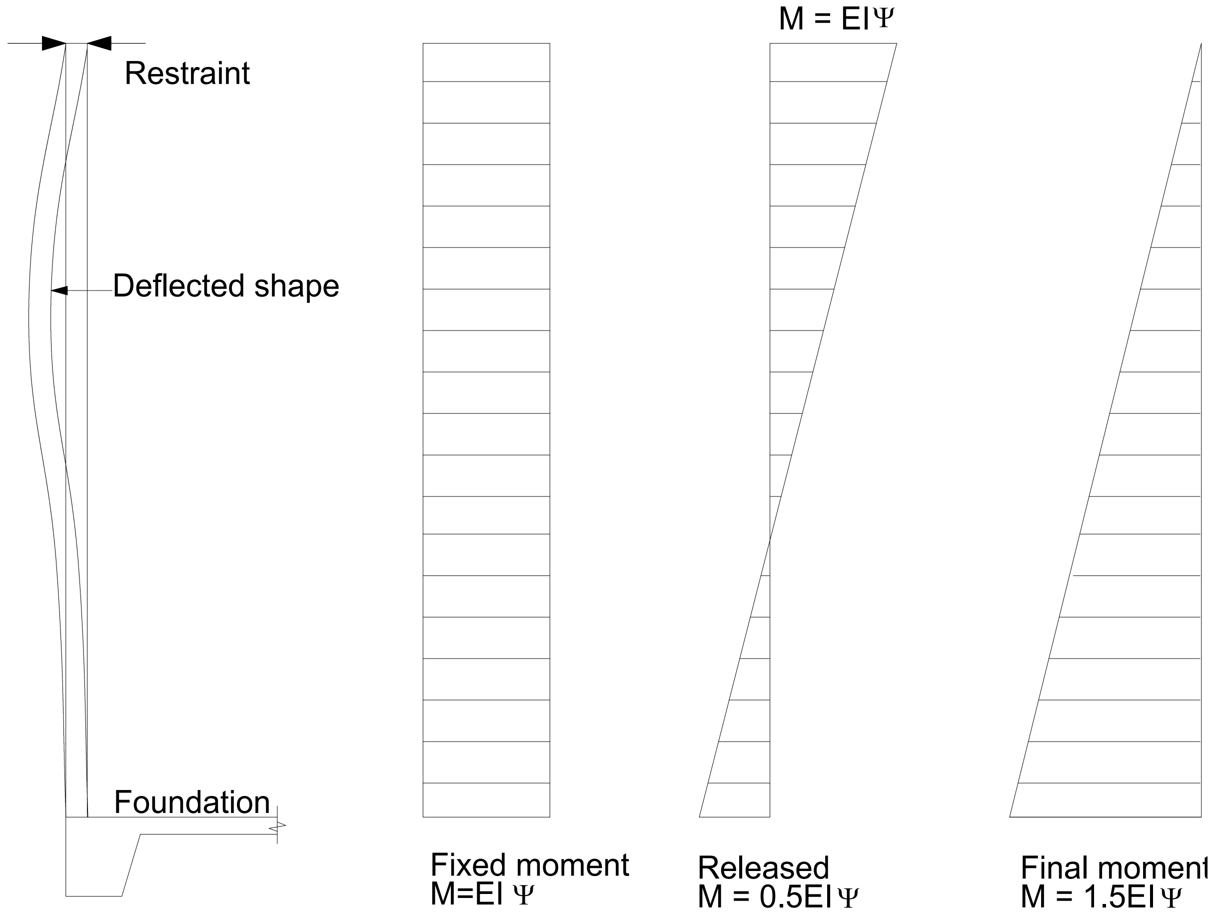


Figure 7. Deflected shape of fixed base wall with a temperature differential through its section and associated bending moment diagram

6. PERFORMANCE CRITERIA

The umbrella structural performance criteria comes from the NZBC [41], which requires that buildings "shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives".

Warehouse-type buildings with slender precast boundary walls generally have an inherent stability in that they are box-like in their form and the lengths of wall available to resist lateral shear loads are usually relatively large, particularly on the side walls. The combination of walls at right angles and a lightweight braced "diaphragm" roof structure means that the predominant lateral load-resisting mechanism is shear and there is flexure in the wall panels which are parallel to the load direction. The diaphragm will generally be in the form of a horizontal cross-braced "wind truss" of steel elements. For the long side walls, the magnitudes of the individual panel design forces are low because of the lightweight roof structure and the abundant length of wall which provides adequate capacity. The situation may be different for the end walls, which may include large doorway openings and the available wall panels will be called upon to resist the inertia forces from the roof and the top half of the long side walls. If there are no portal frames present, a roof "diaphragm" will generally be required to carry the roof and side wall inertial forces to the end walls and torsional effects will occur if the wall stiffnesses are not the same at both ends. Careful design of these end wall panels will be paramount because their collapse could lead to total collapse of the structure.

One exception to the type of structure described above is the type where only one end is open (e.g. a fertiliser store) and the building is long. In this case, the wall elements on the side walls adjacent to the open end will be required to resist loads perpendicular to their faces. Singly reinforced face-loaded walls, while having the ability to behave in a partially ductile manner, exhibit high flexibility because when the steel yields the wall "rocks" on the reinforcing bars as the cracks on both faces open and close [38].

When portal frames are included in the building there is a significant difference in that lateral forces from the roof and top half of the side wall panels are resisted by the portals. An exception to this is in the last half bay at each end of the building, where the loads from a relatively small area of roof and side wall panels are carried by the ends walls. In this instance, Table C1 of AS/NZS 1170.0 [53] recommends a deflection limit of portal spacing/200 under wind and seismic loads to prevent cladding damage. Notes to the table state that "the limiting deflection of the portal frame knee deflections is related to the behaviour of the cladding between the 'free portal' and a more rigid plane (typically the end wall of a structure)". The deflection limit of such portals is based on the bay spacing and ability of the cladding to withstand in-plane shear distortion.

Test results [28][29][38][42][56] have indicated that as long as the wall panels are adequately restrained at the top, total collapse is highly unlikely in a severe seismic event. Under loads both in-plane and out-of-plane in the elastic range of the wall panel, minor cracking is expected to occur. As the loads increase into the inelastic range of response, the panels will develop out-of-plane flexibility as the cracking increases and damage in the form of concrete spalling, bar bending and possibly fracture will occur at the base of the panel. Provided the eaves level connections are properly designed, the panel will continue to respond as a prop, pinned at its base and top. It is highly unlikely that a collapse mechanism will form because of the presence of the roof "diaphragm" and the orthogonal walls and/or portals (if present), which will provide out-of-plane support to the top of the damaged panels.

6.1 Choice of structural ductility factor and structural performance factor

There are three available design options which are dependent on the selection of the structural ductility factor and corresponding structural performance factor. These are presented in Table 2.

For warehouse structures, the majority of designs will fall within either of the first two options (see earlier in this section for the exception). Of these, the first option is by far the most popular among designers, particularly because of the generally large available wall lengths.

Table 2. Ductility factor choice for design of buildings with slender precast wall panels

Structure type	Structural ductility factor, μ	Structural performance factor, S_p
Nominally ductile walls	1.25	0.9
Walls of limited ductility	3	0.7
Single-storey cantilevered face-loaded walls of limited ductility	2	0.8

6.1.1 Nominally ductile walls (μ = 1.25, S_p = 0.9)

Walls that are designed as nominally ductile are expected to sustain only minor yielding damage in the design earthquake (one with an approximately 10% probability of exceedance in 50 years). As the earthquake strength increases beyond the design earthquake towards the maximum credible earthquake, nominally ductile designed structures will experience inelastic behaviour. For slender precast panels, this inelastic response will likely be yielding of the vertical flexural steel near the foundation interface. Experimental investigations [38][42][56] have shown that typical panels have been able to sustain actions well beyond the elastic range without collapse.

6.1.2 Limited ductile walls (μ = 3, S_p = 0.7)

These walls will be designed for a force of approximately one-half of the force that would be used for an equivalent-sized wall designed to respond elastically. However, the actual potential earthquake will not be any less strong, and so it is expected that the limited ductile wall will sustain greater damage than the elastically responding wall when exposed to the same earthquake.

Within the wall, there is little scope to detail the reinforcing to cater for the greater damage. A single layer of steel cannot be confined with steel cages. Hence, it is expected that concrete spalling may occur at the ends of the wall base once the wall has passed its elastic limit and bar buckling may occur. For limited ductile walls, NZS 3101 requires (clause 11.4.4) that the sectional curvature ductility at the ultimate limit state be less than $15\alpha_{fy}$ times the nominal curvature at first yield, γ, where $\alpha_{fy} = 400/f_y$ (but not greater than 1.1), and

$$\gamma = \frac{\varepsilon_y}{d - c}$$

where d is the distance from the compression edge to the centroid of the tension steel, c is the distance from the compression edge to the neutral axis and ε_y is the nominal yield strain in the reinforcing steel. The terms d and c have been calculated for a range of wall lengths between 1 m and 6 m, a steel content range from 0.32% to 1% and for 300 MPa and 500 MPa steel. From the calculations, for uniformly and single layer reinforced walls, d - c approximates 0.55 times the wall length. Therefore, the nominal curvature at first yield is 0.0025/L for 300 MPa steel and 0.00436/L for 500 MPa steel, where L is the length of the wall in millimetres. The corresponding limiting curvatures are 0.042/L and 0.052/L.

The critical factor is the ability of the wall-eaves beam connection to continue supporting the wall against out-of-plane forces and to accommodate any in-plane rotational actions while transferring shear forces into the top of the panel.

6.1.3 Cantilever face-loaded walls (μ = 2, S_p = 0.8)

Such walls are rarely utilised because their face-loaded cantilever moment capacity is relatively low and it is more efficient to use trusses in the roof plane to transfer forces to the in-plane loaded orthogonal walls. However, cantilever face-loaded walls may be necessary for long narrow buildings which have no transverse portal frames and either one or both ends fully open.

Cantilever face-loaded walls must be designed to ensure that no yielding occurs at the base of the wall because with yielding a mechanism will form and the structure will become unstable.

6.2 Foundation conditions

In structures designed to remain elastic, it is common for the foundations to be designed only for the elastic loads determined from the Loadings Standards [53].

For situations where a capacity design approach is used, two options exist for the design of the foundations. In the first case, the foundation capacity exceeds the wall panel over-strength capacity and all damage is confined to the wall panels. In the second case, the foundation is designed to “yield” leaving the wall panels undamaged. The disadvantage with this second option is that it is more difficult to repair following a damaging earthquake.

Alternative design procedures have been developed [42] where the wall panel is allowed to rock on the foundation. Energy is dissipated in the rocking process as stressed tendons are extended elastically in tension, and because the wall is armoured at the bottom corner pivot points, little damage occurs in either the panel or the foundation. Furthermore, at the completion of rocking, the panel returns to its original position and neither the panels nor the foundation are damaged. Damage to waterproofing between panels and other secondary elements may require repair.

7. DEFLECTION LIMITATIONS

7.1 In-plane deflections

NZS 1170.5 [50] requires that the inter-storey deflection limit under ultimate limit state loads shall not exceed 2.5% of the inter-storey height, but this limit is not really applicable to slender precast walls. The ability of the wall to deflect is governed by the aspect ratio of the wall and whether it is designed to behave nominally ductile or in a limited ductile manner. Nominally ductile walls with high aspect (i.e. height-to-length) ratios will deflect elastically as flexurally responding cantilevers.

Limited ductile walls have limiting curvatures placed on them in clause 2.6.1.3.4 of NZS 3101. The maximum allowable in-plane top deflection (drift) must be calculated, based on the limits given in Table 2.4 of NZS 3101. Over a range of practical wall aspect ratios, the maximum drift is between 0.7% and 1.5%.

See also the recommended portal deflection limits in section 6 above.

7.2 Out-of-plane deflections

Out-of-plane deflections are expected from simple flexural bending under face loading and from roof plane bracing flexibility causing lateral translation of the top support under either wind or seismic loads. In the ultimate limit state condition, the combination of deflections from flexural bending of the wall panel and the roof flexibility must be less than the distance to the property boundary.

Considering first the out-of-plane deflection of the mid-height of a wall panel with respect to the top and bottom lateral support, the commentary to the earthquake Loadings Standard [50] recommends that under service loads a wall should not deflect more than its height divided by 400 to maintain functionality. For a 10 m high wall, this equates to 25 mm. However, the deflection under the ultimate limit state face load on the wall (designed as a part) is likely to be the governing criteria.

The out-of-plane deflection at the eaves under the ultimate limit state loads will be dependent on the roof plane flexibility and the contributing inertial mass. The addition of eccentric axial load from a roof (if supported by the wall) will increase the wall deflection at the eaves. This in turn will increase the P-Δ effect and an iterative calculation will be required to calculate the actual deflection.

8. STRUCTURE ANALYSIS AND PANEL DESIGN PROCEDURES

8.1 Basic load distribution principles

The design of the individual panels will be dependent on the layout of the building (i.e. number of panels available to carry the loads in-plane) and the stiffnesses of the critical components of the structure. The critical feature in the structural analysis is the behaviour of the roof. The walls are generally placed around the perimeter of the building.

If the roof is considered to be a rigid diaphragm element (much as is assumed in the design of structures with reinforced concrete floor elements) the lateral forces are distributed to the resisting elements (shear walls) according to their stiffnesses. Transverse walls will also participate in the resistance to torsional forces. The Loadings Standard [50] requires account to be taken of accidental eccentricity by assuming that the centre of mass of the building is offset from the calculated centre of mass by 0.1 times the plan dimension of the structure at right angles to the direction of loading.

If the roof is considered to be a flexible diaphragm element, then the forces will be generally distributed to the perimeter walls running parallel to the direction of action according to their tributary areas. For warehouse type structures with perimeter walls, this means that each side would receive lateral load from its half area of the building.

As an example, a rectangular warehouse structure with no portals may have a door at one end and solid walls along both sides and the other end (Figure 8). In an earthquake, the side wall elements would be lightly stressed in their in-plane direction. In the orthogonal direction the proportions of shear carried by the long end wall and the short end walls need to be determined.

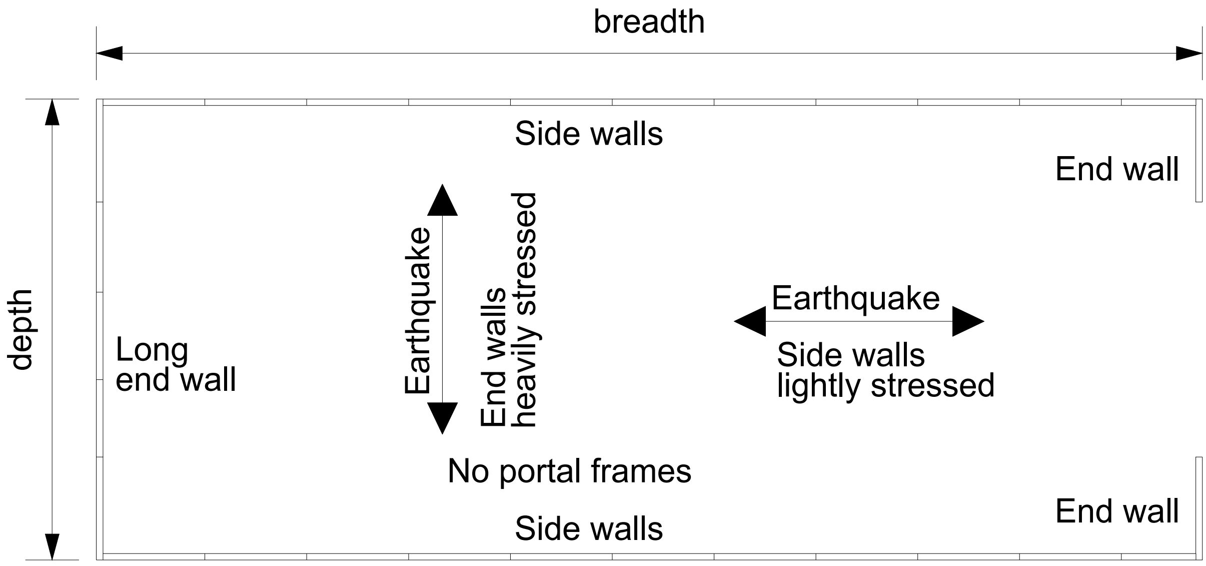


Figure 8. Rectangular warehouse wall loads

Other factors affecting the response include:

- the roof mass, which is generally small.
- the portal frames (if present). The stiffness of the portal frames is expected to be less than one-quarter of the shear stiffness of a bay of the roof diaphragm [57]
- the asymmetry of the structure. The presence of asymmetric wall arrangements will cause asymmetric shear force distributions when some roof diaphragm action is available
- the wall elements. The out-of-plane stiffness of the wall elements is considered to be very small compared to the other components and therefore can be ignored in any analysis. (Note that there may be rare situations where there is no end wall at one end of a long building. In these cases, out-of-plane cantilever resistance must be relied on to resist the loads transverse to the long direction of the structure.)

In an effort to quantify the loads being resisted by elements and transferred between elements of the structure, various three-dimensional dynamic analyses have been undertaken on typical rectangular warehouse structure models [45]. The effect of the relative stiffnesses of the elements and of the breadth-to-depth ratio (see Figure 8) of the building on the various element responses was investigated. It was found that as the flexibility of the roof increased, the proportion of base shear carried by the minor end wall changed in a non-linear fashion, depending on the flexibility of the roof.

It was concluded that for an unsymmetrical structure with a flexible roof, the proportion of base shear ascribed to the walls generally reduced because of the dynamic effects of the differences in the period of the roof and that of the building. However, quantification of the reduction over a range of building configurations was not able to be accurately determined, and therefore allowance has not been made for this in this Guide.

8.2 Building analysis procedure

This procedure is applicable to both symmetric and asymmetric buildings (i.e. those with unequal stiffness end walls) provided the stiffness of the small end wall is greater than 20% of the large end wall and the portal frames (if present) do not contribute significantly to the lateral load resistance of the structure.

