A GUIDE FOR DESIGNING AND DETAILING SLENDER PRECAST PANELS FOR EARTHQUAKE LOAD RESISTANCE

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ABSTRACT

This paper provides a background to the development of the research work, a summary of the research undertaken previously and in a recent collaborative series, and an introduction to the contents of a design guide for slender precast panels. The project evolved because engineers were provided with no guidance in the concrete standard, NZS 3101, for the design of panels if their height to thickness ratio exceeded 30 for non-load-bearing or 25 for load-bearing panels. Research has indicated that for single storey warehouse structures, where the axial load on the panels is less than 0.01f', the limiting H/t ratio may be substantially increased to 60 provided the connection details for the panels are properly designed to carry the post elastic loads without brittle failure.

INTRODUCTION

Over the last ten years or so palletised material handling and tall racking systems for bulk product storage, often at the place of sale, have evolved. This has lead to increasingly high single storey storage facilities, often constructed with thin precast concrete exterior wall panels (Photo 1). Thin boundary wall panels are not a new form of construction. However, their slenderness has been increasing as the panel height has grown to accommodate the higher stacking capabilities of the materials handling equipment (panels up to 12 m high are common), and to keep construction costs to a minimum.

Wall panel construction is relatively simple. The reinforcing is setup and the wall is constructed on top of the finished floor slab of the building, before being tilted into its final position (Photo 2).

The panels are usually simply supported at or near their top against earthquake and wind loading in and out of the plane of the wall and may be supported by a variety of systems at their base. They are sometimes designed to span horizontally between portal frame legs. Examples of some typical base fixings are shown in Figure 1.

Some tall panels found in service are very thin (in the range of 125 mm to 150 mm) compared to their height, giving slenderness ratios (effective height divided by thickness) of between 50 and 85 and they are reinforced with only one central layer of reinforcing steel.

This paper provides a background to the problem, a brief description of the research that has been undertaken and provides an introduction to a design guide being prepared for such panels.

BACKGROUND TO THE PROBLEM

Tilt-up wall panels are not new, but the existing design standards for them appear to have been based on the axial load carrying requirements of the panel, particularly under face loading and face load deflection.

In the late 1970's design recommendations were made in the USA [1] based on the design procedure used. For walls with 0.25% reinforcing steel in the span direction, if an empirical design procedure was used and the wall panels spanned vertically and carried vertical loads, the height to thickness ratio was confined to 25. If they spanned either direction with stiffened edges and carried no axial load, the height to thickness ratio was increased to 36. When an analytical design method was used which included P- effects the respective limits were 36 and 42.

New Zealand Standard Requirements

Generally, slender precast walls are designed for elastic response. However, the concrete standard, NZS 3101 [6], requires the formation of “an admissible mechanism of plastic deformation” in structures designed for elastic response under in-plane earthquake forces. That is, the walls are required to withstand plastic deformation without the risk of collapse.

The standard states that the "overall thickness of non-load bearing wall panels and enclosure panels shall not be less than 100 mm, nor less than 1/30 of the distance between supporting or enclosing members.” This means that for a wall 100 mm thick, the maximum distance between supports is 3 m and for a 150 mm thick wall, 4.5 m. The standard does, however, go on to state that the thickness limits talked of above “may be waived where.............rational analysis or test shows adequate strength and stability at the ultimate limit state”.

The difficulty was that there was little experimental data on which to base any criteria. Some designers were of the opinion that the limits in the New Zealand Concrete Standard were too restrictive, arguing that even when the wall is designed to

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respond to the design earthquake without damage (i.e. elastic response), the stresses in these panels are relatively low, being of a similar size to the long term gravity loads and, for this reason, the design should be treated no differently to a “non-earthquake” situation. Others point to the more restrictive NZS 3101 slenderness rules for shear walls designed to deform safely under earthquake attack and question the validity of the “respond without damage” approach for panels with only a central layer of reinforcement.

During the 1990’s and more recently, a number of investigations have been conducted at the Schools of Engineering at Canterbury and Auckland and at BRANZ [2],[3],[4],[5] with the aim of deriving an acceptable procedure for the safe design of slender wall panels.

