REPORT TO DEPARTMENT OF BUILDING AND HOUSING
REVIEW OF DESIGN AND CONSTRUCTION OF SLENDER PRECAST CONCRETE WALLS
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1. INTRODUCTION TO REVIEW

The initial catalyst for this Review was the "Open Letter to IPENZ Regarding the Parlous State of the Structural Engineering Profession and Construction Industry in New Zealand" written by John Scarry and forwarded to Building Industry Authority in December 2002. [1] In this letter, Mr Scarry identified precast concrete design and construction as a general area of concern and connections between panels as a particular concern. He referred to tilt up panel buildings performing poorly in California in recent earthquakes and in tests in British Columbia. Precast concrete was one area of technical concern raised by Mr Scarry amongst many others, together with his concerns about professional standards of design and construction.

BIA held discussions with IPENZ and subsequently commissioned Sinclair Knight Merz (SKM) to review Mr Scarry's Open Letter and identify the areas of technical concern raised therein, which warranted further action. Simultaneously IPENZ formed a Task Committee to investigate the issues of professional practice raised. [3] SKM reported to BIA in December 2003. The SKM Report [2] identified four areas of specific technical concern including Slender Precast Concrete Walls (SPCWs) as an area of significant concern, which should be investigated further.

The SKM Report identified a particular type of wall panel which has been causing concern within the Structural Design profession dating from around the mid 1990s. These wall panels are used in high stud single-storey "industrial" buildings and they carry horizontal loads both in plane and out of plane. Panels vary in thickness from 120 to 200mm; they are 2.5 to 3.0m long, up to 12m and occasionally 15m high; they have a cantilever base foundation and are laterally supported by a steel eaves tie beam. The concerns focus on their slenderness and the possible buckling failure due to in-plane loading in the event of a major earthquake. SKM also noted that there is a considerable difference of opinion within the profession about the justification of this concern. This Review has also identified the behaviour of SPCWs in the event of fire as another related design concern.

This Review was commissioned in June 2004 by the BIA (Building Industry Authority). On 30 November 2004 the BIA's functions were transferred to the newly established Department of Building and Housing (the Department). This final Review Report of August 2005 is therefore addressed to the Department.

Publication of this Report has been withheld pending the development and publication by CCANZ of a design method for SPCWs incorporating a rational approach to fire stability. This design method was released by CCANZ in May 2005 and published in draft form in their information Bulletin IB82, Slender

Concrete Walls. This method is recommended to designers and is reviewed in some detail in Appendix IV of this Report.

2. OBJECTIVES OF REVIEW

The general brief was to review the current situation with regard to SPCWs including the design, construction, research and standards development, and to recommend actions needed by various industry sectors to address concerns identified. In detail this has involved:

2.1 Determining current practice for:

a. Structural systems utilising SPCWs
b. Typical panel dimensions
c. Typical connection details
d. Design criteria
e. Observation of construction
f. Construction methods and standards
g. Building consents

2.2 Identifying significant concerns amongst the design engineers, territorial authority engineers and contractors

2.3 Reviewing recent research programmes on the behaviour of SPCWs and identifying further research if appropriate

2.4 Investigating the current status of standards development in regard to SPCWs

2.5 Recommending appropriate responses to the Department to any concerns or problems identified

3. METHODOLOGY OF REVIEW

The methodology followed has been as follows:

3.1 Review of the Scarry Open Letter, SKM Report, IPENZ Task Force Report and other relevant literature

3.2. Preparation of a Questionnaire (See Appendix II)

3.3 Identification of Survey participants including:

a. Design engineers active in designing buildings utilising SPCWs
b. Territorial authority engineers from areas with significant numbers of relevant buildings
c. Research engineers involved in the research of SPCWs
d. Contractors and Precast manufacturers active in the field

3.4 Circulation of Questionnaire followed by interviews in Christchurch, Wellington and Auckland (See Appendix III for participants). Efforts were concentrated in these three metropolitan areas because of time constraints and the concentration of industrial buildings. As Section 4 below indicates, Auckland is the major industrial centre with the much greater concentration of the type of building that has raised concerns. Christchurch is also a significant industrial centre but many of the buildings using SPCWs are of more modest size and tend to have more “traditional” structural systems. Wellington has much less industry and therefore fewer buildings using SPCWs. Several telephone interviews were conducted with participants outside the three main centres following circulation of the Questionnaire.

4. RESULTS OF REVIEW

4.1. Structural Systems.

Four structural systems have been identified, which are widely used with SPCW panels. For convenience, they will be described as Types I to IV, and are described below. There is a marked geographical variation in their usage.

Type I is the traditional portal frame tilt panel building with wall panels supported by the portal frame columns. This structural system is the preferred system in the South Island.

The Type II, III, and IV buildings have been developed in the Auckland area in the last 15 years and all employ precast wall panels which can be precast in a factory, or site cast. Factors that have encouraged their development, have been the market demand for larger floor plan, higher stud height industrial buildings, a marked preference for factory precasting, the less rigid regulatory environment introduced by the Building Act in 1991, and associated relaxation of fire rating requirements. The lower seismic loads in Auckland help to make these systems more viable.