Because it is not easily possible to accurately account for the roof diaphragm stiffness, two procedures are required to be followed for a complete analysis.

The first of these procedures assumes that the roof diaphragm is rigid and the stiffer wall is designed using the loads from the analysis, on the basis of wall stiffnesses. The second method assumes a flexible roof diaphragm and the more flexible (shorter) wall is designed on the basis of tributary area. This procedure ensures that the upper bound forces in the two walls are appropriately identified and designed for.

Alternatively, the designer may undertake a more complex structural analysis that includes the essential dynamic behaviour of all elements of the building.

8.2.1 Analysis procedure assuming a rigid diaphragm

(i) Calculate the stiffness of the end cantilever-resisting walls loaded in-plane, k_2 and k_4 (see Figure 9). In each case, this will be the accumulation of the in-plane stiffnesses of the individual panels making up the wall. As a first step, use the suggested values in Table C6.6 of the Commentary to the Concrete Structures Standard [36]. For walls, these are reproduced in Table 3 for convenience. Note that for all slender precast wall panels covered in this Guide, the non-dimensionalised axial load will be very close to zero. For normalised axial loads between 0 and, 1 interpolation may be used between the suggested I_{eff} values.

(ii) Determine the masses contributing to the lateral loading applied at the top of the shear walls, m_2, m_3 and m_4, and the position of the centre of this total mass and the centre of rigidity of the resisting walls.

(iii) To allow for potential accidental eccentricity of the centre of mass, offset the mass by 0.1 times the plan dimension of the building at right angles to the direction of loading in accordance with NZS 1170.5 to cause the worst effect for the stiffer wall in the direction of loading.

(iv) Calculate the translational period of the structure, T_1, where
$T_1 = 2\pi\sqrt{[(m_2 + m_3 + m_4)/(k_2 + k_4)]}$.

(v) Assign a ductility factor, μ, to the design of the wall panels (see section 6.1).

(vi) Calculate the horizontal design action coefficient, $C_d(T_1)$, from NZS 1170.5.

(vii) Calculate the total base shear, V, for the in-plane load-resisting panels:

$$V = C_d(T_1) * (m_2 + m_3 + m_4) * g$$

where g is gravitational acceleration.

(viii) Calculate the proportion of total base shear in each of the two end walls using statics for the worst effect of eccentricity of the centre of mass for each end wall (see Figure 10 for terms) and design the wall elements for the resulting forces. Note that the side walls assist in the resistance of the torsional loads caused by the eccentricity of the centre of mass and that V_2 and V_4 will add to more than the total base shear because of the consideration of the worst accidental eccentricity for each wall.

$$V_2 = V * (k_2/(k_2+k_4)) + (V * c_1) * d_2 * k_2 / J_p$$
$$V_4 = V * (k_4/(k_2+k_4)) + (V * c_2) * d_4 * k_4 / J_p$$

where
$J_p \quad = \quad (k_1 * d_1^2) + (k_2 * d_2^2) + (k_3 * d_3^2) + (k_4 * d_4^2)$

(ix) Calculate the shear force to design the shear connection between the roof and the resisting panels. The connection is designed for the shear force calculated in step (viii) multiplied by the ratio of the seismic design coefficient with $\mu = 1$ to the seismic design coefficient for the assumed ductility. Note that this shear force is likely to be spread equally between the panels in each wall:

Roof shear force, R_2 = (μ = 1 coef.)/(assumed μ coef.) * V_2;
R_4 = (μ = 1 coef.)/(assumed μ coef.) * V_4

(x) Calculate the design face load on the panel using the parts section of AS/NZS 1170.5 and the design "pull out" force on the eaves connections from the face-loaded panels. See Appendix C for a sample calculation.

(xi) Check the stability of the wall panels in accordance with section 8.4.

(xii) Since the roof diaphragm is assumed to be rigid there will be very small forces in the portals. From Davidson's study [45] it is reasonable to assume that 20% of the base shear will be resisted by the portal frames. Hence:

$V_{portal} = 0.2 \times V$ distributed evenly between all portal frames

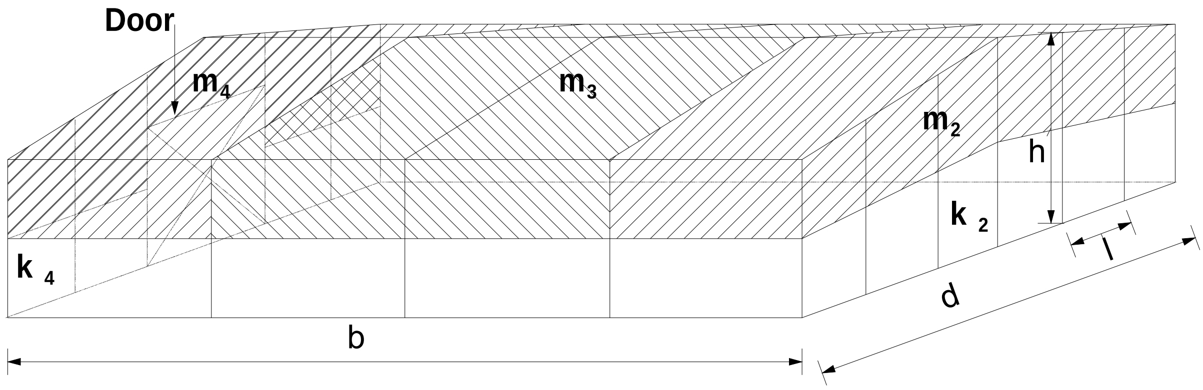


Figure 9. Diagram of typical structure showing contributing masses

Table 3. Effective section properties for walls, I_{eff}

Non-dimensionalised axial load	Ultimate Limit State		Serviceability Limit State	
	f_y = 300 MPa	f_y = 500 Mpa	μ = 1.25	μ = 3
$N^*/A_g f'_c = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g
$N^*/A_g f'_c = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g

N^* = design axial load at the ultimate limit state
A_g = gross cross-sectional area of the wall
I_g= gross moment of inertia of the panel

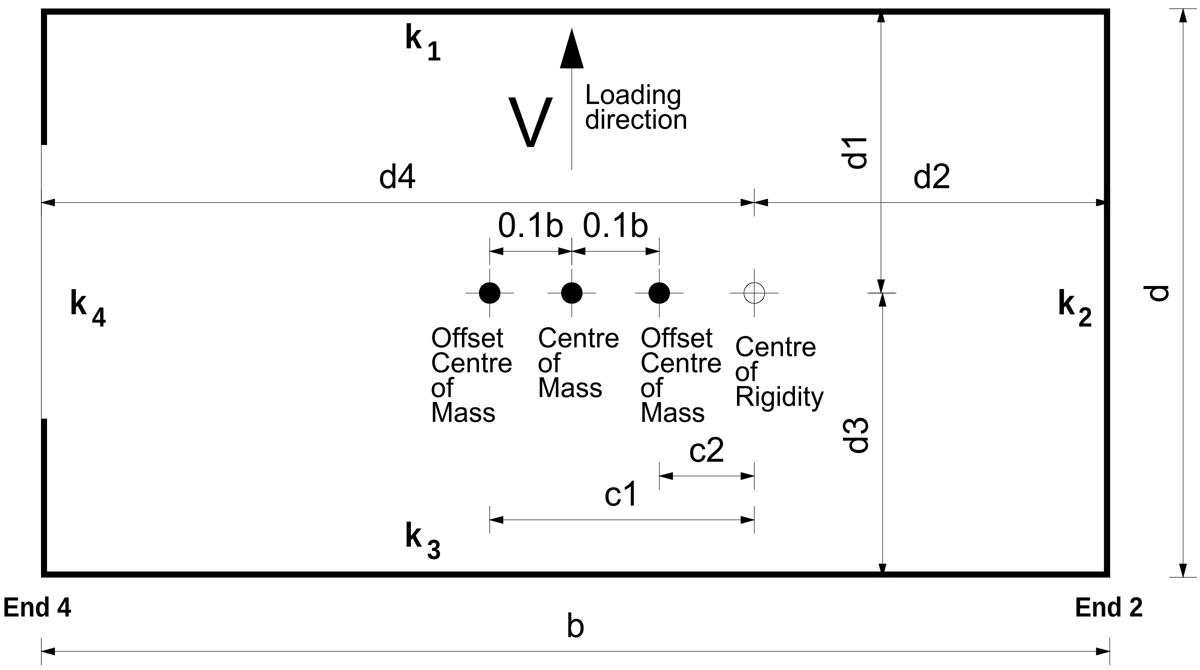


Figure 10. Dimensions for calculation of shears in end walls

8.2.2 Flexible roof diaphragm analysis procedure

(i) Calculate the stiffness of the end cantilever-resisting walls loaded in-plane, k_2 and k_4 (see Figure 10). In each case, this will be the accumulation of the in-plane stiffnesses of the individual panels making up the wall. As a first step, use the suggested values in Table C6.6 of the Commentary to the Concrete Structures Standard [36]. For walls, these are reproduced in Table 3 for convenience. Note that for all slender precast wall panels covered in this Guide, the non-dimensionalised axial load will be very close to zero.

(ii) Determine the mass associated with the dynamic response of the roof, m_3. This may be estimated as one-quarter of the mass of the side walls plus half of the total mass of the roof (Figure 9).

(iii) Determine the masses of the structure that are expected to move with the end walls, m_2 and m_4. These may be estimated for each wall to be half the mass of the resisting wall plus one-eighth of the mass of the orthogonal (face-loaded) walls plus quarter of the total roof mass (Figure 9).

(iv) Calculate the first mode period of the structure, T_1, assuming that the structure is symmetrical and the roof diaphragm is rigid, where $T_1 = 2\pi \sqrt{[(m_2 + m_3 + m_4)/(2k_2)]}$, and where k_2 is the stiffness of the major resisting wall.

(v) The second period used to define the stiffness of the structure is the period of vibration of the roof, T_r, and this is derived assuming the resisting end walls are rigid and the roof is flexible. Thus:

$T_r = 2\pi \sqrt{[m_3/(2k_3)]}$, where k_3 is the shear stiffness of the roof.

(Elastic analysis software may be used to determine the roof period.)

(vi) Calculate the ratio of the roof period, T_r, to the building period, T_1.

For ratios of 2 or less, assume that the roof is rigid and use the procedure given in section 8.2.1 to obtain the shears in the end walls.

For ratios greater than 2, the base shear to be distributed to both end walls is to be based on the mass associated with the tributary area. For each wall, this will generally be the inertial mass of the upper half of the individual wall plus half of the mass of the roof and half the mass of the upper part of the side walls.

(vii) Assign the same ductility factor to the design of the wall panels as in the procedure assuming a rigid diaphragm and calculate the seismic coefficient. Distribute the lateral force to the walls on the basis of contributing areas.

(viii) Design the wall panels for the applied in-plane forces and check the stability of the wall panels in accordance with section 8.4.

(ix) Calculate the shear force to design the shear connection between the roof and the resisting panels. The connection is designed for the shear force calculated in step (viii) multiplied by the ratio of the seismic design coefficient with μ = 1 to the seismic design coefficient for the assumed ductility. Note that this shear force is likely to be spread equally between the panels in each wall:

Roof shear force, $R_2 = (\mu = 1 \text{ coef.})/(\text{assumed } \mu \text{ coef.}) * V_2$;
$R_4 = (\mu = 1 \text{ coef.})/(\text{assumed } \mu \text{ coef.}) * V_4$

(x) Calculate the design "pull out" force for the eaves connection to the face-loaded panels using the parts section of NZS 1170.5 (see Appendix C).

8.2.3 Stiff portal frames parallel to end walls

If stiff portal frames are used to support the structure in parallel with the end shear walls then the end walls and the portal frames should be designed on the basis of tributary area contributions to each element. The portals should be designed to be sufficiently stiff to ensure that they do not deflect more than 1/200 of the span between the frames under serviceability limit state loads [53].

8.3 Influence of building design philosophy on choice of effective wall height coefficient, *k*

The performance of the individual slender panels will be dependent on their preconditioning. The Loadings Standard [50] allows the assumption that consideration of earthquake attack be given in two perpendicular directions independently if the structure is designed as a limited ductile structure. For nominally ductile structures it requires that while 100% of the action is applied in one direction, 30% must be concurrently applied in the orthogonal direction and vice versa.

For structures that are the subject of this Guide, the earthquake deflections sustained in one direction will have a potential influence on the design panel properties in the perpendicular direction. That is, if the bases of panels crack under out-of-plane displacement of their tops, then this will influence the choice of the *k* coefficient for loading in-plane. The *k* coefficient may be chosen from Table 5.

8.4 Wall panel design for stability

Recent research (see Appendix A) has indicated that while in the early stages of reversed cycling in-plane loading (elastic response), the out-of-plane deflected shape of wall panels tends to be a double curvature as shown in Figure 13(a), but as horizontal cracks form and the steel begins to yield at the base of the wall as the displacement ductility increases, the shape tends to a single curvature as shown in Figure 13(b). The tendency to single curvature will also occur if the wall has been displaced out-of-plane by actions orthogonal to the panel either during the same earthquake or by an earlier event (e.g. earthquake attack causing flexure of the roof plane bracing truss). Values for *k* factors for walls are given in Table 11.1 of the Concrete Standard. This table has been reproduced in Table 5 of this Guide to assist the user.

Therefore, four checks are required for stability as follows:

1) $H/t \leq 75$ (represents the upper limit on available test data)

2) $kH/t \leq 65$

3) stability under axial load – “Euler buckling” method

4) stability under Lateral Torsional Buckling

Checks 1) and 2) are self-explanatory.

3) Stability under axial load – “Euler buckling” method

In this check, the in-plane forces are resisted by a simple compression strut approximately diagonally through the panel, with a force balance being provided by the vertical reinforcement, as shown in Figure 11.

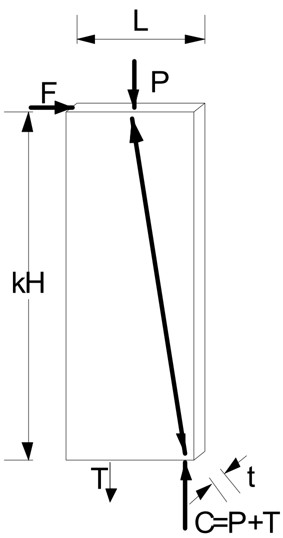


Figure 11. Forces in compression strut model

The force balance equation is as follows:

$$\frac{kH}{t} \leq \frac{15}{\sqrt{\left(\left(\frac{P+0.5W}{f'_c A_g}\right)+\left(0.4\rho_t \frac{f_y}{f'_c}\right)\right)}}$$ **Equation 1**

where

k	=	the effective wall height coefficient
H	=	the height of the wall
t	=	the wall thickness
P	=	the gravity load on wall from the roof
W	=	the weight of the wall
A_g	=	the cross-sectional area of the wall
ρ_t	=	the vertical steel ratio
f_y	=	the yield strength of the vertical steel
f'_c	=	the compressive strength of the concrete

The full derivation of the equation may be found in Appendix A.

The relationship in Equation 1 is plotted in Figure 12.

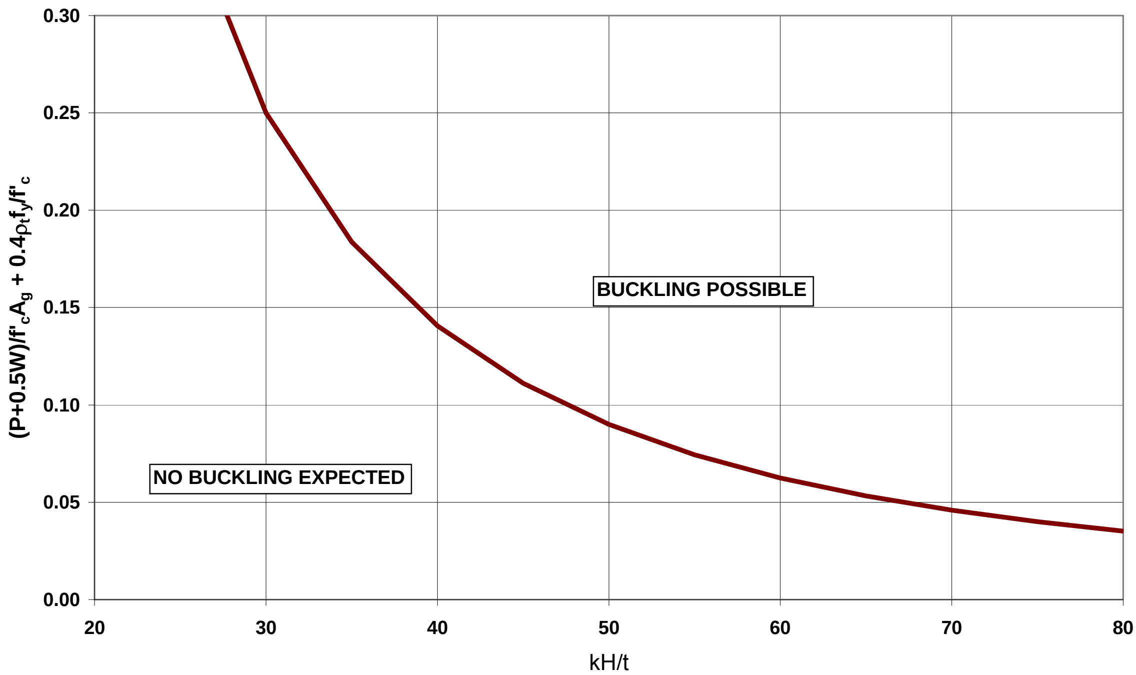


Figure 12. Euler relationship between wall slenderness and compression strut axial load

4) Vlasov/Timoshenko lateral torsional buckling

This check ensures that the panel will remain stable against out-of-plane lateral torsional buckling under the applied loads. The derivation of the Vlasov/Timoshenko lateral torsional buckling formula (Equation 2) is described in more detail in Appendix B.

$$\frac{kH}{t} \le 12\sqrt{\frac{H/L}{\lambda}}$$ **Equation 2**

where

k	=	the effective wall height coefficient (Table 5)
H	=	the height of the wall
t	=	the wall thickness
L	=	the length of the wall
λ	=	the lesser of:

(a) $\dfrac{P + 0.5W}{f_c^{'} A_g} + \rho_t \dfrac{f_y}{f_c^{'}}$ **or** **(b)** $\dfrac{2.2M_e^*}{Lf_c^{'} A_g}$

where

P	=	the gravity load on the wall from the roof
W	=	the self-weight of the wall
f'_c	=	the compressive strength of the concrete
f_y	=	the yield strength of the vertical steel
ρ_t	=	the vertical steel ratio
M_e^*	=	the design moment at the base of the wall corresponding with $\mu = 1.0$
A_g	=	the gross cross-sectional area of the wall

Equation 2, with λ derived from case (a) or (b), may be manipulated to produce a graphical relationship between load and wall geometry (Figure 14). To check a wall for stability, the variable terms on each side of the inequality sign may be calculated and the points plotted on the graph (Figure 14) or the inequality may be checked by calculation. If both the plotted points for cases (a) and (b) are below the line and the other criteria are satisfied, then the panel is expected to be stable. If one or other or both is above the line, or the other criteria are not satisfied, instability of the panel could be expected.