PREVIOUS RESEARCH RESULTS

McMenamin [3] attempted to identify the key engineering issues which needed to be resolved for buildings with slender precast concrete walls. He tested scaled wall specimens with a slenderness ratio of 50 and with no superimposed axial load. The specimens were cast integrally with the base beam. Results from this programme suggested that:

1) Thin panels with a single central layer of reinforcement (singularly reinforced), even within the unsupported height to thickness limit of 30, may begin to fail when they are loaded in-plane in an earthquake because the concrete compression blocks near the ends of the wall can “split” at displacements only slightly more than those required to cause the reinforcing steel to yield (i.e. a ductility demand of 1 to 1.5).

2) If these panels are required to function as fire walls (under the fire burnout condition following an earthquake) then this type of damage needs to be prevented or controlled, otherwise the overall panel stability that is required for fire containment may be in jeopardy.

Chiewanichakorn [2] continued the study by investigating the performance of four scaled wall panels, all with a slenderness ratio of 75. His steel contents were greater than McMenamin’s and on two of his specimen’s extra axial load was applied to the top of the panel. The base connection details were typical of the type used in Canterbury in that bar laps were achieved with corrugated metal ducts in the specimens. He observed significant out-of-plane displacement in only one of his specimens once it had reached a displacement ductility of 4. This specimen had added axial load.

CURRENT RESEARCH

Over the past two years further collaborative investigations have been carried out at BRANZ and the universities. In all instances, scaled specimens have been built and subjected to in-plane racking loads. At the time of writing this paper full results from the universities have not been received but indicative results have been supplied by Auckland University.

BRANZ Research

BRANZ conducted tests on four 40% scale panels, 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. Over the height of the panels some of the longitudinal steel was terminated, at the same proportional heights as in the prototype panels. An eccentric axial load was applied to panels 3 and 4 to simulate full roof weight contribution (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 prototype 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 those expected in alternating and increasing in-plane moments at the base of a full scale wall. The first two elements contained no strength enhancing details but the third contained spiral reinforcing around each of the two main reinforcing bars.

Specimen 1 was initially connected to an unthickened floor by a single layer of starter bars projecting from the side of the panel at 400 mm above the base of the wall. The prototype design rested on piles at the ends of the panels and relied on passive soil bearing pressures coupled with a tie at slab level to resist out-of-plane flexure at the base of the wall. The detail proved to be too weak in the laboratory specimen and the panel had to be subsequently anchored artificially at its base to continue the testing. The remaining three panels were fixed to a thickened floor slab with two rows of starter bars, also typical of foundation connection details used in the Auckland area.

Proprietary expansion anchors are a popular means of attaching the tops of panels to an eaves beam because erection tolerances are then large. To model this type of connection, horizontal in-plane load was initially introduced to the top of Specimen 1 via two proprietary expansion anchors, each situated near the outer edges of the panel. The test rig simulated the influence of adjacent panels by keeping the loading beam horizontal during the cycling (Figure 2). As expected, early in
the test the top corners of the panel failed in tension as the anchors were loaded in shear. 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.

Specimen 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 eventually failed in tension.

Specimen 2 remained planar up to 1.3% drift, then began to deflect significantly out-of-plane between cycle peaks to the point of almost becoming unstable. There was a major crack across the wall width at about mid-height of the panel and this coupled with a major crack at floor level was responsible for the majority of the out-of-plane displacement (Photo 3).

The out-of-plane displacements of Specimen 3 did not exceed 10 mm at any stage up to 1% drift ratio. Beyond this point, the displacement extended to 20 mm at 2.5% drift ratio. However, at the change in loading direction, when the panel axial loads were all being carried on the vertical reinforcing steel, the out-of-plane displacement reached 60 mm after cycles to 2.5% drift ratio. At the completion of these cycles, the outermost bar at one end of the panel had fractured just above the floor and the next three bars were buckling at the same position (Photo 4). Cracks were mainly horizontal, dipping down at the centre width, and were relatively uniform up the lower two thirds of the wall. The horizontal crack at the floor level opened the widest at peak loads.