The wall panels used in these buildings have “grown” higher and thinner, and this has aroused concern in the design profession about their behaviour and possible failure in a major earthquake or fire. The majority of industrial buildings in Auckland have one of these structural systems.

a. Type I

The traditional portal frame tilt panel building is still widely used for smaller industrial buildings. The portal frames are generally spaced at 6 to 8 metre centres and the panels span between the portal legs under face loads and may have additional support at floor level. Walls are typically 6m high. The face loads associated with stability of the boundary wall during a fire, often referred to as the after fire loading, is usually resisted by encasing the portal leg to allow cantilever action. Typically a lightweight roof is supported by cold rolled steel purlins and a roof bracing system will be included to transfer face loads from the end walls to the side walls. Figure 1 shows this system as the single panel option.

b. Type II

As noted above demand developed for wider spacing of frames and higher knee heights during the 1990s so wall panels spanning between frames under face loads became impractical to design and too heavy to lift into position. Hence a system was developed that uses several wall panels per bay. These panels are typically 2.5 to 3.0m wide, which allows for off site casting, up to 12metres high and occasionally 15m. They have a cantilever base foundation and an eaves tie beam and usually no structural connection between panels at the vertical joints. The structural frames are spaced up to 15 metres apart and the roof is lightweight supported on cold rolled steel purlins. A roof bracing system transfers lateral face loads from the end walls to the longitudinal side walls. Figure 1 shows this system as the multiple panel option. With higher knee heights and wider frame spacing Type III structures may well prove more economical.

c. Type III

This type of structural system utilises frames that carry only gravity loads and have no external columns. The wall panels support the outer ends of the rafters. Lateral loads are carried by roof bracing in both directions, transferring them to respective “side walls”. Figure 2 shows this system.

d. Type IV

These buildings rely on SPCW panels cantilevering from their foundations to resist face loads and therefore do not require any roof bracing or frames to resist lateral loads. Hence rafters are supported by wall panels as in Type III buildings. Generally, the panels are a maximum of 6m high because of strength and flexibility limitations. Figure 3 shows a typical building using this system to resist transverse lateral loads, but with roof bracing to transfer longitudinal lateral loads to the side walls.

4.2 Wall Panel Dimensions and Height/Thickness (H/t) Ratios

a. Panel thickness and H/t ratio

Prior to the Building Act of 1991 and the rationalisation of fire design requirements, boundary walls were typically 175mm thick to provide a four-hour fire resistance rating. Typically today precast boundary wall panels are 150mm or 120mm thick with some higher panels 175mm and 200mm. Wall panel heights are frequently 10m and can extend to 12m and occasionally 15m. Hence H/t ratios of 67 are common and can exceed 80. Although written for multi-storey shear wall panels, the current NZ Concrete Standard NZS 3101:1995 limits the wall thickness, t, to 1/30 of the distance between supporting members, except clause 12.3.2.4 which allows relaxation where “rational analysis or test results show adequate strength and stability at the ultimate state” . There are no well established methods of rational analysis relating to the SPCWs being used, so it has been up to the design engineers to use their own judgement, assumptions and experience. Similarly, testing of panels with H/t ratios >30 for in-plane loads has been very limited until recently and in some cases results have yet to be published. Research results are summarised in section 6 of this report.

Clearly many design engineers have been prepared to exceed the empirical limits of the current Concrete Standard in regard to H/t ratio in the belief that it is overly conservative. As discussed in Section 6 below, recent test panel results tend to confirm this judgement.

Various design engineers interviewed use personal limits for SPCW H/t ratios ranging from 48 to 80.

After the survey interviews were completed in July 2004 the Concrete Standard DZ3101 Committee published its draft chapter 11 covering walls in September 2004. This draft included a revised maximum H/t ratio of 75 to prevent lateral torsional buckling under in-plane seismic loads, which recognises the upper limit of walls tested to date. It should also be noted that this limit may be too high to ensure stability of external walls during fire, as discussed in section 4.4.d of this report.

b. Type I Wall Dimensions

Wall panels are generally 6m to 8m long and typically 6m high, although can be higher if sufficient crane capacity is available. It is, however, clear that they are generally relatively squat; H/L ranging up to 1.5. Generally these panels are designed to span between portal legs and are not a concern in regard to slenderness and potential buckling because of the restraint provided.

c. Type II and Type III Wall Dimensions

Wall panels are typically 2.5 to 3.0m long and generally 6m to 12m high. Occasionally they are up to 15m high. Thus in plane they are relatively slender with H/L of 3 to 5. These are the panels that raised concern regarding their stability during major earthquakes.

d. Type IV Wall Dimensions

Walls carry all face loads as cantilevers and this dictates a maximum height of around 6m. Panel length depends on whether the panels are cast off site, up to 3.0m, or on site where length will be dictated by site logistics.

4.3 Connection details

a. Type I Buildings

Generally the main structural connection is between the wall panel and the supporting portal frame leg. Weld plate connections have been used for many years, cast-in socket connections and drill-in anchors are common in association with steel bracket connection to the portal flange and threaded reinforcing bars for concrete encasement. Opinions vary widely on the merits of the various systems. Weld plates cause concern amongst some engineers because of construction tolerance and cracking associated with welding or shrinkage. Sophisticated clamp connections are used, particularly in Christchurch, to resist face loads but allow movement due to shrinkage and temperature. A typical clamp connection is shown in Figure 10.

Connections between wall panels and floor slabs include simple seating, weathered seatings, reinforcing ties into the slab and dowel connections often using Drosbach ducts.

b. Type II Buildings

These buildings, and Types III and IV, require a cantilever base to the wall panels. The two common details involve:

1. A tie bar from the panel into the floor slab, and a footing typically 800 to 1000 below the floor slab, which the panel bears on. A base moment is developed between the floor tie and the friction, or ‘adhesion’ on the underside of the footing. See Figure 4.
2. The floor slab is thickened against the wall panels and two reinforcing ties are provided to develop a base moment. These reinforcing ties are provided either by casting them into the side of the panel base, or by providing cast-in anchors. See Figure 5 and Figure 6, which is a variation involving a separate L footing.
3. The other main connection is this type of building and Type III buildings is the panel-to-eaves tie beam. The beam is normally a channel on the flat spanning horizontally between frames. Connections to panels, typically two or three per panel, are either drill-in anchors, cast in anchors or weld plates. A typical connection is shown in Figure 7 and a superior type in Figure 8. The eave beam is connected to the frames by a bolt cleat. End wall panels usually support an angle with purlin cleats, which is fixed to the panels in the same manner as the eave tie beam. Corner panels are usually connected via the eave beam and end wall angle, and frequently have an additional mid-height fixing to assist alignment.

c. Type III Buildings

As noted above, SPCW panels in these buildings have cantilever foundations and eaves tie beams. The connection between the frame rafter and supporting wall panel is typically a substantial weld plate/bolted cleat as in Figure 9.

d. Type IV Buildings

These buildings have the same cantilever foundations as the Type II and III buildings and a connection to transfer gravity loads from rafter to supporting wall panels, as described for Type III above.