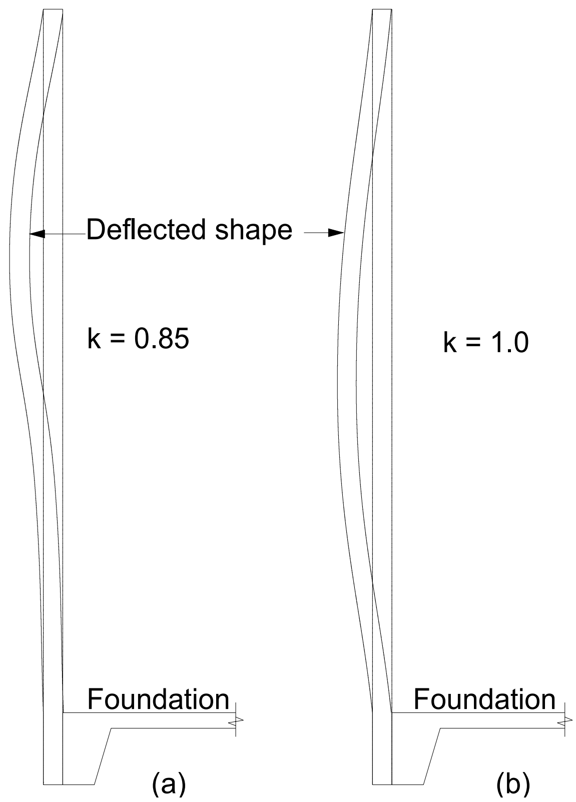


Figure 13 Out-of-plane deflected shapes of slender walls (NDPR case)

Table 5. *k* factors for slender walls

Case	Support condition at		Potential plastic region classification for in-plane loads	*k*
	Base of wall	Top of wall		
1	Fixed	Pinned	NDPR	0.85 or 1.0 where out-of-plane hinge forms at base
2	Fixed	Pinned	LDPR	1.0
3	Fixed	Nil	NDPR	1.4
4	Fixed	Nil	LDPR	1.4
5	Pinned	Pinned	NDPR, LDPR	1.0

NOTE:
(1) Fixed means rotational, lateral and torsional support is provided
(2) Pinned means torsional and lateral restraint is provided, but not rotational restraint
(3) Nil means none of torsional, rotational or lateral restraint is provided
(4) Abbreviations for potential plastic region classifications
NDPR = Nominally Ductile Plastic Region
LDPR = Limited Ductile Plastic Region

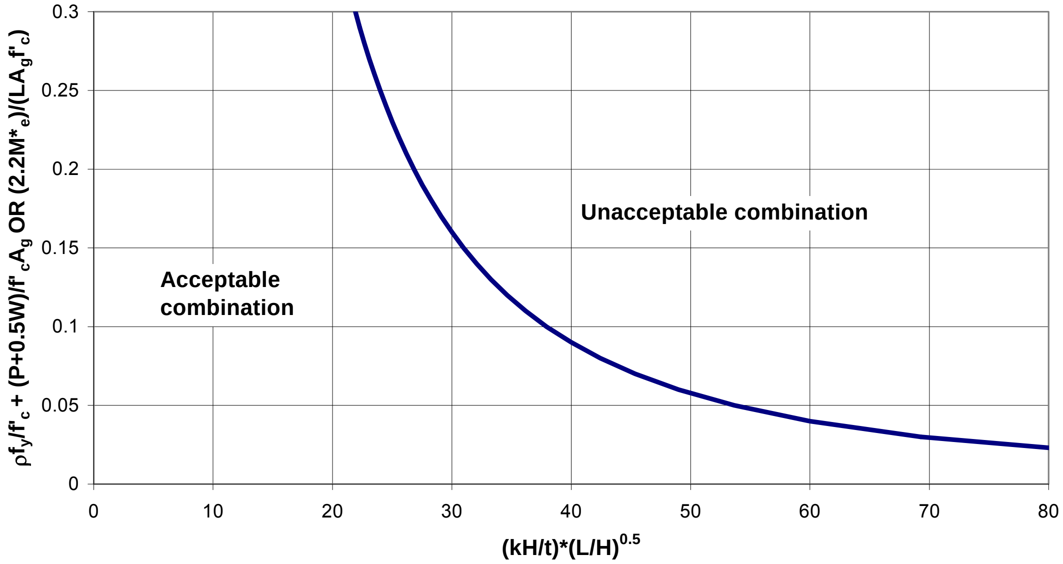


Figure 14. Relationship between wall panel slenderness and axial load (Vlasov)

9. FOUNDATION CONNECTION DETAILS

The foundation details for warehouse-type structure boundary walls are variable but tend to fall within two distinct categories.

In the first category the panels are often seated on top of a completed foundation and the connection between the two is provided by vertical starter bars that are grouted into corrugated metal tubes in the bottom of the panel (Figure 15(a)). These tubes are either concentrated near the end regions of the wall or are evenly spaced along the wall. In both instances, the tubes form an offset lap to the vertical bars in the wall. Narrow U-shaped horizontal bars are sometimes installed to provide some confinement to the end region, but these cannot be construed as satisfying the longitudinal steel confinement requirements of NZS 3101. With the wall panels well anchored to the foundation, the in-plane overturning forces from adjacent panels introduce shear forces through the foundation which must be designed for.

In the second category, the panels are seated on a cast in-situ foundation beam (which may only be unreinforced concrete) and rows of starter bars (generally two) project horizontally from the wall. An element of the floor slab adjacent to the wall is not cast until the wall panels are in position and the wall starter bars are cast into this element and lapped with the foundation slab bars. Alternatively, proprietary sockets are cast into the panel and starter bars are threaded into the sockets before the closing section of slab is poured (Figure 15(b)). The beam is generally reinforced as a beam with longitudinal flexural steel and shear stirrups to resist shear forces that are introduced to the thickened foundation.

The first category is commonly used in the South Island and the second category is used in both the North and South Islands.

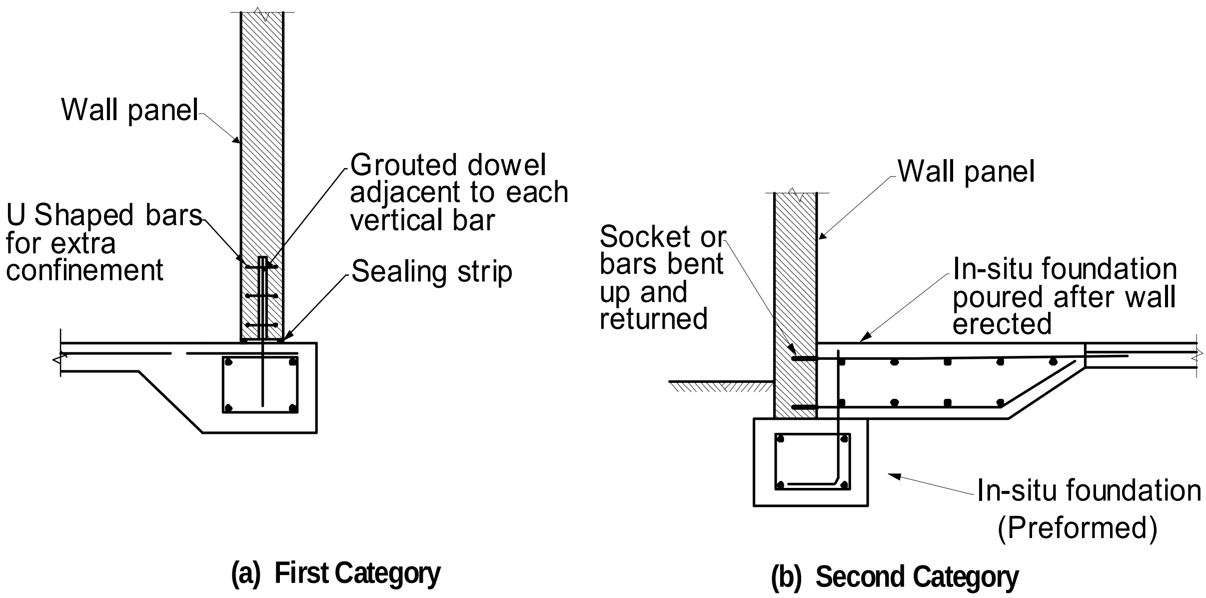


Figure 15. Different categories of foundation connection details

9.1 Connection design philosophy

Two options exist. The first is to design the foundation and foundation connection for the capacity of the wall panel in in-plane flexure so that damage in the event of the maximum credible earthquake is confined to the wall panel. The second is to design the panel-to-foundation connection to remain elastic under the elastically determined design forces, and accept that in the maximum credible event that steel yielding will either occur in the connection or in the foundation.

Information from the industry indicates that it is rare for the connection to be designed for the flexural capacity of the slender panel and the second of these options is the more favoured because smaller foundations are required. With portal-framed structures, there is generally sufficient length of longitudinal wall available to resist elastic in-plane loads well within the capacity of the wall panels because other factors, such as service load cracking and out-of-plane bending, will govern the design. Hence, the wall panels are never likely to reach their in-plane flexural capacity.

9.2 Design procedure for grouted bar connection

The development length of a vertical bar is dependent on both the steel nominal yield strength and the concrete compressive strength (NZS 3101 clause 8.6.3). Table 6 shows the range of development lengths for steel and concrete strengths.

When bars are lapped in non-contact splices, as with grouted ducts, and the bars are greater than $3d_b$ apart, the development length must be extended by a further 1.5 times the bar spacing (NZS 3101 clause 8.7.2.5). For example, a D16 bar with a 300 MPa yield strength in 70 MPa grout and with a spacing from the lapped bar of 50 mm will require a development length of 18 x 16 + 1.5 x 50 = 360 mm.

The beneficial effect of the grouted duct on the performance of the wall panels is in the confining action from the duct, thus minimising the chance of bar buckling.

Table 6. Development length of reinforcing steel

f_y (MPa)	f'_c (MPa)	20	30	40	50	70 (grout)
		Development length (Number of d_b*)				
300		34	27	24	21	18
500		56	46	40	35	30

*d_b = diameter of the bar (mm)

9.3 Design procedure for horizontal starter bar connection

A wall panel attached to the foundation as shown in Figure 15(b) will pivot on its compression edge on the in-situ foundation beam, thus placing most of the starter bars in upward shear as shown in Figure 16. There will also be a horizontal shear force shared between the starter bars. The joint can be designed using the shear friction principles of clause 7.7 of NZS 3101 only if the response at the critical shear plane remains elastic.

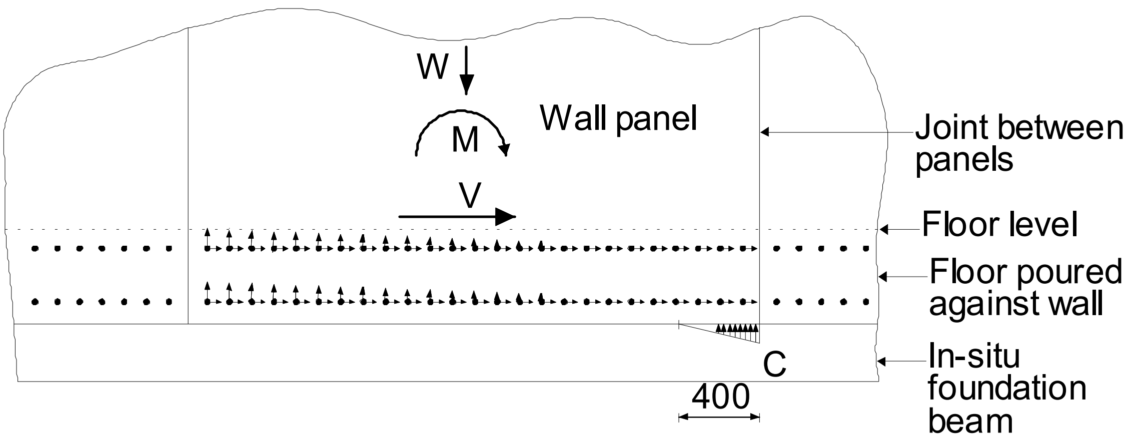


Figure 16. Forces at foundation connection

Consider an example case:

A warehouse building in Auckland located on a flexible soil site with slender precast panels on all perimeter walls. The building dimensions are 70 m x 70 m with an 8 m eaves height. The panels are 3 m long and 125 mm thick with 40 MPa compressive strength concrete and DH16 vertical bars spaced at 150 mm centres (ρ = 1%). At this percentage, no confining steel is required. Horizontal steel is DH12 at 300 mm.

Step 1. Calculate the forces on a side panel under nominally ductile seismic loads (μ = 1.25) and under wind loads in accordance with NZS 1170.5 [50] and AS/NZS 1170.2 [55] and compare with the strength of the panel. There is no requirement for the foundation connection to be designed for anything greater than the elastic response loads.

(a) Seismic demand (see ref [50])

Assume a roof weight of 0.3 kPa, giving a total roof weight of 0.3 x 70^2 = 1470 kN.

The weight of the top half of the end (face-loaded) walls is:

0.125 m x 70 m x (8/2) m x 2 ends x 24 kN/m^3 = 1680 kN.

Therefore, the total weight contributing horizontal in-plane seismic shear load to the top of the side walls is 1470 + 1680 = 3150 kN.

In accordance with the earthquake Loadings Standard, allow for an accidental eccentricity in the position of the centre of mass of 0.1 times the building width.

Therefore, the contributing weight to one side wall is ((70/2 + 3.5 m) / 70 m) x 3150 = 1733 kN.

In accordance with NZS 1170.5, determine the seismic loading on the in-plane loaded side shear wall. Assume the building is located on a shallow soil site in Auckland. The walls are stiff with an in-plane period of vibration of 0.4 s.

$C(T) = C_h(T) \times R \times Z \times N(T,D)$

$C_h(T) = 2.36$, R = 1.0, Z = 0.13, $N(T,D)$ = 1.0

Therefore, $C(T)$ = 2.36 * 0.13 = 0.31

$$C_d(T) = \frac{C(T_1)S_p}{k_\mu}$$

S_p = 0.9, from clause 2.6.2.2.1 of NZS 3101 [32]

Because T = 0.4 s, $k_\mu = \frac{(\mu - 1)T}{0.7} + 1 = \frac{(1.25 - 1)0.4}{0.7} + 1 = 1.14$

Therefore $C_d(T)$ = 0.31 * 0.9 / 1.14 = 0.245, and the seismic shear = 1733 * 0.245 = **425 kN.**

The base shear force is 425 / 70 = 6.1 kN/m (or **18.3 kN per panel**) and the moment at the base of each panel due to the seismic load is 18.3 kN x 8 m = **146 kNm**.

(b) Wind demand (see ref [55])

The site wind speed is given by the equation:

$V_{sit,\beta} = V_R M_d (M_{(z,cat)} M_s M_t)$

For Auckland, region A6 wind speed, V_R = 45 m/s.

For consideration of wind from any direction, the wind direction multiplier, M_d, is 1.0.

$M_{(z,cat)}$ = 0.83.

$M_s = M_t$ = 1.

Therefore, $V_{sit,\beta}$ = 45 * 1.0 * 0.83 * 1 * 1 = 37.35 m/s.

The design wind pressure, p, is:

$p = 0.6 * (V_{sit,\beta})^2 C_{fig} C_{dyn}$

$C_{fig} = C_{p,e} K_a K_c K_l K_p$ for external pressures.

For the whole building, $C_{pe} = 1.1$, $K_c = 0.8$ and $K_a = K_l = K_p = C_{dyn} = 1.0$.

Therefore, $p = 0.6 * (37.35)^2 * 1.1 * 1.0 * 0.8 * 1.0 * 1.0 = 736$ Pa.

Therefore the total wind force to be resisted by the side shear walls is:

p * building width * (building height/2) = 0.736 x 70 x (8/2) = **206 kN or 103 kN/side**. (cf **425 kN** seismic shear force)

Therefore, the seismic demand is greater than the wind demand and will govern the design of the connection between the panels and the foundation.

(c) Calculate the ultimate strength of the panel at the base connection

Neglect the contribution from the steel in compression and assume that the compressive stress block is 10% of the panel width and all tension steel is yielding:

$a = (A_s \times f_y) / (0.85 \times f'_c \times b) = 201 \times 18 \text{ bars} \times 500 / (0.85 \times 40 \times 125) = 425$ mm

Adjust the length of the compression block based on the number of bars in tension:

$a = (A_s \times f_y) / (0.85 \times f'_c \times b) = 201 \times 17 \text{ bars} \times 500 / (0.85 \times 40 \times 125) = 402$ mm

The ultimate strength of the panel is given approximately by:

$$M_{ult} = \phi A_s f_y \left(d - \frac{a}{2} \right) = 0.85 \times 17 \times 201 \times 500 \times (1625 - (402/2)) \times 10^{-6} = \mathbf{2068\ kNm}$$

which is well in excess of the design moment of 146 kNm for a nominally ductile wall.