Specimen 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 (Figure 3). Beyond that level, the displacement increased markedly, reaching about 50 mm at 2m above the floor in the opposite direction to the initially applied top offset when unloaded at cycle direction changes (Figure 4). The change in panel curvature was uniform over the height with no concentrated curvature, like that observed in specimens 1 and 2.

Only indicative results have been made available from the University at the time of writing this paper, but the indications are that none of the panels buckled severely out-of-plane until they had been cycled to more than 2% drift.

University of Canterbury Research

At the University of Canterbury, wall panel specimens are being subjected to real time shake table testing and the results compared to slow cyclic testing on identical panels. The aim of this work is to confirm that the results from the slow cyclic testing at BRANZ and the University of Auckland will be applicable in real-time earthquake events. Testing was due to begin at the time of submission of this paper.

The panels to be tested have an H/t ratio of 60 and a height to length ratio of 3.1. The base fixing details are typical of Canterbury construction methods, where starter bars from the foundation are grouted into corrugated metal tubes in the panels.

DESIGN GUIDE

The design guide begins by identifying the typical features of slender precast wall panels and situations where they will be utilised. Such features include:

- A single layer of reinforcement
- Little or no direct connection between panels
- Low axial loads
- Lateral load resistance is provided by in-plane action
- Roof diaphragms transfer forces to the in-plane loaded walls

Rules are being developed for limits on such parameters as the H/t ratio, axial load, drift limits, steel percentages and concrete strengths for the assumed system ductility.

Provided the panels are supported at their top, test results have shown that in-plane drifts of up to 1.5% can be accommodated before out-of-plane buckling features.

Analysis procedures for buildings utilising slender panels as their seismic resisting elements will decide the detail required in the design. Dynamic computer analyses have shown [7] that the relative stiffnesses of the walls (in-plane), the portal frames (if present) and the roof diaphragm have a significant influence on the demands placed on the wall panels and on their connections. The panels are very flexible out-of-plane and their stiffness in this direction can therefore be ignored.
From the analyses, Davidson [7] concluded that two simple approaches could be used for the design of structures with slender precast walls. The first of these makes a conservative assumption that the roof diaphragm is rigid and the second requires the flexibility of the roof diaphragm to be calculated.

If individual panels have a height to length ratio that is three or more then flexural behaviour dominates and effective ductilities greater than two may be possible to achieve. It is therefore suggested that a maximum ductility of three can be assumed if the H/t ratio is three or more. For walls with H/t ratios of one or less use a minimum ductility of one. Between these two limits; the assumed ductility is equal to H/t.

**Structure Analysis with Assumed Rigid Roof Diaphragm**

An estimate of the structure period is made taking account of proportions of the structure component masses and from this the base shear coefficient for the resisting wall line is calculated. For the design of the connections transferring the shear forces into the resisting panels, an amplification factor of three times the base shear coefficient is proposed times the contributing mass from the roof and the face loaded panels. The connections between the roof and the face loaded panels are also designed for tension forces equivalent to the mass of the upper part of the face loaded wall multiplied by the base shear coefficient and an amplification factor of three. If portal frames are expected to work with the slender panels to resist the loads, a minimum stiffness limit is required for the portals.

**Structure Analysis with Assumed Flexible Roof Diaphragm**

For this procedure the period of vibration of the roof structure is calculated. The ductility of the resisting wall is limited to a maximum of two and the system ductility is determined from a derived relationship between the period of the roof and the ratio of the roof mass to the total structure mass. The method then calculates the base shear coefficient for the resisting wall line using the derived system ductility. The roof connections are designed similarly to the rigid roof diaphragm case but the amplification is calculated taking into account the period of the structure and the roof. The portal stiffness requirements are the same as for the rigid roof diaphragm case.

**Panel Design**

For the calculation of panel stiffness in the structure analysis, an effective EI of the cracked panel section is required. NZS 3101 recommends 0.25I_g, where I_g is the moment of inertia of the gross concrete section at the ultimate limit state. At the service limit state and elastic response, the standard recommends the use of I_g, reducing to 0.5I_g at a ductility of three. Results of the recent testing undertaken at BRANZ suggest that these values are too large and more appropriate ratios for the service condition and ultimate condition are 0.25 and 0.05 respectively.