4.4 Design criteria

a. Face loads

Generally the thickness of SPCWs is dictated by the Fire Resistance Rating requirements and reinforcing is determined by face loads of wind, earthquake or the fire stability loading of 0.5 kPa. Earthquake loads should include an appropriate ductility factor.

Clearly, deflections of Type IV buildings that are solely reliant on the cantilever action of the wall panels can be substantial. The behaviour of these buildings must be carefully considered with regard to P-delta effects, potential yielding under earthquake loads and their lack of redundancy.

b. In-Plane loads

Typically, in-plane loads on each panel are small because there are a large number of wall panels to share the load. Most designers assume elastic behaviour and adopt a nominal ductility factor of 1.25. Furthermore, they believe because stresses additional to self-weight are so small, the possibility of a buckling failure is correspondingly low

There are numerous cases in Type I, II and III buildings where there are only a few panels available to carry in-plane loads because of wall openings or the use of light-weight cladding. Thus higher stresses result. Typically design engineers modify their design to meet these situations by thickening panels, connecting several panels or introducing steel cross-bracing. Alternatively, they can check by rational analysis whether the panels are adequate.

As discussed below, in-plane stresses are also increased by gravity loads applied to panels.

c. Gravity loads

Most SPCW panels in Type I and II buildings carry no gravity loads except their self-weight. End wall panels usually carry a small contribution from the lightweight roof.

In Type III structures, rafter loads are carried on the panels and are often applied with eccentricity in both directions. No doubt it was the appearance of this type of panel in the 1990s that raised concerns in the design profession. It is clear that this type of panel has higher axial loads, frequently at the ends of the panels. However, most engineers interviewed indicated that these rafter loads are typically less than individual panel weights and are a modest addition to very low gravity stresses. Similarly bending stresses introduced by the eccentricity of the connections are small compared to the bending stresses due to the fire stability or 'after fire' face loads on the panel.

d. Fire Design Requirements

Clearly fire is a significant risk in industrial buildings. The performance requirements of the New Zealand Building Code (NZBC:1992) require that:

1. occupants can safely escape
2. fire fighters can enter to fight the fire and rescue occupants
3. fire must be prevented from spreading to adjacent properties by either radiation or structural collapse.

To achieve these performance requirements, especially the latter, the most crucial requirement is that wall panels must be designed and detailed so that they are prevented from collapsing outwards. The draft Concrete Standard DZ 3101 includes a section 4 "Design for Fire Resistance". A specific clause 4.8 introduces requirements to ensure the connections between wall panels and steelwork will ensure this behaviour.

In Type I buildings, the normal design approach is to support the SPCW panels with fire rated cantilever columns and connections.

In Type II, III and IV buildings, roof structural steel is normally unprotected and generally has been disregarded with respect to external wall stability. The walls have traditionally been checked for strength as cantilevers under the fire stability loading of 0.5 kPa. This method has the virtue of great simplicity for the design engineer, but as SPCWs have grown taller and thinner, some research has been directed at investigating actual wall behaviour during a fire.

A paper by HERA [9] published in 1996 confirms that structural steel does continue to provide restraint to wall panels at elevated temperatures, and provides a methodology for checking this behaviour. However, this methodology is too complicated for everyday design of modest value buildings.

An unpublished BRANZ Design Guide [5] includes charts for maximum cantilever wall heights for walls of varying thickness and reinforcing content during fire. These charts are derived for the 0.5 kPa fire stability loading and allow for the P delta effect of the wall panel weight. They are an advance on the traditional approach but still disregard the restraint of the roof steelwork. The maximum single layer reinforcing content allowed without confinement reinforcement, which is impractical in SPCWs, is 1%. At this level of reinforcing the maximum H/t ratios correspond to 50 for 300mPa reinforcing and 60 for 500mPa reinforcing.

A 2003 SESOC paper by Buchanan and Lim[4] reports on a computer study of SPCWs in fire including restraint by roof steelwork. It includes recommendations for a maximum wall height and slenderness ratio, but subsequent correspondence indicates further research is required before these can be included in the Concrete Standard NZS 3101.

This Report was completed in December 2004. In May 2005 CCANZ published a draft Information Bulletin IB82, Slender Concrete Walls. This Bulletin develops a rational design method for checking the stability of SPCWs during an assumed standard fire. This method is reviewed in Appendix IV of this Report and is recommended to designers. The method addresses the basic Building Code requirement of preventing walls from collapsing outwards during fire and it complies with the relevant requirements of DZ3101.

e. Other Design Considerations

1. Cover and Tolerances. Although some wall panels could be reduced to 100mm, general opinion amongst the design engineers interviewed was that 120mm is a practical

minimum to satisfy cover requirements and tolerances. There is a particular aspect regarding tolerances that warrants closer attention by designers and contractors. This is the central positioning of vertical steel in Type II, III and IV panels designed to act as cantilevers or propped cantilevers. If the reinforcing is 10mm out of position in a 120mm panel, not unlikely under site conditions, the cantilever capacity will be reduced in one direction by approximately 20%.

2. Shrinkage and temperature movements provoke a range of opinion and concern. Given the normal construction pressure to lift panels as early as possible, it is prudent to address shrinkage and potential cracking in regard to detailing and construction sequence. Several comments were made about the bowing of slender panels under differential shrinkage and temperatures.
3. Lifting stresses. Practice varies as to whether this is addressed by the design engineer or contractor. Cracking during lifting is not uncommon, but generally is simply repaired and does not have serious consequences structurally or for weathering. One Auckland supplier recommended an H/t ratio of 67 as a practical maximum for lifting stresses.
4. Local cracking around the weld plates is not uncommon in Type I buildings due to welding or shrinkage, especially when several panels are effectively interconnected via weld plates through the portal legs. Many engineers avoid weld plates for this reason. Type II to IV buildings do not normally have this problem because the requirement for weld plates is more limited, there are no connections between panels, and stresses can be accommodated in the relatively short panels.
5. Acoustic and thermal insulation requirements can be a design requirement dictating panel thickness or the adoption of a Thermomass or similar proprietary system.