Step 2. Check the shear strength of the panel under the ultimate moment (clause 11.3.10).

Design shear force in panel is 18.3 kN = V^* (from step 1(a)).

The nominal concrete shear stress is calculated from equation 11-12 in NZS 3101.

$$v_c = 0.17\sqrt{f_c^{'}} = 0.17 * \sqrt{40} = 1.08 \text{ MPa}$$

The effective cross-sectional area of the wall for shear is 0.8 * L * t

= 0.8 * 3000 * 125 = 300000 mm^2.

Therefore, $V_c = 1.08 * 300000 = 324$ kN > 18.3 kN – okay.

Therefore, minimum horizontal shear steel is required in the panel

$$A_v = \frac{0.7 b_w s_2}{f_{yt}}$$

For this panel, the maximum spacing of shear reinforcement is three times the wall thickness = 375 mm (clause 11.3.10.3.8(c)). However, bars have been placed at 300 mm centres.

Minimum horizontal steel is therefore:

(0.7 x 125 x 300) / 500 = 52 mm^2. HD12 bars at 300 mm centres satisfy this requirement.

Step 3. Determine the mechanism for transfer of seismic forces for the nominally ductile panel to the foundation.

Forces from the panel are expected to be transferred to the foundation via the dowel starter bars. Moments will be resisted by a force couple between the supporting foundation and the dowel bars and the accompanying shear will be resisted by the dowel bars.

The lever arm between the compression force in the foundation concrete and the tension force in the starter bar group is approximately 2/3 the length of the panel = 2 m. Therefore:

$$T = C = M/_{leverarm} = 146/2.0 = 73kN$$

With a triangular stress distribution, the maximum compressive stress in the concrete =

$\frac{73000 * 2}{125 * 1000} = 1.2$ MPa – okay.

The shear force is shared between two rows of DH12 starter bars, say 400 mm apart with the bars spaced at 150 mm centres in each row.

The upward shear force due to the bending moment is a triangular distribution increasing towards the tension end of the wall. The maximum bar force is = 2 x 73/(2 x 17 bars) = 2.2 kN.

The total horizontal shear force, V^* = 18.3 kN (see above).

Therefore, the horizontal shear force = 18.3/40 bars = 0.46 kN/bar.

The resultant maximum shear force, V^*_{bar}, on the heaviest loaded bar is $\sqrt{(2.2^2 + 0.46^2)}$ = 2.3 kN.

The nominal shear strength, V_n, is calculated using equation 7-13 of NZS 3101, and V^*_{bar} must be less than φV_n

$V_n = (A_v f_y + N^*)\mu$

where

A_v	=	the area of the steel crossing the interface
f_y	=	the yield strength of the steel
N^*	=	the design force acting normal to the shear plane
μ	=	the coefficient of friction for the joint (= 1.0 for this joint)
φ	=	the strength reduction factor (0.75)

Therefore φV_n = 0.75 * (113 * 500 + 0) * 1.0 = 42 kN, which is greater than V^*_{bar}.

Two rows of DH12 horizontal starter bars at 150 mm centres in each row are therefore sufficient to transfer the base moment and shear. Note that in order to use a joint friction, μ, of 1.0, the face of the panel abutting the foundation must be free of laitance and intentionally roughened to an amplitude of not less than 2 mm.

Step 4. Design the foundation beam for the introduced shear forces resulting from the panel actions.

The foundation is subjected to the shear forces from the discrete wall panels introduced via the dowel bars and bearing of the panel base on the foundation, as shown in Figure 15. Actions will generally be resisted by equal but opposite actions in adjacent panels, but the forces must be transferred via the foundation.

For the above example, the design moment for the nominally ductile panel is 146 kNm.

Therefore, the design shear force in the foundation, V^*, is 146/3 m = 48.7 kN.

Assume that the foundation consists of a thickened floor slab 600 mm deep by 600 mm wide and reinforced with two D16 bars top and bottom (ρ_w = 0.0027) and with a concrete strength of 30 MPa.

NZS 3101 (clause 7.5) requires $\varphi V_n \geq V^*$ and φ = 0.75. Section 9 of NZS 3101 is used to calculate the available shear strength.

$V_n = v_n * A_{cv}$

The maximum nominal shear stress, $v_n \leq 0.2f'_c \leq 0.2 * 30 = 6$ MPa

$A_{cv} = b_w d = 600 * 500 = 300000$ mm^2

Therefore the maximum $V_n = 6 * 300000 * 10^{-3} = 1800$ kN.

The shear resisted by the concrete, $v_c = k_d k_a v_b$

$k_d = (400/d)^{0.25}$ = (400/500) * 0.25 = 0.95

$k_a = 1.0$

v_b is the smaller of $(0.07+10\rho)\sqrt{f'_c}$ = 0.53 MPa or $0.2\sqrt{f'_c}$ = 1.1 MPa.

Therefore v_c = 0.95 * 1.0 * 0.53 = 0.5 MPa.

Therefore V_c = 0.5 * 300000 = 150 kN and φV_c = 0.75 * 150 = 112.kN > $2V^*$.

Therefore, no shear reinforcement is required.

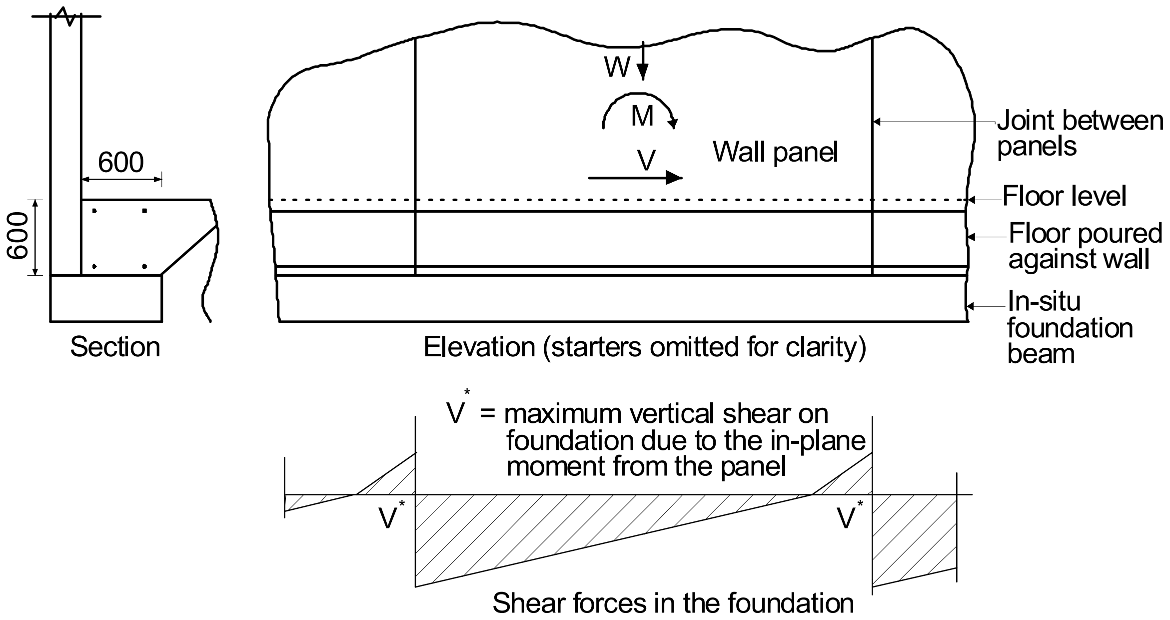


Figure 17. Vertical shear forces in foundation from the in-plane loaded panels

10. EAVES CONNECTION DETAILS

For construction efficiency, slender boundary panels are invariably built to cantilever from the floor, but be propped at the eaves level. Should the building be gutted by fire, the connections between the boundary wall must remain intact throughout the fire (see section 5.1).

10.1 Design forces

In an earthquake, the tenacity of the connection between the panels and the eaves beam is paramount to the effective resistance of the earthquake actions. Dynamic computer analyses [45] have shown that the connection forces are magnified by up to 1.25 times depending on the flexibility of the roof structure and, as a result, it is necessary to design the connections for these amplified forces. (See also Appendix C for determination of earthquake forces on parts according to NZS 1170.5 [50] which yield a base shear magnification factor for the design of connections to withstand potential face loads on the panels due to their own inertia.)

Fastener characteristic strengths may be calculated using Chapter 17 of the Concrete Standard.

10.2 Accommodation of panel in-plane rotations

When subjected to in-plane shear loading, narrow wall panels will rotate individually about their bases and vertical misalignment will occur at the interfaces as shown in Figure 18. As the panel width-to-height ratio increases, the panel displacement will shift from flexure governed to shear governed, and the need to accommodate differential vertical movement between panels will disappear.

When flexural panel rotation is expected to occur, as is expected in the maximum credible seismic event, this must be accommodated in the fixings between the panels and the eaves channel to ensure the integrity of the joint is maintained. In research at BRANZ [38], drilled proprietary fixings were installed through close tolerance holes between the channel and the panel at the outer edges of the panel (i.e. total of two fixings per panel). Cracks formed in the panels, emanating from the proprietary anchors, as the top corners were forced off the panels at drifts of 0.5%. Fixings through close tolerance holes are likely to cause significant local damage in the panel, to the extent that the strength of the fixings may be compromised.

It is recommended that a minimum of three fixings per panel be used. The centre connection should be placed through a normal tolerance hole in the eaves member to locate the eaves member vertically. Adjacent connections are placed through vertically slotted holes so that all fixings provide support for the panel against face loading, but in-plane rotation of the panel can occur about the centre connection without damage occurring. The required vertical dimension of the slot is related to the panel width. The following approximate relationship may be used as a guide for the slot length of the furthest fixing from the centre of the panel:

Slot length (mm) = (Panel width (mm) – 2 * fixing edge distance (mm)) * 0.02

Equation 3

This equation is based on a maximum drift of ±1%.

For fixings intermediate between the centre and the outside fixings, the slot length may be linearly reduced in length, but designers may elect to keep the slots all the same length for simplicity. It is imperative that the fixings are installed at the centre of the slot length to allow for reversed loading.

All the fixings should be snug tightened and lock-nutted to keep the friction forces between the eaves member and the panels to a minimum. Cast-in fixings or anchors that pass through the panel with outside bearing plates are recommended so that the panel remains firmly attached to the eaves beam. Expansion anchors rely on being tightened to a specified torque to mobilise the wedging action of the anchor and this is likely to result in friction forces that will prevent rotation from occurring easily. Glued-in anchors, while providing excellent support at normal temperatures, have been found to behave poorly at elevated temperatures [48] and should not be used if not protected from fire.

Proprietary inserts are available from Australia for installation in the panels at the time of casting, which allow for movement of the fixing in all directions in the plane of the panel up to about ± 30 mm. However, satisfactory performance relies on accurate placement of the fittings at the time of casting. The minimum panel thickness possible with these anchors is 150 mm.

Flexural in-plane rotation of panels is not expected to be significant until the height-to-width ratio is greater than 3.0. For panels with ratios less than this slotted holes would be unnecessary. There is insufficient research data available to be more specific. However, panels designed within the 1% maximum vertical steel percentage before confinement is required will still be flexurally governed.

See section 8.2 for determination of loads on eaves connections.

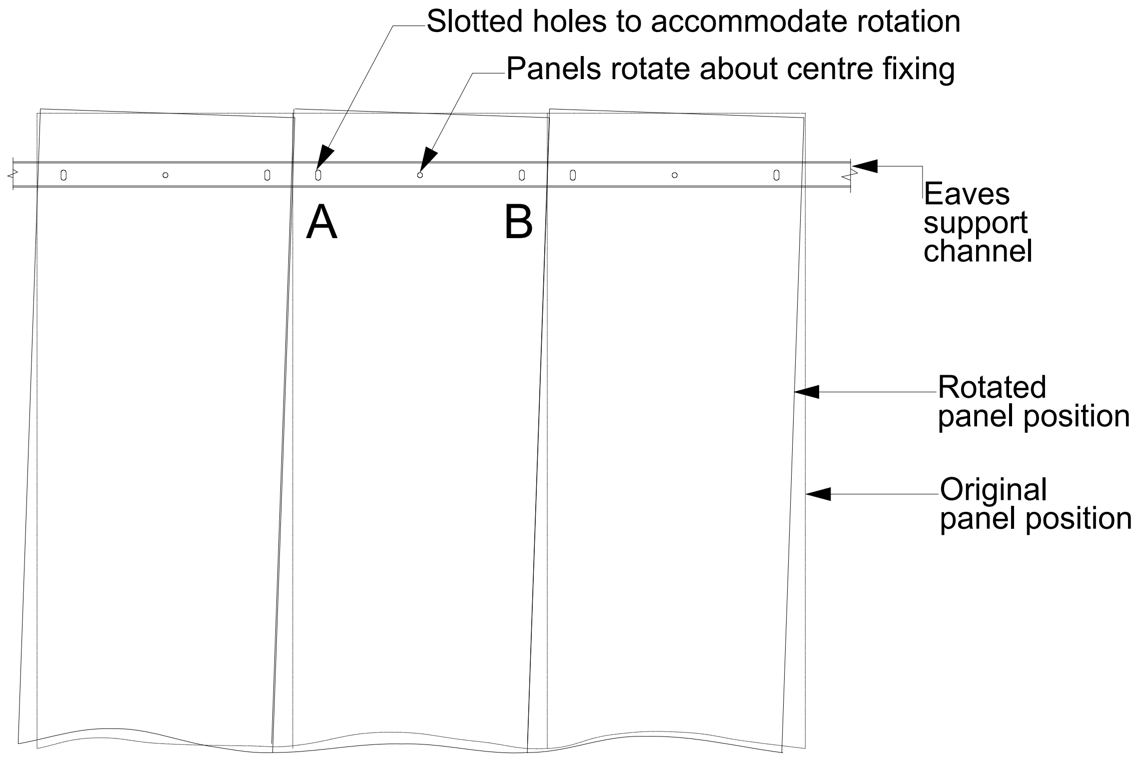


Figure 18 Displaced shape of abutting wall panels under in-plane loading. (Note that if the slotted hole details are not provided at A and B then there is a risk that the bolts will be sheared off as the panels rotate.)

11. PANEL-TO-PANEL CONNECTIONS

Typically, slender panels used on perimeter walls of warehouse structures have no mechanical connection between them. This allows for differential out-of-plane movements without introducing secondary stresses at connections, which may damage the wall panels. Figure 6 shows a typical method of sealing the joint against weather penetration.

12. EXAMPLE DESIGN CALCULATIONS

12.1 Example 1 sample calculations

Consider a supermarket structure with plan dimensions of 59 m by 54 m located in Tokoroa on shallow soil (site subsoil class C). Three sides of the building have 9 m high by 3 m long by 150 mm thick precast panels providing weather protection and structural support. The remaining side has two short lengths of panels and glazing between. Bracing resistance is provided by these shear panels along this face. Six steel portal frames span in the 54 m direction at approximately 8.5 m centres and one bay is cross-braced in the roof plane to transfer out-of-plane forces from the end walls and the roof forces to the side walls. On the end walls the panels exist over the full length of the wall. They are connected together with a steel member on a rake to match the rake of the portal frames. This member is also the support for the roof purlins.

Details of the plan of the structure are shown in Figure 17. For this design example, the roof is assumed to be connected to the wall panels at their tops.

The concrete strength is assumed to be **40 MPa**.

The reinforcing steel strength is assumed to be **500 MPa**.

The wall thickness is less than 200 mm, therefore a single layer of reinforcing steel may be used (see section 4.1).

The minimum horizontal steel percentage is **0.14%** (see section 4.4.1).

The minimum vertical steel percentage is **0.32%** (see section 4.4.2).

The maximum reinforcing bar size is **20 mm** generally and **16 mm** in potential plastic hinge regions (see section 4.4).

The maximum allowable steel spacing in both directions is **450 mm** (see section 4.4).