The Guide will also contain procedures to be used when designing panels using the strut and tie method. This method is particularly useful for the consideration of panels with penetrations.

**Connection Design**

**Eaves Connection**

Careful consideration needs to be given to the design of the eaves connection because this is required to transfer loads from the face loaded panels to the eaves beam and to transfer shear loads from the eaves beam to the shear wall panels. It is also required to accommodate in-plane displacements that may occur as the panels rotate under the in-plane loads without failing.

As mentioned earlier, the design forces are amplified to ensure that the connections do not fail prematurely as their integrity is critical to the safe response of the panels.

Example details of suitable connections are provided in the design guide.

**Foundation Connection**

Various types of foundation connection have been used in the past. Some of these are presented in Figure 1. In the South Island, the grouted starter bar option is preferred while in the North options that involve horizontal starter bars are more popular.

Consideration needs to be given to the transfer of the shear forces and in-plane bending moments at the base of the panels, once again to ensure that the failure mechanism is confined to the panel. Similarly, the strength of the foundation needs to be sufficient to resist the potential over-strength of the wall panel. Example calculations will be provided in the Guide.

Laboratory testing [2] has shown that the spaced bar laps that occur with grouted tubes may contribute to local concrete crushing failure. Guidance will be provided on steel reinforcement ratio limits to prevent this from occurring.

**Panel to Panel Connections**

Very often the panels stand in isolation with respect to the adjacent panels, with the only link being the common foundation and eaves beam. Typical
Details will be provided for panel to panel connections and calculation procedures for the detailing of the connection to ensure that differential movement between the panels will not cause local cracking.

**Non-structural Limits**

The design guide contains a section on panel design limits which are not for in-service structural reasons but which nevertheless must be considered.

The New Zealand Building Code [8] requires that adjacent household units and other property be protected from horizontal fire spread by thermal radiation or structural collapse. In industrial buildings, the concrete wall panels on the boundary must not collapse outwards, thus endangering the lives of fire fighters and damaging adjacent property.

NZS 4203 [9] states that for during fire emergency conditions when the structure is subject to elevated temperatures the likes of boundary walls must be designed for a combination of dead and live load. Further, it states that wall elements that could collapse outwards must additionally be able to resist a face load of 0.5 kPa on the wall unless a detailed analysis taking into account the effects of elevated temperatures is undertaken. It is most likely that designers will follow the simple face pressure option.

Designers are reminded that the effect of temperature differentials through the panel thickness cause panel curvature. Particularly, if two adjacent panels are subjected to different temperature gradients because of partial shading, there may be significant differences in their curvatures. Waterproofing details need to be designed to accommodate the differential movements.

**CONCLUSIONS**

The problem with the lack of design guidance for slender precast panels has been discussed and a summary of recently undertaken research has been provided.

The research has indicated that for single storey warehouse structures, where the axial load on the panels is less than 0.01f’cAg the limiting H/t ratio may be substantially increased to 60 provided the connection details for the panels are properly designed to carry the post elastic loads without brittle failure.

Details are given of the information that will be included in a guide for the design of slender precast wall panels which is expected to be published before the end of the year.

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**REFERENCES**


Photo 1  Example warehouse structure with thin precast concrete exterior walls

Photograph 2  Cured panel being lifted from its casting site to its final position
Photograph 3  Edge view of Specimen 2 from the side and above in the latter stages of the test

Photograph 4  Damage at base of Specimen 3 at test completion (Note that the concrete has been removed in the right side image to expose the bent bars)
Figure 1

Typical examples of base fixity for wall panels

Note that the tilt-up panel steel has been omitted for clarity but is generally a single layer of steel at the centre.
Figure 2 Test rig setup for testing Specimen 1 (Note that the sloping side struts were not used for Specimens 2 to 4 because load was introduced via the single loading pin)
Figure 3  Plots of the deflected shape at positive loading cycle peaks
Figure 4 Plots of deflected shape at zero loading after the positive cycle peaks