4.5 Observation of Construction: Typical Practice

All the Design Engineers interviewed in this survey observe construction in the traditional manner at critical operations such as concrete pours and panel lifting. Most Territorial Authorities require the Design Engineer to complete a PS4 Producer Statement certifying the building has been constructed according to the documents. Several TAs indicated they require a PS3 Producer Statement from the contractor.

SPCW buildings often benefit from close cooperation between designer and contractor during design and construction to ensure the design suits the contractor’s equipment and preferred construction methods. Well-established relationships are common involving close cooperation during both design and construction.

There is marked division of opinion about the adequacy of observation. The engineers interviewed were generally confident their level of observation was adequate to ensure a satisfactory standard of construction. Contractors and manufacturers on the other hand believe levels of observation by design engineers are inadequate on many contracts. Comments were made about unauthorised changes of drill-in anchors, broken off starter bars not being replaced and out of position fixings not being repaired adequately.

4.6 Construction Methods and Standards

Buildings with SPCWs, especially where site cast tilt up panels are employed, require careful planning and execution. General opinion is that there is a core of highly skilled contractors in all major cities experienced in this construction. There is, however, concern about a fringe element, especially in the overheated Auckland market, who are inexperienced and probably ignorant of some of the risks they are taking. The same contractors and suppliers who expressed concern about the level of engineer observation are concerned about these fringe operators and would welcome a Contractor Registration requirement.

4.7 Building Consents: The Regulatory Environment and TA Procedures

The Building Act of 1992 introduced a system whereby Territorial Authorities have been able to rely on Producer Statements from a Design Professional to support building consent applications, often together with a Design Review by an approved peer reviewer. Territorial Authorities have modified their procedures and organisations in various ways and have sometimes moved away from the detailed checking undertaken under the previous Building Permit regime. Thus the degree of checking of designs varies considerably throughout the country.

One large TA interviewed maintains a checking department of experienced engineers. The engineers exercise their judgement and spend more time on new and unfamiliar consultants' designs than on well-established firms' submissions.

Another smaller TA employs a company staffed largely by ex-employees and has no in-house engineering staff. Every tenth Building Consent Application is audited by this company, and it appears that no engineering judgement is involved in the selecting the Applications for Audit. It seems unlikely that this system will ensure adequate technical checks.

There is no doubt that this regulatory environment is more flexible, and this has benefits and costs. The benefits are that competent professionals have more freedom to exercise their judgement. The costs are that there are fewer checks and balances on the less competent that could result in unsafe buildings.

5. SPCW BUILDINGS: WHAT ARE THE SIGNIFICANT CONCERNS?

There is a range of concerns amongst Design Engineers and TA Engineers and a range of opinion as to the severity of those concerns. It is pertinent to note that many of the interviewees in Christchurch and Wellington were aware of the Type II, III and IV buildings, but had not had any personal experience of them. The following concerns were shared by a significant number of the interviewees.

a. Panel Slenderness and Potential Buckling

Despite the qualification above, there is a general concern about potential buckling in panels in Type II and III buildings with H/t ratios exceeding 50 to 70. This concern is based on uncertainty and the lack of guidance available rather than a conviction that these panels have a high probability of failing by buckling in a major earthquake. There is a general awareness that research has been undertaken by BRANZ and the Universities of Auckland and Canterbury, and a degree of frustration that test results are not yet available. Design engineers would like a simple guideline on safe H/t ratios for SPCWs.

Type I panels do not provoke concern about buckling because there is effective restraint at each portal leg and effective H/t ratios are lower generally.

Type IV building panels are limited by structural design constraints to approximately 6m, but the effective slenderness ratios are high because the buckling length is doubled. Many design engineers are concerned about the high deflections associated with these systems and the lack of redundancy.

b. Robust Connections

There is a general consensus that connections in SPCW buildings need to be robust for construction reasons and confidence that this will provide an extra margin of safety against “secondary” stresses, such as those induced by shrinkage.

c. Weldplates

Weldplates are widely used in Type I buildings because of their convenience for casting within the formwork. Concern raised included potential embrittlement when welds are detailed at bar bends, potential cracking due to the inhibition of shrinkage and temperature movements especially during welding. The historical concern about weld plates is addressing the problem of incorrect location.

d. Bent Out Bars and Starters

There is general consensus that bent out bars and protruding starters should be avoided if at all possible, because of the difficulty of controlling what will happen to them on site. Some designers avoid them and detail cast-in anchors, others use low yield bars and carefully specify regulation bend radii, others use drill-in anchors, which are very dependent on workmanship. Many Type II, III and IV building panels are detailed, and cast in precast factories, with long starter bars protruding to provide the cantilever base reinforcing. Typically these bars are bent over for transportation and rebent on site. This is a critical connection, and this practice is a significant concern. Because the outcome cannot be reasonably guaranteed, the detail should not be used in this critical situation.

e. Cast-in Fixings

Cast in fixings largely overcome the danger of site damage if they are designed and anchored correctly. They are generally satisfactory, if they can be positioned to reasonable tolerances. They are not favoured in Type II and III buildings by the industry in Auckland for fixing eaves beams, because of the difficulty of site alignment.

f. Drill-in Anchors

These are widely used because they are convenient for construction. They are the preferred method of fixing eaves tie beams to panels in Auckland. These fixings are required to act in tension under face loads and this is a significant concern because:

(i) There is a variety of products on the market of varying quality, something that is not necessarily appreciated on the work site.

(ii) All products are dependent on correct installation in frequently difficult site conditions.

(iii) Load capacities are generally based on static load tests, whereas in an earthquake loads are dynamic and usually in both directions.

(iv) Fixings relying on epoxy are a particular concern because they do not perform satisfactorily under fire conditions.