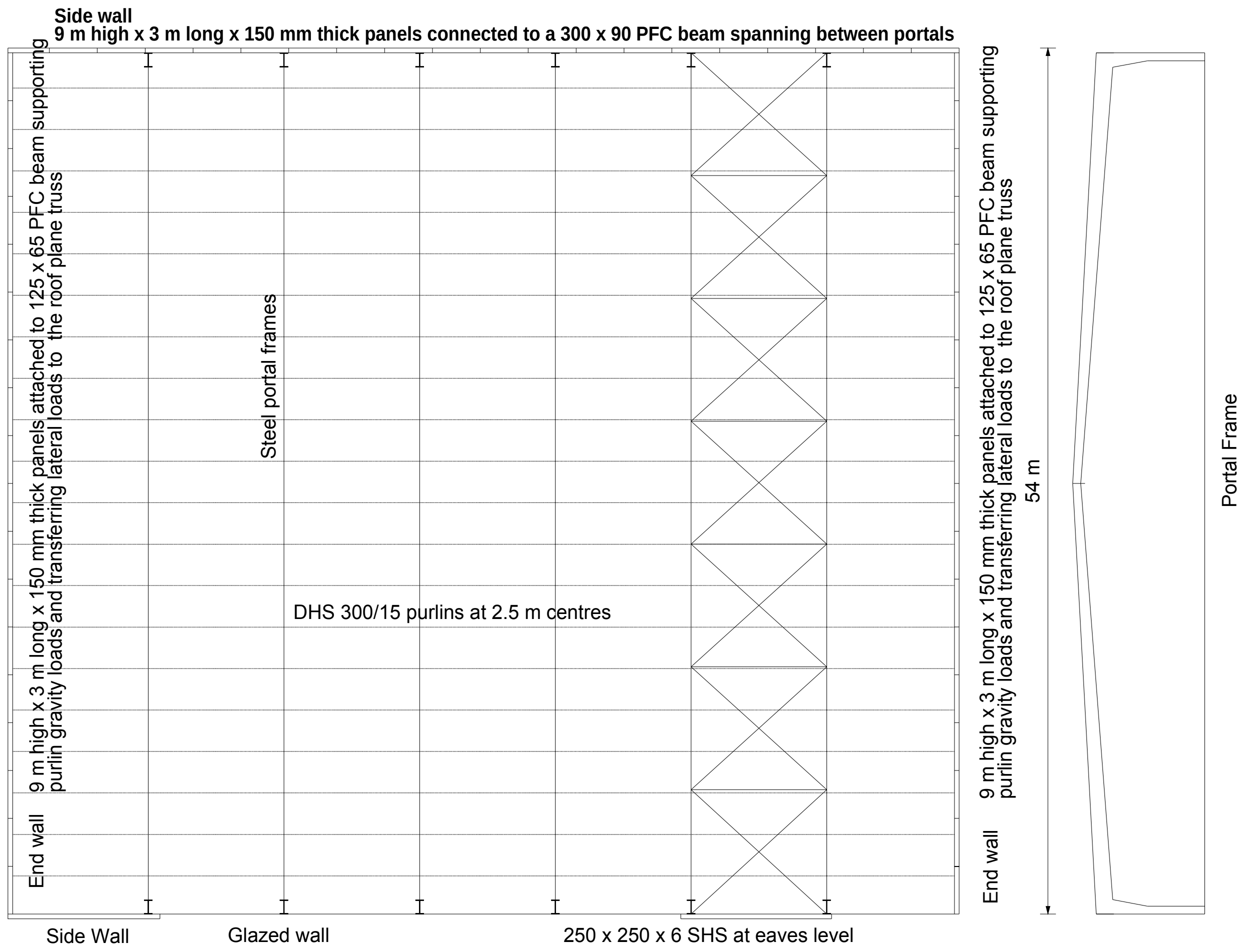


Figure 19 Plan of supermarket structure

12.1.1 Calculate the weights of the various components of the building:

Roof cladding weight (50 mm thick EPS)	0.11	kPa
Purlin equivalent uniform weight (DHS 300 x 15 at 2.5 m centres)	0.027	kPa
Portal rafter equivalent uniform weight	0.16	kPa
Plant equivalent uniform weight	0.05	kPa
Total roof plane uniform weight	0.35	kPa
Precast concrete panel unit weight (0.15 x 24)	3.6	kPa

12.1.2 Seismic loads in the direction of the portals (assume rigid roof diaphragm)

(see section 8.2.1 for process steps)

Step 1. Calculate stiffness of the end walls.

In this case the end walls will have the same stiffness as they are both continuous and the same length. Each wall panel has a stiffness of $3EI_e/H^3$. The concrete strength is 40 MPa. Therefore, *E* is 3320 * √40 + 6900 = 28 GPa. $I_e = 0.25I_g$ = $(3000)^3$ * 150 /12 = 3.37E11 mm^4 (NZS 3101 Table C6.6). If the load is applied at the top of the wall, then H = 9000 mm.

Therefore, the panel in-plane stiffness is 3 * 28000 * 3.37E11 / 9000^3 = 39000 kN/m. For 18 panels the combined stiffness is 702000 kN/m.

Step 2. Determine the masses m_2, m_3 *and* m_4 and the centre of mass of the roof and side walls (refer Figure 9).

The roof weight = 0.35 kPa * 54 m * 59 m = 1115 kN (114 tonne).

The weight of the top half of the side walls

= 3.6 kPa * 4.5 m * (59 m + 2 * 9 m) = 1247 kN (106 tonne).

The weight of the top half of the end walls

= 3.6 kPa * 4.5m * 54 m * 2 ends = 1750 kN (178 tonne).

Therefore $m_2 + m_3 + m_4$ = 114 + 106 + 178 = 398 tonne.

Given the masses of the side wall with glazing are small compared to the mass of the continuous wall, assume that the centre of mass is at the mid-length of the building.

In this direction, the centre of rigidity will be at the centre length as the two end walls are the same stiffness.

Step 3. Determine accidental eccentricity.

Building length is 59 m. Therefore accidental eccentricity is 5.9 m from the mid-length of the building.

Step 4. Calculate the period of the structure.

$T_1 = 2\pi\sqrt{[(m_2 + m_3 + m_4)/(k_2 + k_4)]}$ = 2π√[(398 / (702000 + 702000)] = 0.1 s.

Step 5. Assign a ductility for the wall panel design.

Assign μ = 3, because the design is assuming a limited ductile response.

Step 6. Calculate the horizontal design action coefficient, $C_d(T_1)$ (ref section 3.1 of NZS 1170.5).

$C(T)$	=	$C_h(T)ZRN(T,D)$
$C_h(T)$	=	2.36 (site subsoils class C and building period of 0.1 s)
Z	=	0.21 (building situated in Tokoroa)
R	=	1.0
$N(T,D)$	=	1.0 (not near an active fault)

Therefore, $C(T)$ = 0.496

$k_\mu = ((\mu-1)T_1/0.7) + 1$ = ((3-1) * 0.1 / 0.7) +1 = 1.28

Therefore, $C_d(T_1) = (C(T_1)S_p/k_\mu$ = 0.496 * 0.7 / 1.28 = **0.27**

(For the case of μ = 1, $k_\mu = ((\mu-1)T_1/0.7) + 1$ = ((1-1) * 0.1 / 0.7) +1 = 1.0 and S_p = 1. Therefore $C_d(T_1)$ = **0.496.**)

Step 7. Therefore the total base shear, V, is 0.27 * 398 * 9.81 = **1054** kN.

Step 8. Distribute the base shear, allowing for accidental eccentricity and design wall steel

$V_{end,max}$ = ((59/2)+5.9)/59 * V = 0.6 * V = 0.6 * 1054 = **632 kN.**

Because the accidental eccentricity could be in either direction, design both end walls for $V_{end,max}$.

There are 18 – 3 m long panels along each end wall.

Therefore, 632/18 = **35** kN/panel is the shear at the base of each panel.

The base moment for each panel is made up of moment from the self-weight of the panel plus a moment due to the horizontal load being introduced from the roof.

From the weights calculated in Step 2 and the seismic coefficient calculated in Step 6, determine the shear forces applied to the individual panels.

Allowing for the eccentricity of the centre of mass (Step 3), the seismic force from the roof

= (1115 + 1247) * 0.6 * 0.27 = 382 kN.

This force is resisted by the 18 panels. Hence the shear force at the top of each panel is 382/18 = **21.3 kN**.

The self-weight of the panel is 3.6 kPa * 9 m * 3 m = 97.2 kN. Therefore, the seismic force from the self-weight of the panel, applied at the mid-height of the panel, is 97.2 * 0.27 = **26.2 kN**.

Therefore the in-plane moment at the base of the panel, M_e* = 21.3 * 9 + 26.2 * 4.5 = **310 kNm**.

Consider using **H12 bars at 200 mm centres** – 15 bars per panel (ρ = 0.0038) (near minimum steel content of 0.32%).

Therefore, $\varphi M_n = \varphi Asfy(d-a/2)$

$a = A_sf_y/0.85f'_cb$ = 14 * 113 * 500 / (0.85 * 40 * 150) = 155 mm.

Therefore, φM_n = 0.85 x (113 x 14) x 500 x (1600 – 155/2) = 1023 kNm > 310 kNm – okay.

Check the shear strength in the plane of the panel for μ = 1.

The μ = 1 base shear may be determined by multiplying the μ = 3 base shear by the ratio of the seismic coefficients (see step 6 above):

V* = 35 * 0.496/0.27 = 64.3 kN.

The effective shear area, A_{cv} is 0.8 times the length of the panel times the panel thickness

= 0.8 * 3000 * 150 = 360000 mm^2.

The maximum nominal shear stress, v_{max}, shall be not greater than $0.2f'_c$ (= 8MPa) nor 8 MPa (NZS 3101 clause 7.5.2).

The maximum shear able to be resisted by the concrete, v_c, is $0.17\sqrt{f'_c} = 0.17\sqrt{40} = 1.1$ MPa.

Therefore, $V_c = v_c * A_{cv} = 1.1 * 360000 * 10^{-3} = 396$ kN >> $V*$.

However, a minimum quantity of horizontal steel is required by clause 11.3.10.3.8 of NZS 3101.

The minimum required steel area, $A_v = (0.7 * b_w * s_2) / f_{yt}$

where	b_w	=	the wall thickness (mm)
	s_2	=	the spacing of the horizontal steel (mm)
	f_{yt}	=	the yield strength of the horizontal steel (MPa)

The maximum horizontal steel spacing is 450 mm. If grade 500 bars are used, then:

$A_v = 0.7 * 150 * 450 / 500 = 94.5$ mm^2. Therefore **H12 bars at 450 centres** may be used for horizontal steel.

Step 9. Calculate the shear force in the wall/roof connection.

Design top connection for shear of 0.496/0.27 x 21.3 = 39.1 kN per panel (where 0.496/0.27 is the ratio between the $\mu = 1$and $\mu = 3$ seismic design coefficients).

With bolts at 1 m centres starting at 0.5 m from the ends, this means three bolts per panel or 13.3 kN/bolt.

Therefore, three M20 bolts either cast-in or through the wall panel will suffice. The calculation of the strength of the bolts in shear is as per section 17 of NZS 3101 and has not been included here.

Step 10. Calculate the design face load on the side panels and check steel content.

The Parts section of NZS 1170.5 is focused on multi-level buildings. To apply it to a single-storey slender panel structure requires a critical assumption to be made that the driving force will be half the sum of the calculated force at the base of the panel and the force at the top of the panel.

From NZS 1170.5, the part category of the wall panel and its connections loaded out-of-plane is P1, resulting in a part risk factor, R_p, of 1.0.

The horizontal force for the design of the face-loaded panel is given by:

$F_{ph} = C_p(T_p)\ C_{ph}\ R_p\ W_p$

where

W_p	=	the weight of the part
$C_p(T_p)$	=	the horizontal design coefficient for the part
C_{ph}	=	the part horizontal response factor
R_p	=	the part risk factor

$C_p(T_p) = C(0)\ C_{Hi}\ C_i(T_p)$

where

$C(0)$	=	the site hazard coefficient for $T = 0$ (see section 3.1 of NZS 1170.5)
C_{Hi}	=	the floor height coefficient for level i
$C_i(T_p)$	=	the part spectral shape factor at level i

For the wall panels loaded out-of-plane on the side walls:

$C(0) = C_h(0)ZRN(T,D) = 1.33 * 0.21 * 1.0 * 1.0 = 0.28$

$C_{Hi} = (1 + h_i/6) = 1 + 9/6 = 2.5$

The period of the panel in the out-of-plane direction determines the value of $C_i(T_p)$.

The frequency of the panel out-of-plane is calculated to be:

$$f = \frac{\pi^2}{2\pi H^2}\left(\frac{EI}{m}\right)^{0.5} = \frac{\pi}{2x9000^2}\left(\frac{0.28e8x1000x150^3}{3.6x12}\right)^{0.5} = 0.9Hz\ \ (T_p = 1.1\text{s})$$

Therefore, $C_i(T_p) = 2(1.75\text{-}T_p) = 2(1.75\text{-}1.1) = 1.3$

And $C_p(T_p) = 0.28 * 2.5 * 1.3 = 0.91$

$C_{ph} = 1.0$

Therefore, $F_{ph} = 0.91 * 1.0 * 1.0 * W_p = 0.91 * W_p$

and the face load on the panel at the top = 0.91 * 3.6 = 3.3 kPa.

At the base of the panel $C_p(T_p) = 0.28 * 1.0 * 1.3 = 0.36$.

Therefore, $F_{ph} = 0.36 * 1.0 * 1.0 * W_p = 0.36 * W_p$

and the face load on the panel at the bottom = 0.36 * 3.6 = 1.3 kPa.

The mean face load on the panel is therefore (3.3 + 1.3)/2 = **2.3** kPa.

Moment in the panel at mid-height, M_a^* = 2.3 kPa * 9^2 /8 = 10.1 kNm/m, assuming it is simply supported at the top and the bottom.

However, the bending moment strength of the section must be able to cater for moment magnification due to P-delta effects (see section 4.6).

The axial load at the mid-height of the section, P^* = 24 kN/m * 0.15 m thickness * 4.5 m high = 16.2 kN/m length of panel.

$$A_{se} = \frac{P^* + A_s f_y}{f_y} = \frac{16.2e3 + 113 * 5 * 500}{500} = 597\,\text{mm}^2$$

$$n = \frac{E_s}{E_c} = \frac{2.1e5}{27879} = 7.5$$

$$k = -\rho n \pm \sqrt{\left(\rho^2 n^2 + 2\rho n\right)} = -(0.0038 * 7.5) \pm \sqrt{0.0038^2 * 7.5^2 + 2 * 0.0038 * 7.5} = 0.21$$

$$I_{cr} = nA_{se}(d - kd)^2 + \frac{l_w kd^3}{3} = 7.5 * 597 * (75 - 0.21 * 75)^2 + \frac{1000 * 0.21 * 75^3}{3}$$

$= 45{,}249{,}793 \text{ mm}^2$

$$M^* = \frac{M_a^*}{1 - \left(\dfrac{5P^* H^2}{(0.75)48 E_c I_{cr}}\right)} = \frac{10.1}{1 - \left(\dfrac{5 * 16.2e3 * 9e3^2}{0.75 * 48 * 27897 * 45{,}249{,}793}\right)} = 11.8 \text{ kNm}$$

$\varphi M_n = \varphi Asfy(d - a/2)$

$a = (A_s f_y)/(0.85 f'_c b)$ = (113 * 500)/(0.85 * 40 x 1000) = 1.7 mm.

Therefore φM_n = 0.9 * 113 * 5 * 500 * (75 – 1.7/2) * 10^{-6} = 18.8 kNm/m > 11.8 kNm/m – okay.

Check that the vertical steel percentage, ρ, is less than the balanced condition.

$$\rho_b = \frac{0.85 * f_c' * \beta_1}{f_y}\left(\frac{0.003 E_s}{0.003 E_s + f_y}\right) = \frac{0.85 * 40 * 0.85}{500}\left(\frac{0.003 * 2.1e5}{0.003 * 2.1e5 + 500}\right) = 0.032$$

ρ = 0.0038 (from above) << ρ_b – okay.

With bolts at 1 m centres, the tension load on the bolts from the face-loaded wall panels is:

1.3 kPa * 9 m height / 2 + 2/3 * 2 kPa * 9 m height / 2 = 11.8 kN per bolt.

Therefore M20 bolts will suffice.

Step 11. Check the stability of the end wall panels under in-plane shear load. There are four conditions to check.

Condition 1: *H/t* = 9000/150 = **60** < **75**, therefore okay.

Condition 2: *kH/t* = 1.0 * 9000 / 150 = **60** < **65**, therefore okay.

Condition 3: Euler buckling stability check (see section 8.4 for the checking process).

Panel geometry: $\left(\frac{kH}{t}\right) = \left(\frac{1.0 * 9000}{150}\right) = 60$

Panel loads:

P = 0.35 kPa * 8.5/2 * 3 m wall length = 4.5 kN

0.5*W* = 0.5 * 9 * 3 * 3.6 kPa = 49 kN

$$\left(\frac{P+0.5W}{f_c^{'}A_g}\right)+\left(0.4\rho_t\frac{f_y}{f_c^{'}}\right)=\left(\frac{(4.5+49)*10^3}{40*3000*150}\right)+(0.4*0.0038)\left(\frac{500}{40}\right)=0.022$$

The intercept of the geometry value and corresponding load value may be plotted on Figure 12 to determine whether the wall element will be stable (see Figure 20 for plotted point). The combination is acceptable.

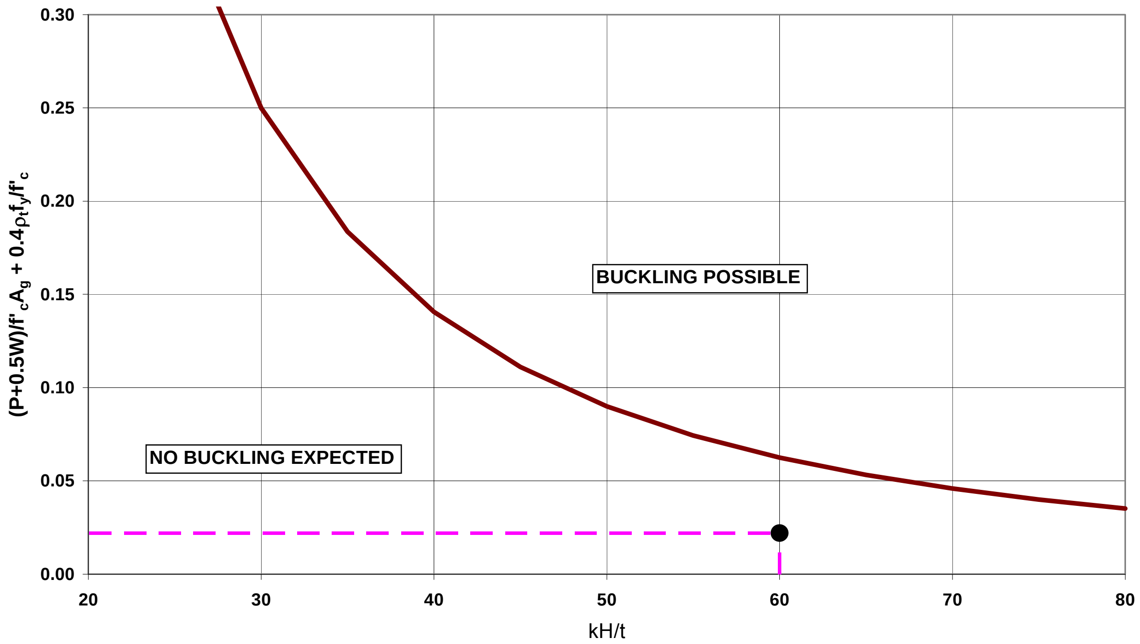


Figure 20. Euler buckling stability check

<u>Condition 4</u>: Lateral torsional buckling check

Wall geometry for case (a): $\left(\frac{kH}{t}\right)\left(\frac{L}{H}\right)^{0.5}=\left(\frac{1.0*9000}{150}\right)\left(\frac{3000}{9000}\right)^{0.5}=34.6$

Wall geometry for case (b): $\left(\frac{kH}{t}\right)\left(\frac{L}{H}\right)^{0.5}=\left(\frac{0.85*9000}{150}\right)\left(\frac{3000}{9000}\right)^{0.5}=29.4$

Case (a) loads:

P = 0.35 kPa * 8.5/2 * 3 m wall length = 4.5 kN

0.5W = 0.5 * 9 * 3 * 3.6 kPa = 49 kN

$$\frac{\rho f_y}{f_c^{'}}+\frac{P+0.5W}{f_c^{'}A_g}=\frac{0.0038*500000}{40000}+\frac{4.5+49}{40000*0.15*3}=0.05$$

Case (b) Elastic load case:

For the elastic case, $\mu = 1$ and $S_p = 1$. Hence, the seismic coefficient is 0.496 (see Step 6)

$M_e^* = 310$ kNm (from Step 8) factored by 0.496/0.27 = **569 kNm**

$$\frac{2.2M_e^*}{Lf_c^{'}A_g} = \frac{2.2*569*10^6}{3000*40*3000*150} = 0.023$$

The intercept of the geometry values and the two corresponding load values may be plotted on Figure 14 to determine whether the wall element will be stable (see Figure 21 for plotted points). The combinations are acceptable.