It is therefore difficult to be confident these fixings will prove reliable in an earthquake and therefore they cannot be recommended.

6. RESEARCH ON SPCWs

There has been considerable effort by industry and the research organisations to research SPCWs, although some results are unpublished.

6.1 Potential Buckling under In-Plane Loads

The concerns that arose in the mid-1990s about potential buckling in SPCWs during an earthquake, particularly as they evolved in Type II and III buildings, resulted in some early research at the University of Canterbury by McMenamin [6] and Chiewanichakorn [7] involving five and four test panels respectively. The five McMenamin test panels all had H/t ratios of 50 and H/L ratios between 1.25 and 2.5. Two panels were recorded as failing by buckling but the definition of buckling is arguable.

The four Chiewanichakorn test panels all had H/t ratios of 75 and H/L ratios of 3.75, so were the first reasonable simulation of the Type II and III building wall panels. Three of these four test panels did not buckle when tested to displacement ductility of 4. The fourth panel did buckle at displacement ductility of 2.5. All panels were heavily loaded and reinforced compared with typical panels identified in this review.

In 2000 it was agreed that BRANZ and the Universities of Canterbury and Auckland would collaborate on a further test programme of SPCW panels. BRANZ agreed to coordinate the programme, and consultative committees of design engineers and other industry participants were convened in Auckland and Christchurch to advise the researchers and ensure the test panels would be realistic simulations.

BRANZ [8] tested four panels with H/t ratios of 62.5 and H/L ratios of 4.17. Two panels had eccentric axial loads to simulate a Type III building. All four panels survived well to high displacement ductility; they displaced significantly out of plane but did not buckle or become unstable.

The University of Auckland tested 13 panels with H/t ratios from 27 to 78 and H/L ratios from 4.8 to 8.4. Axial load and reinforcing content was varied. Results have not been published but a review of the raw test data by BRANZ showed the walls generally remained stable up to displacement ductilities of 4.

The University of Canterbury [Sudano, 10] has tested two panels on the shake table. Both panels had H/t ratios of 60 and H/L ratios of 3.1. Both were subject to three earthquake displacement records of increasing amplitude. Failures were of a sudden brittle nature, and are discussed in more detail in the BRANZ Report [5] referred to below. Some doubts have been expressed about the severity of the earthquake being simulated in these tests.

BRANZ has drafted a “Design Guide for Slender Precast Concrete Panels” [5] based on this research, but it is yet to be published. This document includes a comprehensive summary of the test panel results.

The results of this research have been incorporated in the requirements of Chapter 11, DZ3101.

6.2 Behaviour during Fire

Section 4.4 (d) above refers to recent research into the behaviour of SPCWs in fire and includes:

(a) A paper by HERA[9] prepared for a seminar and published in 1996.

(b) A 2003 paper by Buchanan and Lim [4] on a computer simulation study of SPCWs in fire.

(c) An unpublished BRANZ Design Guide, Slender Precast Concrete Panels [5] which includes some charts relating wall heights to thickness and reinforcing concrete.

(d) A draft CCANZ Information Bulletin IB82, Slender Concrete Walls, [10].

7. STANDARDS DEVELOPMENT AND CCANZ INFORMATION IB82: SLENDER CONCRETE WALLS

The Concrete Standard NZS3101 has been amended and the new version, currently in circulation as DZ3101.1PB1, is likely to be published in late 2005.

There are two sections particularly relevant to SPCWs: Section 4.8, External Walls that could collapse outwards in fire, is referred to in section 4.4(d) above; Section 11.3.5, Dimensional limitation for stability, sets various criteria to ensure stability of SPCWs under concentrated gravity loads and in-plane seismic loads.

A draft CCANZ Information Bulletin IB82, Slender Concrete Walls [10] was circulated at Precast Concrete Seminars in May 2005. This bulletin develops a rational design method for checking the stability of SPCWs during a fire. It complies with the requirements of DZ3101 listed above. It is discussed in detail in Appendix IV of this Report.

8. CONCLUSIONS AND RECOMMENDATIONS

It should be recorded that these conclusions are drawn from a limited survey. Four types of buildings using SPCWs have been identified, but this report focuses on Types II and III because they represent the majority of the buildings and they employ the particular type of panels that has been causing concerns.

8.1 SPCW Buildings being Designed and Built

a. Type I to IV buildings are being built throughout the country, but there is a geographical bias.

b. Type I remains the most common type in Christchurch and the South Island.

c. Types II and III are the most common types in Auckland and probably, because of the concentration of population and industry, in New Zealand.

d. Type IV is not uncommon in Auckland and appears occasionally elsewhere. Seismic and wind loads are a limitation outside Auckland.

Tilt slab on-site casting is the preferred method outside Auckland, whereas within Auckland the preference is for off-site precasting.

8.2 Design of SPCW Panels in Type II, III and IV Buildings

There has been widespread concern and uncertainty in the design profession about the increasing slenderness of these types of panel and their likely behaviour during major earthquakes and during fire.

Research at BRANZ and the Universities of Auckland and Canterbury has indicated this type of panel will behave satisfactorily when loaded in plane during a major earthquake.

The draft Concrete Standard DZ3101 addresses the issues of fire, stability and buckling under in-plane seismic loads in regard to SPCWs as discussed in section 7 above.

A draft CCANZ Information Bulletin IB82 develops a design method for addressing these issues and is recommended. It is discussed in detail in Appendix IV of this Report.

8.3 Connections of SPCW Panels in Types II, III, and IV Buildings

The behaviour of these panels is dependent on the cantilever foundation and the eaves tie beam. Three details identified as being of concern follow.

a. The cantilever foundation which relies on a couple being developed between a tie bar into the floor slab and the friction/adhesion force on the underside of the foundation is shown in Figure 4. There are concerns about the ability of this detail to develop moments between the underside of the footing and the panel. It is recommended that this detail be investigated further.
b. The L footing cantilever detail is shown in Figures 5 and 6. This is often constructed as described in 5.d. and becomes in effect a ‘bent out bar’ detail. This is doubtful practice because of the difficulty of site control and the risk of these bars snapping off. Therefore it should not be used. Cast-in anchors with adequate anchorage and proven ductility are the recommended alternative.
c. Eaves tie beams are frequently connected to SPCW panels using drill-in anchors for construction convenience, as shown in Figure 7. For reasons outlined in 5.f., this detail is suspect and an alternative detail using cast in anchors such as that shown in Figure 8 should be used.

8.4 Construction Standards

The opinion of design engineers surveyed was that construction standards are generally satisfactory, but suppliers and contractors are concerned about less competent operators, especially during buoyant market conditions. All would welcome an effective form of Builder Registration to ensure basic competence and address concerns about site safety. The Building Act 2004 requires a form of registration for various “Licensed Building Practitioners” and this will address this concern when it is actioned.

In the meantime, the Department should recommend to TAs that Contractors building SPCW buildings be required to supply a PS3 Producer Statement certifying that the building has been built in accordance with the drawings and specification. This requirement should be included as a condition of the Building Consent.

8.5 Observation of Construction

All design engineers interviewed in this Survey observe the construction of their designs and believe the situation to be satisfactory. Contractors and suppliers, however, believe some poor construction practice occurs and that this would be reduced by Territorial Authorities requiring engineer observation of all SPCW building contracts and PS4 Producer Statements from the observing engineer. The Department should recommend to all TAs that engineer observation and PS4 Producer statements be included as conditions of a Building Consent for SPCW buildings.

8.6 Building Consents

The widespread acceptance by Territorial Authorities of Producer Statements and Design Reviews to support Building Consent Applications is satisfactory, providing the TA is assured that the person or company is acting within their level of competence. Many TAs have lists of individuals and companies approved to issue Producer Statements and Design Reviews to meet this objective. However, it is prudent for TAs to have an audit system in place, in particular to address new and unfamiliar applicants or designs. It is essential that a TA operates an audit system that utilises experienced engineering judgment in selecting which Consent Applications should be audited. TAs should be advised accordingly.

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7. 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.

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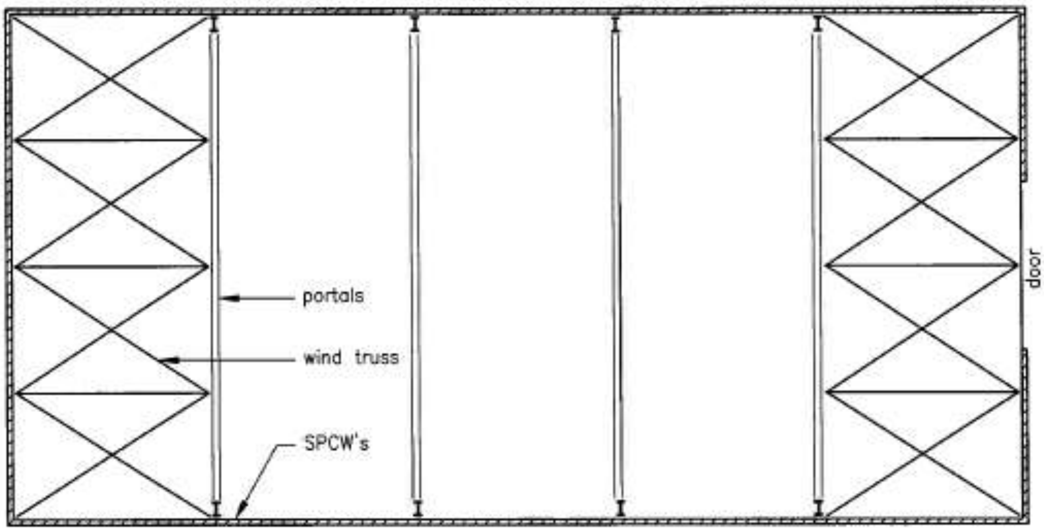
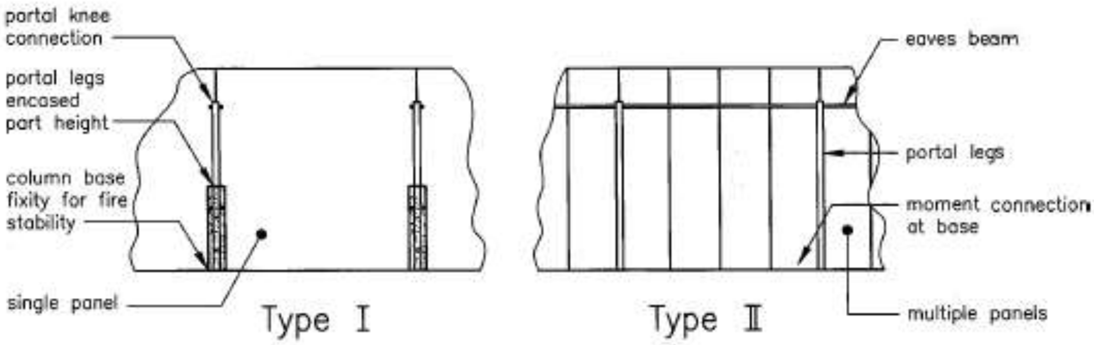
APPENDIX I

FIGURES



Plan of Type I & II



Single panel between portals fixed to the portal leg, floor slab connection optional. Portal leg concrete encased part height and fixed base for fire stability.

Multiple panels spanning between the foundation and the eaves beam (no panel connection to portal legs). Foundation connection moment resisting. Generally no physical connection between panels.