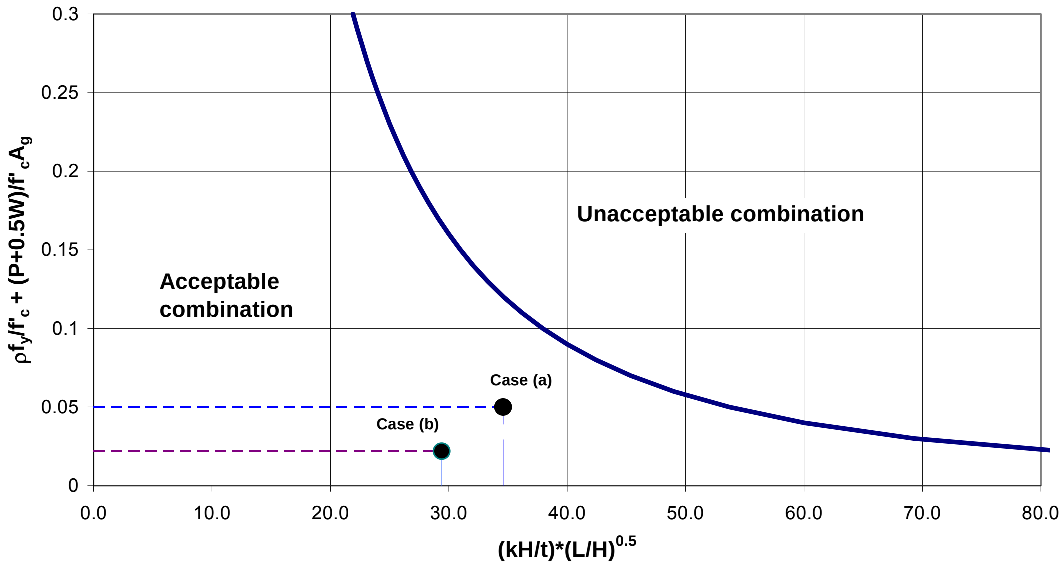


Figure 21. Vlasov lateral torsional buckling stability check

12.1.3 Seismic loads in the direction at right angles to the portals

The building has one side wall totally of precast panels and at the other side, two of the seven bays have precast shear panels. The panels are 9 m high with the eaves connection at the top.

To determine the design loads on the major wall, assume a rigid roof diaphragm and design on the basis of wall stiffnesses.

To determine the design loads on the minor wall, assume a flexible roof diaphragm and design on the basis of contributing areas.

Major Wall Design Process (assume rigid roof diaphragm)

(see section 8.2.1 for steps)

Step 1. Calculate stiffness of the side walls.

The procedure is as detailed in Step 1 of section 12.1.2.

The in-plane stiffness of a single 3 m long panel is 39000 kN/m.

Therefore, in-plane stiffness of the <u>major</u> wall = 59/3 x 39000 kN/m = **767000** kN/m.

And in-plane stiffness of the <u>minor</u> wall = 6 x 39000 kN/m = **234000** kN/m.

Step 2. Determine the centre of mass of the building (see Figure 22).

The roof weight = 0.35 kPa * 54 * 59 = 1115 kN (114 tonne)

The weight of the top half of the end walls

= 3.6 kPa * 4.5 m * 54 m * 2 ends = 1750 kN (178 tonne).

Combined weight of the roof and the top half of the end walls = 1115 + 1750 = **2865 kN**.

The weight of the top half of the major side wall

= 3.6 * 4.5 * 59 = **956 kN** (97 tonne).

The weight of the top half of the minor side wall = 3.6 * 4.5 * 3 m * 6 panels

= **292 kN** (30 tonne).

Total contributing weight = 2865 + 956 + 292 = **4113 kN** (419 tonne).

The distance of the centre of mass from the major wall is:

$$\frac{(1115+1750)*54/2+(292*54)}{4113}=22.6\,\mathrm{m}$$

Calculate the position of the centre of rigidity.

767000 * x = 234000 * (54-x) where x is the distance of the centre of rigidity from the major wall.

Therefore, x = 12.6 m.

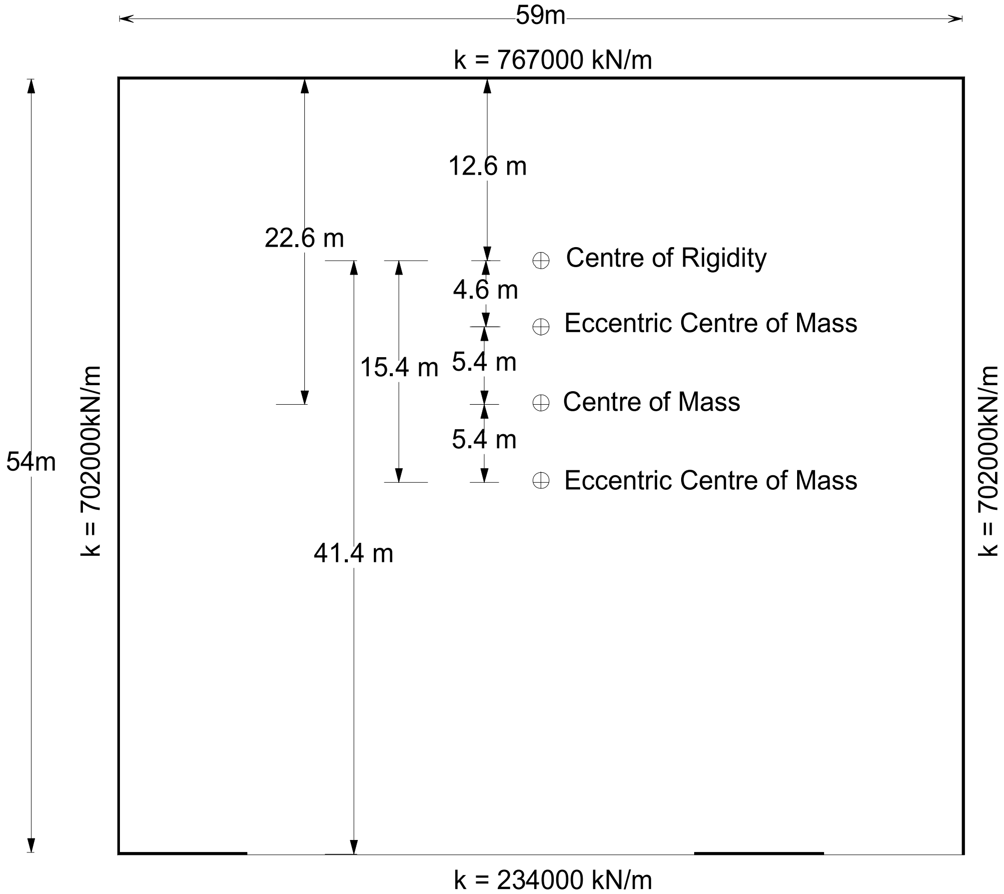


Figure 22. Example building plan showing dimensions and stiffnesses

Step 3. Accidental eccentricity is 0.1 * 54 = 5.4 m.

Step 4. Calculate the period of the structure.

$T_1 = 2\pi\sqrt{[(m_2 + m_3 + m_4)/(k_2 + k_4)]} = 2\pi\sqrt{[(419 / (767000 + 234000)]} = 0.13$ s

Step 5. Assign a ductility for the wall panel design.

Assign μ = 3, because the design is assuming a limited ductile response in both directions.

Step 6. Calculate the horizontal design action coefficient, $C_d(T_1)$ (ref section 3.1 of NZS 1170)

$C(T) = C_h(T)ZRN(T,D)$

$C_h(T) = 2.36$

$Z = 0.21$

$R = 1.0$

$N(T,D) = 1.0$

Therefore, $C(T) = 0.496$

$k_\mu = ((\mu\text{-}1)T_1/0.7) + 1 = ((3\text{-}1) * 0.13 / 0.7) +1 = 1.37$

Therefore, $C_d(T_1) = (C(T_1)S_p/k_\mu$ = 0.496 * 0.7 / 1.37 = **0.25**.

(Note for the case of μ = 1, $k_\mu = ((\mu\text{-}1)T_1/0.7) + 1$ = ((1-1) * 0.13 / 0.7) +1 = 1.0 and S_p = 1. Therefore $C_d(T_1)$ = 0.496.)

Step 7. Therefore the total base shear, V, is 0.25 * 4113 = **1028** kN and the shear from the roof and the top half of the end walls, V_{re}, is 0.25 * 2865 **= 716** kN.

Step 8. Distribute the shear from the roof and the end walls to the major wall on the basis of stiffness, allowing for accidental eccentricity, and design wall steel.

J_p = (702 * (59/2)^2) * 2 + (767 * 12.6^2) + (234 * 41.4^2) = 1,744,666

V_{major} = V * (767 / (767 + 234) + (V_{re} * 4.6) * 12.6 * 767 / J_p

= 1028 * 0.766 + 1028 * 44,455 / 1,744,666

= **813 kN (42 kN/panel)**

V_{minor} = 1028 * (234 / (767 + 234) + (1028 * 15.4) * 41.4 * 234 / 1,744,666

= **328 kN (55 kN/panel)**

Seismic shear from the self-weight of the panel = 3.6 * 9 * 3 * 0.25 = 24.3 kN.

The base moment for each panel is made up of moment from the self-weight of the panel plus a moment due to the horizontal load being introduced from the roof.

Therefore, the in-plane moment at the base of the major wall panel

= 42 * 9 + 24.3 * 4.5 = **487.4 kNm**, and

the in-plane moment at the base of the minor wall panel = 55 * 9 + 24.3 * 4.5 = **604.4 kNm**.

Design the major wall panel for the in-plane base moment.

Consider using **H12 bars at 200 mm centres** – 15 bars per panel (ρ = 0.0038) (near minimum steel content of 0.32%).

Therefore, $\varphi M_n = \varphi Asfy(d\text{-}a/2)$

$a = A_s f_y/0.85f'_c b$ = 14 * 113 * 500 / (0.85 * 40 * 150) = 155 mm

Therefore, φM_n = 0.85 x (113 x 14) x 500 x (1600 – 155/2) = 1023 kNm > 487.4 kNm – okay.

Check the shear strength in the plane of the panel for μ = 1.

The μ = 1 base shear may be determined by multiplying the μ = 3 base shear by the ratio of the seismic coefficients (see above):

V^* = 42 * 0.496/0.25 = 83.3 kN

The effective shear area, A_{cv}, is 0.8 times the length of the panel times the panel thickness

= 0.8 * 3000 * 150 = 360000 mm^2.

The maximum nominal shear stress, v_{max}, shall be not greater than $0.2f'_c$ (= 8MPa) nor 8 MPa (NZS 3101 clause 7.5.2).

The maximum shear able to be resisted by the concrete, v_c, is $0.17\sqrt{f'_c} = 0.17\sqrt{40} = 1.1$ MPa.

Therefore, $V_c = v_c * A_{cv} = 1.1 * 360000 * 10^{-3} = 396$ kN >> V^*.

However, a minimum quantity of horizontal steel is required by clause 11.3.10.3.8 of NZS 3101.

The minimum required steel area, $A_v = (0.7 * b_w * s_2) / f_{yt}$

where	b_w	=	the wall thickness (mm)
	s_2	=	the spacing of the horizontal steel (mm)
	f_{yt}	=	the yield strength of the horizontal steel (MPa)

The maximum horizontal steel spacing is 450 mm. If grade 500 bars are used, then:

$A_v = 0.7 * 150 * 450 / 500 = 94.5$ mm^2. Therefore **H12 bars at 450 centres** may be used for horizontal steel.

Step 9. Calculate the shear force in the wall/roof connection.

Design top connection for shear of 0.496/0.25 x 42 = 83.3 kN per panel (where 0.496/0.25 is the ratio between the $\mu = 1$and $\mu = 3$ seismic design coefficients).

With bolts at 1 m centres starting at 0.5 m from the ends, this means three bolts per panel, or 27.8 kN/bolt.

Therefore, three M20 bolts either cast-in or through the wall panel will suffice. The calculation of the strength of the bolts in shear is as per section 17 of NZS 3101 and has not been included here.

Step 10. Calculate the design face load on the panel and check steel content.

This process is identical to that in Step 10 above for the design in the orthogonal direction.

Also check that the vertical steel percentage, ρ, is less than the balanced condition.

$$\rho_b = \frac{0.85 * f_c' * \beta_1}{f_y}\left(\frac{0.003E_s}{0.003E_s + f_y}\right) = \frac{0.85 * 40 * 0.85}{500}\left(\frac{0.003 * 2.1e5}{0.003 * 2.1e5 + 500}\right) = 0.032$$

$\rho = 0.0038$ (from above) << ρ_b – okay.

With bolts at 1 m centres, the tension load on the bolts from the face-loaded wall panels is:

1.3 kPa * 9 m height / 2 + 2/3 * 2 kPa * 9 m height / 2 = 11.8 kN per bolt.

Therefore M20 bolts will suffice.

Step 11. Check the stability of the panel.

This step will not be completed until the design load test on the minor wall panels has been carried out because it may yield a more critical case.

Minor Wall Design Process (assume flexible roof diaphragm)

(see section 8.2.2 for steps)

Step 1. Calculate stiffness of the side walls.

This procedure has already been undertaken in the Major Wall Design Process above.

In-plane stiffness of the major wall = 59/3 x 39000 kN/m = **767000** kN/m.

In-plane stiffness of the minor wall = 6 x 39000 kN/m = **234000** kN/m.

Step 2. Determine the mass associated with the dynamic response of the roof, $m_{3,}$ (see Figure 9).

m_3 = ((0.35 * 59 * 54 / 2) + (3.6 * 4.5 * 54 * 2 / 4)) / 9.81 = 101 tonne

Step 3. Determine the masses that are expected to move with the major and minor walls, m_2 and m_4.

m_2 = ((3.6 * 59 * 9 / 2) + (3.6 * 54 * 9 * 2 / 8) + (0.35 * 54 * 59 /4)) / 9.81 = 170 tonne

m_4 = ((3.6 * 6 * 3 * 9 / 2) + (3.6 * 54 * 9 * 2 / 8) + (0.35 * 54 * 59 /4)) / 9.81 = 103 tonne

Step 4. Calculate the first mode period of the structure, T_1

$T_1 = 2\pi \sqrt{[(m_2 + m_3 + m_4) / (2\ k_2)]}$

$= 2\pi \sqrt{[(170 + 101 + 103) / (2 * 767000)]} = 0.1$ s

Step 5. Calculate the period of the roof structure, T_r

$T_r = 2\pi \sqrt{[(m_3\ ) / (2\ k_r)]}$

k_r is the shear stiffness of the roof. This may be calculated or may be assumed to be low if a flexible diaphragm is assumed.

Step 6. Calculate the ratio of the roof period, T_r, to the building period, T_1.

By assuming a flexible diaphragm, the period ratio will be greater than 2 and the minor wall is designed on the basis of tributary areas.

Step 7. The ductility factor, μ, for the wall design is 3

$C_d(T_1)$ = **0.25** (from Step 6 for the rigid roof case).

The roof weight is 1115 kN (from Step 2 for the rigid roof case).

The weight of the top half of the end wall is 1750 kN (from Step 2 for the rigid roof case).

Therefore the seismic force at the roof level at each side of the building is:

0.25 * (1115 + 1750) / 2 = **358 kN.**

For the minor wall, this force is shared between 6 panels.

Therefore, the force per panel is **60 kN**.

The self-weight of the panel is 3.6 kPa * 9 m * 3 m = 97.2 kN. Therefore, the seismic force from the self-weight of the panel, applied at the mid-height of the panel, is 97.2 * 0.25 = **24.3 kN**.

Therefore the in-plane moment at the base of the panel = 60 * 9 + 24.3 * 4.5 = **649 kNm** (compare 604.4 kNm for the rigid roof assumption and 569 kNm for the end walls).

Step 8. By comparison with the wall panel design for the end walls, it can be seen that both the major and minor wall panels, while having and M_e* greater than the end walls, will still be satisfactory with the same steel detailing as the major wall panel.

13. SELECTED REFERENCES

[1] Lander MRE. 2001. *Truss Modelling of Reinforced Concrete Cantilever Wall Behaviour.* A research report submitted in partial fulfilment of the requirements for the degree of Master of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

[2] Portland Cement Association. 2001. *PCA-WALL – Structural Analysis and Design of Slender Concrete Walls.* Portland Cement Association, Skokie, Illinois, USA.

[3] Salama, Ayman El-Sayed. 1995. *Seismic Analysis, Design and Qualification of Precast R/C Tilt-Up Walls.* Dissertation submitted in partial satisfaction of the requirements for the degree of Doctor of Philosophy, University of California, Irvine, USA.

[4] Brooks, Hugh. 1994. *Tilt-Up and Earthquakes – A Post-Northridge Assessment*. A handout provided at a seminar sponsored by the Tilt-Up Concrete Association, Mt Vernon, USA.

[5] Agbabian Associates. 1999. *Concrete Tilt-Up Warehouse Earthquake Damage Repair, Seismic Evaluation and Strengthening.* A post-earthquake inspection report from Agbabian Associates Inc, Pasadena, California, USA.

[6] Frohmader DL. 1991. '*Tilt-Up in Seismically Active Areas'*. Concrete Construction (August 1991).

[7] Brooks, Hugh. 2000. *The Tilt-Up Design and Construction Manual* (5th edition). Tilt-Up Concrete Association, Mt Vernon, USA.

[8] American Concrete Institute. 1997. *Practitioner's Guide to Tilt-Up Concrete Construction.* ACI, Farmington Hills, MI, USA.

[9] Restrepo JI, Crisafulli FJ and Park R. 1996. '*Seismic Design Aspects for Tilt-Up Buildings*'. Journal of the New Zealand Structural Engineering Society (9) 2: 9–24.

[10] Lemieux, Kevin, Sexsmith, Robert and Weiler, Gerry. 1998. '*Behaviour of Embedded Steel Connectors in Concrete Tilt-Up Panels*'. ACI Structural Journal (July-August 1998): 400-411.

[11] Brown, Barry. 1998. *Some Stability Considerations for Tilt-Up Precast Concrete Panels Under In-plane Seismic Loading.* Australasian Structural Engineering Conference, Auckland, New Zealand (October 1998).

[12] Azizinamini A, Glikin JD and Oestrele RG. 1994. *Tilt-Up Walls Test Results*. Portland Cement Association R&D Serial No. 1863.

[13] Brown, Barry. 1997. '*Some Stability Issues for Tilt-Up Precast Panels Under In-plane Seismic Loading*'. SESOC Journal (June 1997): 10(1).

[14] Concrete Institute of Australia. 1992. *Recommended Practice Design of Tilt-Up Concrete Wall Panels.* CIA, Australia.

[15] Lim L. 2000. '*Stability of Precast Concrete Tilt Panels in Fire'.* Fire Engineering Research Report 00/8. University of Canterbury, Christchurch, New Zealand.

[16] Ma M. 2000. *Methods of Joining Precast Components to Form Structural Walls.* Extracts from a Master of Engineering thesis, University of Auckland, New Zealand.

[17] Negus, Christine. 2000. '*Tilt-panel Apartment Pushes Limits of Technology*'. New Zealand Engineering (January 2000): 24–26.

[18] DiPasquale RA. 2000. '*Tilt-up Concrete Construction'*. The Construction Specifier (December 2000): 29–34.

[19] Simpson MS. 1982. *Test Report on Slender Walls*. American Concrete Institute, Southern California Chapter, and Structural Engineers Association of Southern California report of the Task Committee on Slender Walls.

[20] Weiler G. 1986. '*Connections for Tilt-up Construction'*. Concrete International (June 1986): 24–28.

[21] ICBO. 1997. *Uniform Building Code – Section 1914*. International Conference of Building Officials, Whittier, California, USA.

[22] Kripanarayanan KM. 1980. '*In-place Loads and Tilt-up Bearing Walls – A Design Approach*'. Concrete International (April 1980): 43–47.

[23] Weiler G and Nathan ND. 1980. '*Design of Tilt-up Wall Panels*'. Concrete International (April 1980): 48–51.

[24] Kripanarayanan KM. 1974. '*Tilt-up Load-Bearing Walls – A Design Aid'*. Portland Cement Association Engineering Bulletin. Portland Cement Association, Skokie, Illinois, USA.

[25] Adham SA. 1981. *Guidelines for Mitigation of Seismic Hazards in Tilt-up-wall Structures – Phase 1.* Agbabian Associates, El Segundo, California, USA.

[26] Adham SA. 1985. *Guidelines for Mitigation of Seismic Hazards in Tilt-up-wall Structures – Phase II – Final Report.* Agbabian Associates, El Segundo, California, USA.

[27] Adham SA. 1982. *Guidelines for Mitigation of Seismic Hazards in Tilt-up-wall Structures – Phase II – Test Plan - Progress Report*. Agbabian Associates, El Segundo, California, USA.

[28] Chiewanichakorn, Methee. 1999. *Stability of Thin Precast Concrete Wall Panels Subjected to Gravity and Seismic Forces.* A draft thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering, University of Canterbury, Christchurch, New Zealand.

[29] McMenamin, Andrew. 1999. '*The Performance of Slender Precast Reinforced Concrete Cantilever Walls with Roof Level Lateral Displacement Restraint Under Simulated In-plane Seismic Loading*'. Civil Engineering Research Report 99/4. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

[30] Glass J. 2000. '*Wall Panel Renaissance: Tilt-Up Concrete Construction'*. Proceedings of the Institution of Civil Engineers Structures and Buildings 140(3): 277–289.

[31] Clifton GC. 1996. '*Support of Portal Frame Fire Rated Vertical Spanning External Concrete Wall Panels in Severe Fires'*. Journal of the Structural Engineering Society 9(2): 26–36.

[32] Standards New Zealand. 2006. *Concrete Structures Standard Part 1 – The Design of Concrete Structures NZS 3101:Part 1:2006.* SNZ, Wellington, New Zealand.

[33] Park R. 1989. '*Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing*'. Bulletin of the New Zealand National Society for Earthquake Engineering 22(3): 155–166.

[34] Centre for Advanced Engineering. 1999. *Guidelines for the Use of Structural Precast Concrete in Buildings* (2nd edition). CAE, Christchurch, New Zealand.

[35] NZ Heavy Engineering Research Association. 1989. *New Zealand Structural Steelwork Design Guides – Vol 2*. HERA, Manukau City, Auckland, New Zealand.

[36] Standards New Zealand. 2006. *Concrete Structures Standard Part 2 – Commentary on the Design of Concrete Structures NZS 3101:Part 2:2006*. SNZ, Wellington, New Zealand.

[37] Whiteside M. 2000. *The Performance of Slender Precast Conventionally Reinforced Cantilever Walls Incorporating Steel Fibre Reinforced Concrete Under Seismic Forces.* Masters Thesis, University of Canterbury, Christchurch, New Zealand.

[38] Beattie GJ. 2004. '*Design of Slender Precast Wall Panels – Experimental Testing'*. BRANZ Study Report SR 129. BRANZ Ltd, Judgeford, New Zealand.

[39] Personal communication with Mark Williams of Stresscrete, Otaki, New Zealand.

[40] Salonikios TN, Kappos AJ, Tegos IA and Penelis G.G. 1999. '*Cyclic Load Behaviour of Low-Slenderness Reinforced Concrete Walls: Design Basis and Test Results*'. ACI Journal, 96(4): 649–660.

[41] Department of Building and Housing. 2004. *New Zealand Building Code*. DBH, Wellington, New Zealand.

[42] Sudarno I. 2003. '*Performance of Thin Precast Concrete Wall Panels Under Dynamic Loading*'. Research Report 2003-05. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

[43] Cement and Concrete Association of New Zealand. 2003. *Grade 500 Reinforcement Design Construction and Properties.* Seminar presented by CCANZ and Reinforcing New Zealand Inc, Wellington, New Zealand (September 2003).

[44] Standards Australia International. 2001. *Steel Reinforcing Materials AS/NZS 4671:2001*. Jointly published by SAI Ltd, Sydney, Australia and Standards New Zealand, Wellington, New Zealand.

[45] Davidson BJ. 2004. *Determination of Seismic Design Forces for Slender Precast Slab Structures.* Proceedings of the 2004 New Zealand Society for Earthquake Engineering Conference, Rotorua, New Zealand (March 2004).

[46] Lim L and Buchanan, A. 2003. '*Stability of Precast Concrete Tilt Panels in Fire*'. Journal of the Structural Engineering Society of New Zealand Inc 16(2): 44–54.

[47] Ma M. 2000. *Methods of Joining Precast Components to Form Structural Walls.* Master of Engineering Thesis, University of Auckland.

[48] Barber David J. 1994. '*Fire Resistance of Epoxied Steel Rods in Glulam Timber'*. Research Report 94-1. University of Canterbury, Department of Civil Engineering, Christchurch, New Zealand.

[49] Vlasoz VZ. 1959. *Thin-walled Elastic Beams*. Translated by Y Schechtman (1963). Israel Program for Scientific Translations, Jerusalem, Israel.

[50] Standards New Zealand. 2004. *Structural Design Actions Part 5: Earthquake actions – New Zealand NZS 1170.5:2004.* SNZ, Wellington, New Zealand.

[51] Standards New Zealand. 1997. *Concrete Construction NZS 3109:1997 – Incorporating Amendments 1 and 2.* SNZ, Wellington, New Zealand.

[52] Cement and Concrete Association of New Zealand. 2005. 'Slender Concrete Walls'. *CCANZ Information Bulletin: IB 82*. CCANZ, Wellington, New Zealand.

[53] Standards New Zealand. 2002. *Structural Design Actions Part 0: General Principles AS/NZS 1170.0:2004.* SNZ, Wellington, New Zealand.

[54] Standards New Zealand. 2002. *Structural Design Actions Part 1: Permanent, Imposed and Other Actions AS/NZS 1170.1:2004.* SNZ Wellington, New Zealand.

[55] Standards New Zealand. 2002. *Structural Design Actions Part 2: Wind Actions AS/NZS 1170.2:2002.* SNZ, Wellington, New Zealand.

[56] Davidson BJ. 2003. *Unpublished Report on Testing of Slender Precast Wall Panels.* Department of Civil and Resource Engineering, University of Auckland, Auckland, New Zealand.

[57] Heldt TJ and Mahendran M. 1998. *Full-scale Experiments of a Steel Portal Frame Building*. *Steel Construction* 32(4).

APPENDIX A: SUMMARY OF RESEARCH RESULTS

A1 Definition of "failure" for slender panels

It is important to define "failure" in the context of slender precast industrial warehouse walls. Failure is considered to be the point when the panel is no longer able to perform its function of resisting the applied loads. Under in-plane loading, this is often referred to as the point when the panel can no longer resist 80% of the peak load achieved in a cyclic loading test.

For out-of-plane displacement, "failure" is defined as the point when the stability of the panel is lost and collapse ensues. McMenamin [29] defines buckling as uncontrolled increase in out-of-plane displacement. Single layer reinforced slender panels are capable of displacing out-of-plane well beyond half their thickness because the resisting mechanism of the couple between the steel and the concrete compression block will resist the out-of-plane moment generated by the relatively small P-Δ effect. Aligned with the couple so formed, as the in-plane load increases, the cracks at the compression end of the panel close, causing the wall out-of-plane stiffness to be regained. Hence, an out-of-plane buckling failure is not easy to achieve.

However, to provide a conservative but common reference point for analysis of the experimental tests undertaken in the derivation of the hypothesis put forward in this Guide, a panel has been considered to "buckle" when the out-of-plane deflection at mid or around mid-height exceeds one-half of the wall panel thickness.

A2 Recently undertaken research

Little research had been undertaken overseas on the performance of in-plane loaded panels before they began to gain popularity in New Zealand in the 1990s. Since then, research has been undertaken at BRANZ and the Universities of Auckland and Canterbury. A range of wall types has been subjected to in-plane loading and the load-resisting behaviour observed.

This began with five 40% scale wall panels tested by McMenamin [29]. The walls were cast integrally with the foundation beams. All panels had a height-to-thickness ratio, H/t, of 50, the panels varied in length from 1 m to 2 m and the vertical steel contents were either 0.6% or 1.1%. All walls were singly reinforced. No lateral buckling failure was observed. The displacement ductilities ranged from 1.25 (0.5% drift) for the almost square specimen with 1.1% reinforcement ratio to 3 (0.6% drift) for the narrower and less heavily reinforced specimens. It should be noted that NZS 3101 [32] requires walls with >1% vertical steel to have this steel enclosed with lateral ties (see section 4.4), but in this test series this limitation was exceeded without the use of confining steel.

Chiewanichakorn [28] investigated the performance of four panels, all with H/t ratios of 75. The panels all had a height-to-length ratio of 3.75 and the vertical steel content was 1.7% in a single layer (see note above about confinement of vertical steel). Two panels had added dead weight applied eccentrically to the centre thickness of the panel. The connection of the panels to the foundation beam was by means of four starter bars concentrated at each end of the specimens. In two specimens the bars were grouted into corrugated metal ducts in the specimens. The other two specimens had welded laps above the foundation. In both cases, the ducts or laps were encircled with ties in an attempt to confine the joint zone. No lateral buckling failure occurred in three of the specimens up to the finish of the test at displacement ductility 4. However, the fourth unit, which had grouted ducts and an additional axial load, began to deflect out-of-plane significantly during the cycles to displacement ductility 2.5 (drift ratio = 0.9%).

BRANZ conducted tests on four 40% scale panels [38], all with H/t ratios of 62.5. The height-to-length ratio of all panels was 4.17. The first two panels had vertical steel contents of 0.44%, while the third and fourth had 0.71% vertical steel. An eccentric axial load was applied to panels 3 and 4 to simulate full roof contribution weight (i.e. no portals) and panel 4 was maintained at an out-of-plane displacement of 80 mm at the top during the test to model expected out-of-plane displacement under angular earthquake attack. As well, three full-scale elemental compression-tension tests were undertaken on sections of wall constructed to replicate the bottom corner of the full-scale wall. In each of these, tension load was applied to the reinforcing steel followed by compression load to the end of the concrete block. The load levels corresponded to alternating and increasing in-plane moments at the base of the prototype wall. The first two elements contained no strength-enhancing products, but the third contained spiral reinforcing around each of the two main reinforcing bars.

Scaled panel 1 was initially connected to the floor by a single layer of starter bars projecting from the side of the panel which proved to be too weak and the panel was subsequently anchored artificially at its base. The remaining three panels were fixed to a thickened floor slab with two rows of starter bars. Horizontal load was initially introduced to the top of the panel via two proprietary expansion anchors situated near the outer edges of the panel. The test rig simulated the influence of adjacent panels on keeping the loading beam horizontal and early in the test the anchors burst out of the panel. The remainder of the testing on that panel and subsequent panels employed a pinned joint at the centre width of the panel to apply the load.

Scaled panel 1 exhibited ductile performance up to a drift ratio of 3% with no sign of out-of-plane buckling. There was some spalling of concrete at the floor level and outermost vertical reinforcing bars failed in tension.

Scaled panel 2 remained planar up to 1.3% drift ($\mu \approx 5$), then began to deflect significantly out-of-plane between cycle peaks, as the loading direction reversed. There was a major crack across the wall width at about mid-height of the panel which, when coupled with a series of full width cracks at the wall base, was responsible for the majority of the out-of-plane displacement. The panel was very flexible out-of-plane at this point, but was not unstable because of the couple which formed between the vertical steel and the concrete compression edge.

The out-of-plane displacements of scaled panel 3 did not exceed 10 mm at any stage up to 1% drift ratio ($\mu \approx 2.5$). Beyond this point, the displacement extended to 20 mm at 2.5% drift ratio ($\mu \approx 6$). However, at the change in loading direction, when the panel axial loads were all being carried on the vertical reinforcing steel, the displacement reached 60 mm after cycles to 2.5% drift ratio. At the completion of cycling at 2.5% drift, the outermost bar at one end of the panel had fractured just above the floor and the next three bars were showing distinct buckling at the same position. Cracking was relatively uniform up the lower two-thirds of the wall, but the horizontal crack at the floor level opened the widest at peak loads.

Scaled panel 4 hysteresis loops suggest that there was ductile behaviour up to 2.5% drift ratio (when the test was stopped). However, out-of-plane displacement was only small until 1.5% drift ratio. Beyond that level, the displacement increased markedly, reaching about 50 mm at 2 m above the floor in the opposite direction to the initially applied top offset. The change in panel curvature was uniform over the height with no concentrated curvature, like that observed in scaled panels 1 and 2.

As expected, there was no stability failure of the small specimens. However, transverse cracks developed during the tension half-cycles which closed up again during the compression cycles. In all three specimens, when the concrete compressive strain reached approximately 0.003, crushing/shearing occurred and the load dropped off significantly. Inspection of specimens S1 and S2 revealed a longitudinal crack in the plane of the reinforcing bars. When the specimens were split open along the crack there was no visible sign of bar buckling but it is thought that the buckling, although small, was sufficient to spall the concrete. Specimen S3 demonstrated similar behaviour until failure by shear at one end of the unit. Once again, a longitudinal crack appeared on the top surface of the specimen but when it was split open the crack clearly did not penetrate the concrete core contained inside the spirals.

The University of Auckland conducted 13 in-plane shear tests on scale models of panels [No formal report]. The *H/t* ratio varied from 27 to 78 and the height-to-length ratio varied from 4.8 to 8.4. The axial load was varied and there were also variations in the steel content. Review of the raw test results showed that the walls generally remained stable up to at least 1.5% drift and displacement ductilities of 4. Cracking tended to be horizontal and uniform over the bottom half of the panels (i.e. no shear cracking).

Shake table testing was undertaken on two panels at the University of Canterbury [42]. Both panels were 2.8 m high by 900 mm wide and 47 mm thick. The *H/t* ratio was therefore 60 and the height-to-length ratio was 3.1 to 1. The longitudinal bars in the first panel continued through the junction with the foundation and were anchored in grouted ducts buried in the foundation, while in the second panel the longitudinal bars were terminated at the bottom of the panel and starter bars from the foundation were grouted into ducts adjacent to the longitudinal bars.