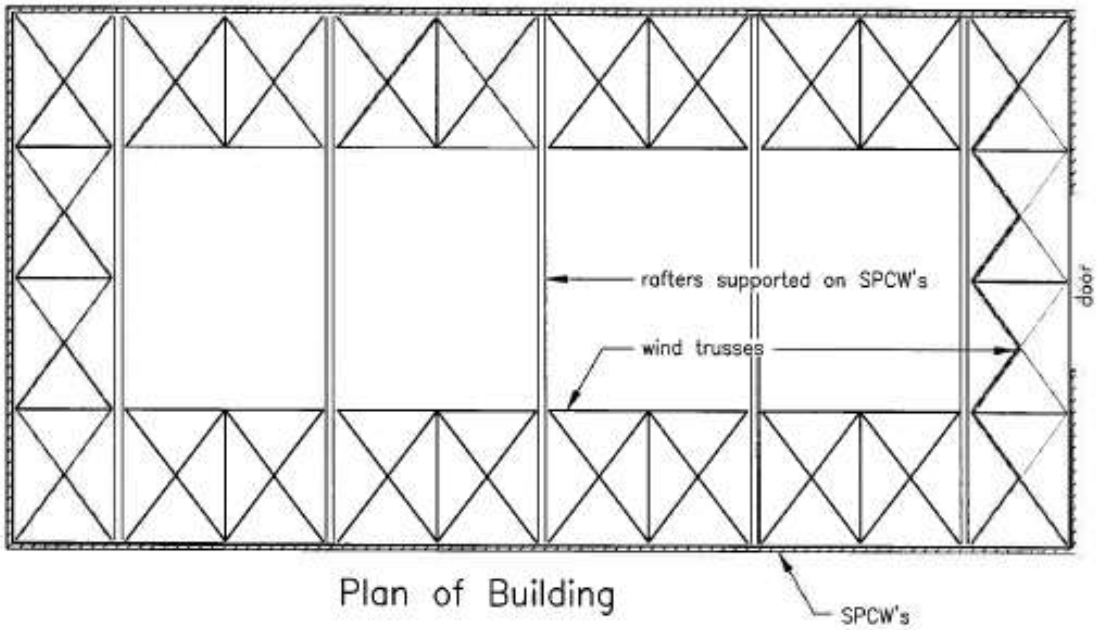
Elevations of Side Walls

Structural Systems

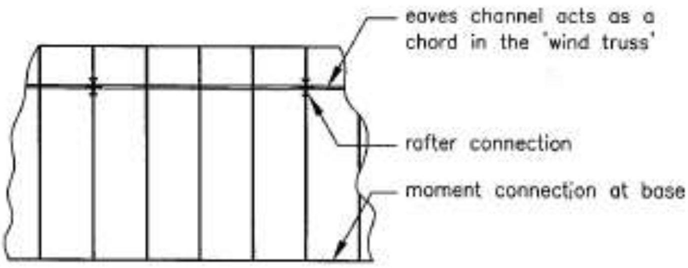
1. Transverse wind and seismic forces resisted by portal action, end wall panels resist local forces by in-plane shear/flexure.

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

Figure 1: Type I & II Buildings



Plan of Building

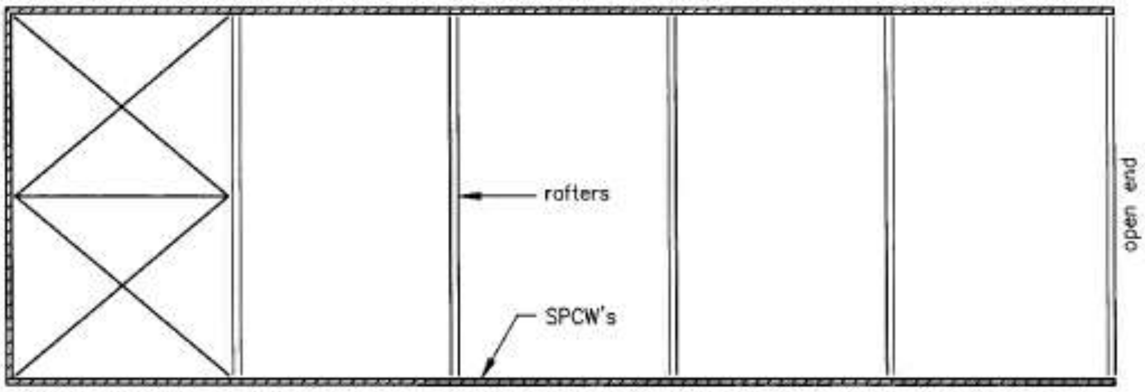


Side Wall Elevation

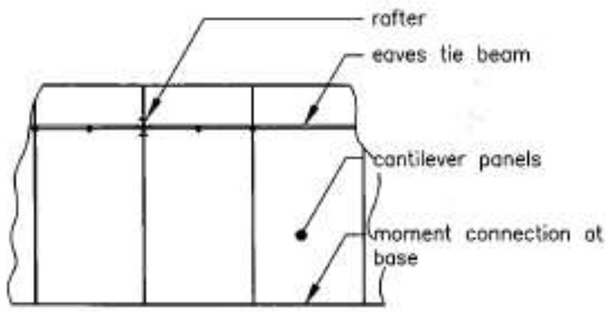
Structural Systems

1. Transverse wind and seismic forces on upper half of side wall panels and roof transferred by longitudinal 'wind truss' to the end wall panels which resist the forces by in-plane shear/flexure
2. Longitudinal wind and seismic forces from the roof and top half of the end walls transferred by transverse 'wind truss' to the side walls which resist the transferred forces by in-plane shear/flexure.
3. Roof gravity loads supported by side wall panels.

Figure 2: Type III Building



Plan of Type IV Building
with Cantilevered SPCW's



Elevation of Wall Panels

Structural Systems

1. Transverse wind and seismic forces resisted by cantilever action, left 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 wall transferred to the side walls via roof bracing. Side walls resist the transferred forces by in-plane shear/flexure.