The physical modelling of the system was based on constant acceleration similitude. Both were subjected to a gravity load of 17 kN from a concrete block on the top of the wall to simulate the true axial stress at the base of the prototype wall. The difficulty with maintaining axial stress similitude between the prototype and the model at the base of the wall in this fashion is that the axial stress over the remainder of the height of the model wall is over-provided. This will result in a greater potential for out-of-plane buckling in the model and also a greater contributing inertial mass.

The first wall had a 1.27% vertical steel ratio and 0.8% horizontal steel ratio. The second had 0.54% vertical steel and the same horizontal steel. Both specimens were subjected to low acceleration level white noise in order to establish their first mode natural frequency before being subjected to scaled earthquake displacement records of increasing amplitude. These were the Taft record with a peak ground acceleration (PGA) of 0.2g, followed by El Centro N-S with a PGA of 0.4g, and Kobe with a PGA of 0.8g.

The first panel displaced to a peak drift of 0.9% (displacement ductility, μ, 2.7) when subjected to the Taft record. When excited by the El Centro record a peak drift of 1.6% was recorded ($\mu = 4.8$). Early on in the Kobe record the panel displaced out-of-plane approximately 200 mm above the foundation as the concrete spalled and the longitudinal bars buckled.

The second panel survived the Taft 0.2g record with evidence of minor cracking. In the early stages of the El Centro displacement record the starter bars failed abruptly in tension and the test was terminated. The first indication was that the vertical steel percentage may have been too small to resist the imposed bending moments, but close inspection suggested that the bars did not appear to exhibit sufficient elongation after yield (which may have been due to the tight confinement provided by the high strength grout in the ducts).

Theoretical studies have been made by Lander [1] to predict the response of thin rectangular walls under in-plane seismic loading. In his thesis, Lander notes that Salonikios et al [40] concluded from earlier studies that the relative contribution of flexure and shear to the lateral load behaviour of walls is generally governed by the height-to-length (*H/L*) ratio of the wall. Generally, it was found that if this ratio was less than 1, the behaviour tended to be shear critical. If it was greater than 2, the behaviour was flexure critical. In between these two ratios, mixed modes of shear and flexure may occur.

Lander developed a pass/fail test for predicting the out-of-plane performance of a wall. He suggested that if the slenderness of the wall (*H/t*) was more than 30 and the slenderness ratio (*kH/r*) was greater than 100, then the out-of-plane performance of the wall may be inadequate and an out-of-plane deflection analysis was required. The majority of slender precast walls on warehouse structures fall into this category. A rather complicated iterative procedure is hypothesised to consider the point at which a wall becomes susceptible to global buckling. Lander applied the theory to a number of test specimens investigated earlier at the University of Canterbury. All specimens had slendernesses and slenderness ratios that suggested an out-of-plane deflection analysis was required. However, in 7 of 11 specimens where theory predicted a buckling problem, the walls remained stable in the tests.

Lander further proposed a pass/fail test to determine whether or not a given wall may experience distress due to inelastic instability of the plastic hinge zone. A relationship was developed relating the critical displacement ductility for the wall to the slenderness, height-to-length ratio and the mechanical reinforcing ratio. Better correlation was achieved between the theoretical displacement ductility and the recorded displacement ductility at which instability occurred than for the global performance. The theory has been applied to the more recent panel test results from BRANZ [38] and the University of Auckland [56], and in all cases the theoretical prediction was conservative and may therefore not be useful.

APPENDIX B: PANEL STABILITY

B1 Development of theory for prediction of the stability of slender precast concrete panels

A slender panel would be expected to eventually become unstable out-of-plane under post-yield in-plane bending excursions. Two options exist for describing the relationship between the applied loads and the geometric properties of the wall panel. The first of these is the Euler buckling equation for either a diagonal compression strut from the top centre of the panel to the bottom corner (particularly applicable to panels with large height-to-length ratios) (see section B1.1). The second is the Vlasov/Timoshenko theory of lateral torsional buckling [49], which has been adapted to apply to slender precast walls as shown in section B1.2.

B1.1 Euler buckling of a diagonal "equivalent strut"

In this methodology, it is assumed that the panel's internal resistance to in-plane loading is provided by a simple compression strut approximately diagonally through the panel, with a force balance being provided by the vertical reinforcement, as shown in Figure 23. The lateral force applied to the panel, *F*, will be used to determine the required steel content and from this the value of *T* will be determined.

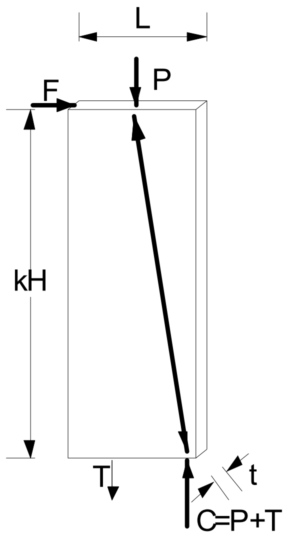


Figure 23 Forces in compression strut model

The Euler buckling axial load capacity, *C*, for a concrete compression strut is given by equation B1.

$$C = \frac{\pi^2 EI_{eff}}{H_e^{\ 2}}$$ **Equation B1**

where EI_{eff} = effective out-of-plane flexural rigidity of the strut
H_e = effective strut length when buckling out-of-plane

$H_e = kH$, where H is the height of the wall and k is the fixity (effective length) factor. The contributing axial load is composed of the axial load applied to the top of the wall (from the roof) and that balancing the tension from the yielding reinforcing steel. These walls tend to be subjected to low applied axial compression because, at most, they support light roofs and the eccentric application of this load causes little out-of-plane moment in the wall panel. Therefore, it is reasonable to assume that the compression strut acts near the toe of the wall (being the compressive stress block from the in-plane flexural action).

The critical section for the Euler buckling load is at mid-height of the wall panel. For a panel with vertical reinforcement uniformly distributed along the length of the wall, when the nominal moment capacity is reached at its base, the force at mid-height will be approximated by:

$0.4A_{st}f_y = 0.4\rho_t A_g f_y$ where A_s is the area of vertical steel and f_y is the yield strength.

At this height, the direct compression from the gravity load will be $P + 0.5W$, where W is the weight of the wall panel.

Hence, the Euler buckling equation may be re-written as:

$$P + 0.5W + 0.4\rho_t f_y A_g = \frac{\pi^2 EI_{eff}}{k^2 H^2}$$

If both sides of the equation are normalised by dividing by $f'_c A_g$, where f'_c is the concrete compressive strength and A_g is the gross cross-sectional area of the wall, the following equation is developed:

$$\left(\frac{P + 0.5W}{f'_c A_g} + 0.4\rho_t \frac{f_y}{f'_c}\right) = \left(\frac{\pi^2 EI_{eff}}{EI_g}\right)\frac{EI_g}{k^2 H^2 f'_c A_g}$$

Equation B2

where E is the modulus of elasticity of the concrete and I_g is the gross second moment of area of the panel bending out-of-plane ($I_g = Lt^3/12$ where L is the wall length and t is the thickness).

From clause 5.2.4 of NZS 3101, E is approximately equal to $750f'_c$ for the typical concrete strengths used for slender wall panel construction. However, since it is appropriate to use the dynamic modulus, a conservatively low value of $E = 900f'_c$ can be adopted.

If it is also assumed that the wall is uniformly compressed over one-quarter of its length and the remainder of the wall is cracked, then it may be assumed that $EI_{eff}/EI_g = 0.25$. While it is true that the remainder of the wall may be cracked, the reinforcing steel in this section is in tension, which has a stabilising effect on the panel as a whole. This situation was clearly observable during the laboratory investigations [38].

Thus, equation B2 can be rearranged to:

$$\frac{kH}{t} \leq \frac{15}{\sqrt{\left(\left(\frac{P + 0.5W}{f'_c A_g}\right) + \left(0.4\rho_t \frac{f_y}{f'_c}\right)\right)}}$$

Equation B3

The relationship in equation B3 is plotted in Figure 24 and is a possible check of a wall design to identify whether it is likely to buckle out-of-plane.

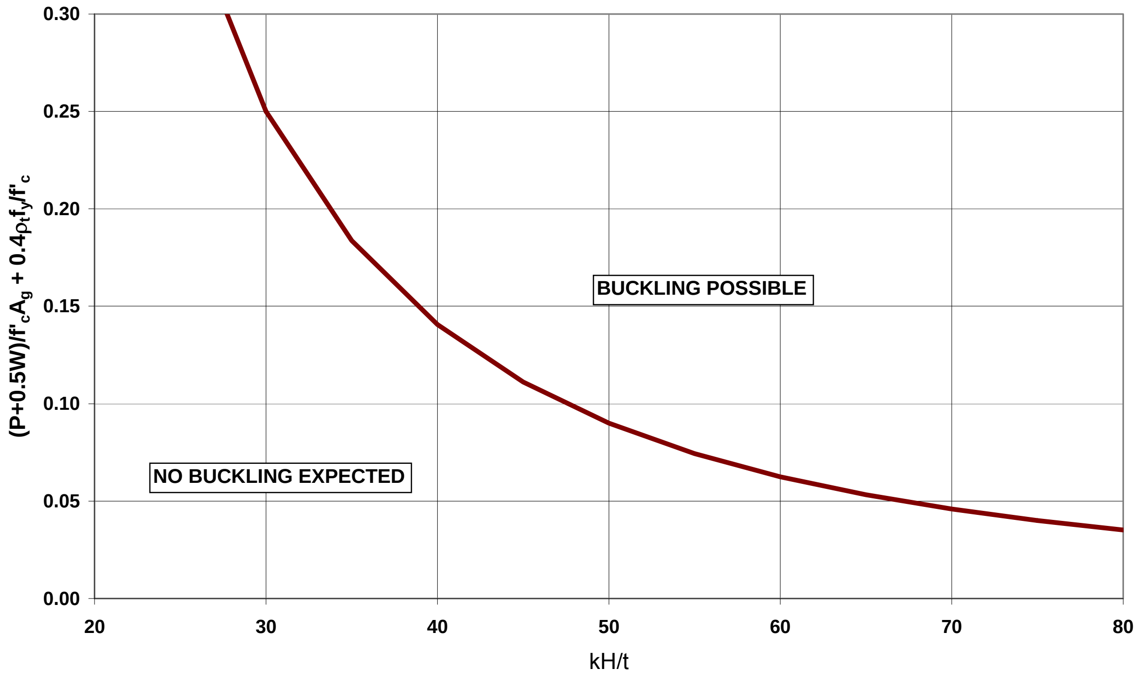


Figure 24 Euler relationship between wall slenderness and compression strut axial load

B1.2 Vlasov/Timoshenko lateral torsional buckling equation

McMenamin [29] developed a form of the Vlasov equation applicable to the special characteristics of slender concrete walls, where P_{crit} is the lateral force to cause buckling, as follows:

$$P_{crit} = \frac{0.6Et^3L}{H^2} \qquad \textbf{Equation B4}$$

Therefore, the critical moment on the panel is

$M_{crit} = P_{crit}\ x\ H \approx 0.5\ (P + 0.5W + A_{st}f_y)L.$

Assuming that $E = 900\, f'_c$

then equation B4 can be manipulated to give:

$$\frac{H}{t} \leq 16.5\sqrt{\frac{H/L}{\left((P+0.5W)/f'_cA_g + \rho f_y / f'_c\right)}} \qquad \textbf{Equation B5}$$

With the incorporation of the k factor and some rounding, the final form of equation B5 is:

$$\left(\frac{kH}{t}\right)\left(\frac{L}{H}\right)^{0.5} \leq \frac{12}{\sqrt{\left(\frac{P+0.5W}{f'_c A_g}+\frac{\rho f_y}{f'_c}\right)}}$$ **Equation B6**

The results of the experimental tests have been plotted on a graph of the above relationship in Figure 25. In the figure, the transition line from "acceptable" to "unacceptable" has been plotted for the case of an in-plane displacement ductility of 3, where it is expected that an out-of-plane hinge will have formed at the base of the wall (k=1). The symbols identifying the experimental results are solid if the out-of-plane displacement exceeded $t/2$ (an arbitrarily defined definition of lateral buckling) before the displacement ductility of 3 was reached. Note that the three cases where the out-of-plane displacement exceeded $t/2$ at μ=3 are on the unacceptable side of the line (open circles in Figure 25).

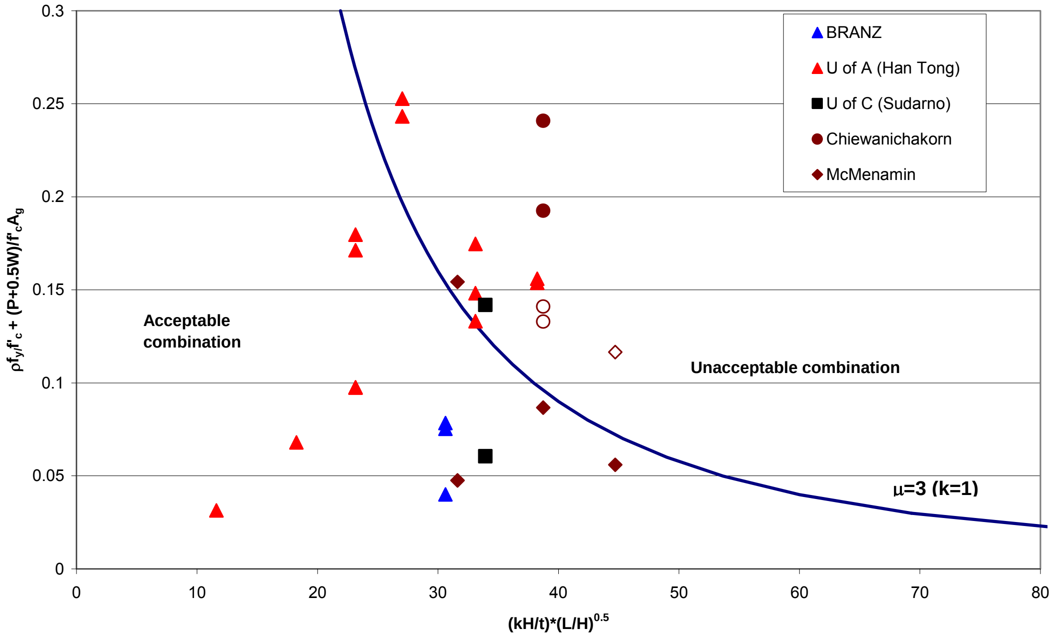


Figure 25 Vlasov/Timoshenko relationship between wall geometric properties and axial load

APPENDIX C: DETERMINATION OF FORCES ON BUILDING PARTS (NZS 1170.5 Section 8)

The Parts section of NZS 1170.5 is focused on multi-level buildings. To apply it to a single-storey slender panel structure requires a critical assumption to be made that the driving force will be half the sum of the calculated force at the base of the panel and the force at the top of the panel.

From the Standard, the part category of the wall panel and its connections loaded out-of-plane and supported at its base and top, is P1, resulting in a part risk factor, R_p, of 1.0.

Consider an elastically designed warehouse building with 8 m high slender boundary panels 125 mm thick on shallow soil in Auckland and with an estimated structure period of 0.4 s.

Panel design for face load

The horizontal force at the top for the design of the face-loaded panel is given by:

$F_{ph} = C_p(T_p)\ C_{ph}\ R_p\ W_p$

where

W_p	=	the weight of the part
$C_p(T_p)$	=	the horizontal design coefficient for the part
C_{ph}	=	the part horizontal response factor
R_p	=	the part risk factor

$C_p(T_p) = C(0)\ C_{Hi}\ C_i(T_p)$

Where

$C(0)$	=	the site hazard coefficient for T = 0 (see section 3.1 of NZS 1170.5)
C_{Hi}	=	the floor height coefficient for level i
$C_i(T_p)$	=	the part spectral shape factor at level i

For wall panels loaded out-of-plane:

$C(0) = C_h(0)ZRN(T,D)$ = 1.33 * 0.13 * 1.0 * 1.0 = 0.17

$C_{Hi} = (1 + h_i/6)$ = 1 + 8/6 = 2.3

The period of the panel in the out-of-plane direction determines the value of $C_i(T_p)$.

The frequency of the panel out-of-plane is calculated to be:

$$f = \frac{1.875^2}{2\pi H^2}\left(\frac{EI}{m}\right)^{0.5} = \frac{3.52}{2\pi 8000^2}\left(\frac{0.28E8x1000x125^3}{3.0x12}\right)^{0.5} = 0.34Hz\ (T_p = 1.05\text{ s})$$

Therefore, $C_i(T_p) = 2 * (1.75 - T_p)$ =2 (1.75 – 1.05) = 1.4

And $C_p(T_p)$ = 0.17 * 2.3 * 1.4 = 0.55

C_{ph} = 0.85 (μ = 1.25)

Therefore, at the top of the panel, F_{ph} = 0.55 * 0.85 * 1.0 * W_p = 0.47 * W_p

The horizontal force coefficient at the bottom of the panel is given by:

$C = C_h(T)ZRN(T,D)$ = 1.19 * 0.13 * 1 * 1 = 0.15

Therefore, at the bottom of the panel, F_{ph} = 0.15 * W_p

And the mean force on the panel = (0.47 + 0.15) / 2 * W_p = 0.31 * W_p

Pull-out load on fixings from panel face load

For the calculation of the load on the fixings at the top of the panel:

The part spectral shape coefficient, $C_i(T_p)$ = 2.0 (period of bolted connection < 0.75 s)

The ductility of the connection, μ = 1.25 (unless able to be shown that they are able to sustain not less than 90% of their design action effects at a displacement of greater than twice their yield displacement under reversed cyclic loading). Therefore, C_{ph} = 0.85.

Therefore, the loading on the connections at the top of the panel is calculated using:

$C_p(T_p)$ = 0.17 * 2.3 * 2.0 = 0.78

F_{ph} = 0.78 * 0.85 * 1.0 * W_p = 0.66 * W_p

Therefore, the loading on the fixings is:

0.66 * 0.125 * 24 * 8 / 2 = **7.9 kN/m.**