Figure 3: Type IV Building

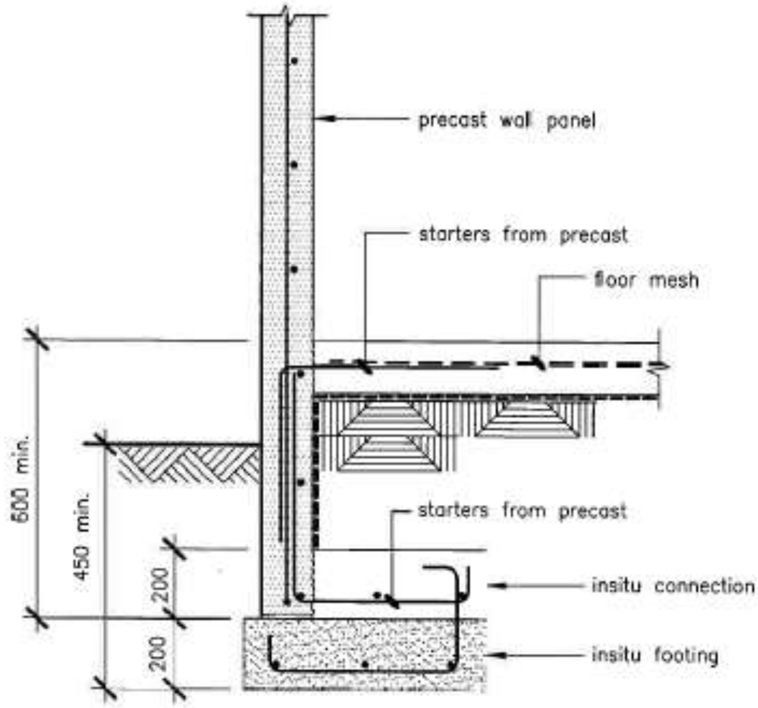


Figure 4

Cantilever Foundation –
Floor Tie / Foundation Friction

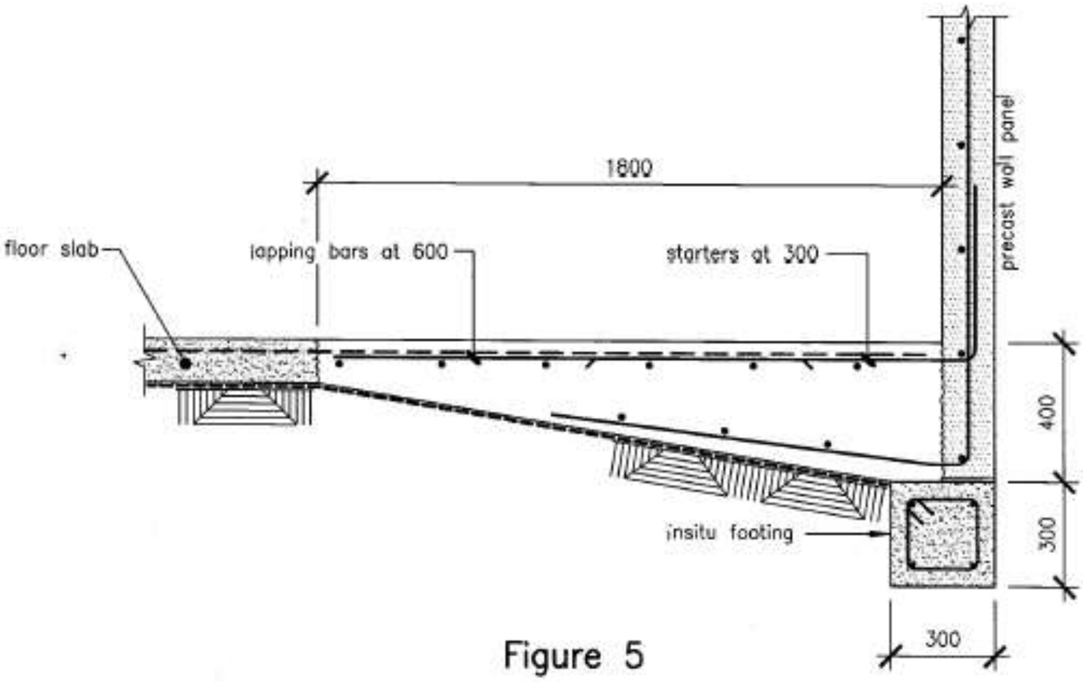


Figure 5

Cantilever Foundation –
Floor Thickening

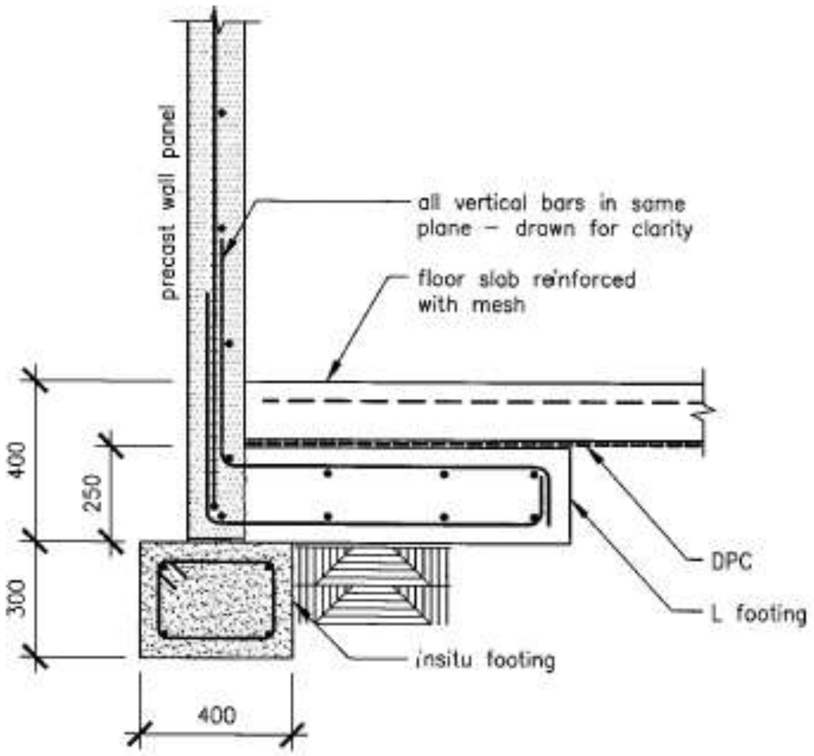


Figure 6

Cantilever Foundation –
L Footing Under Slab

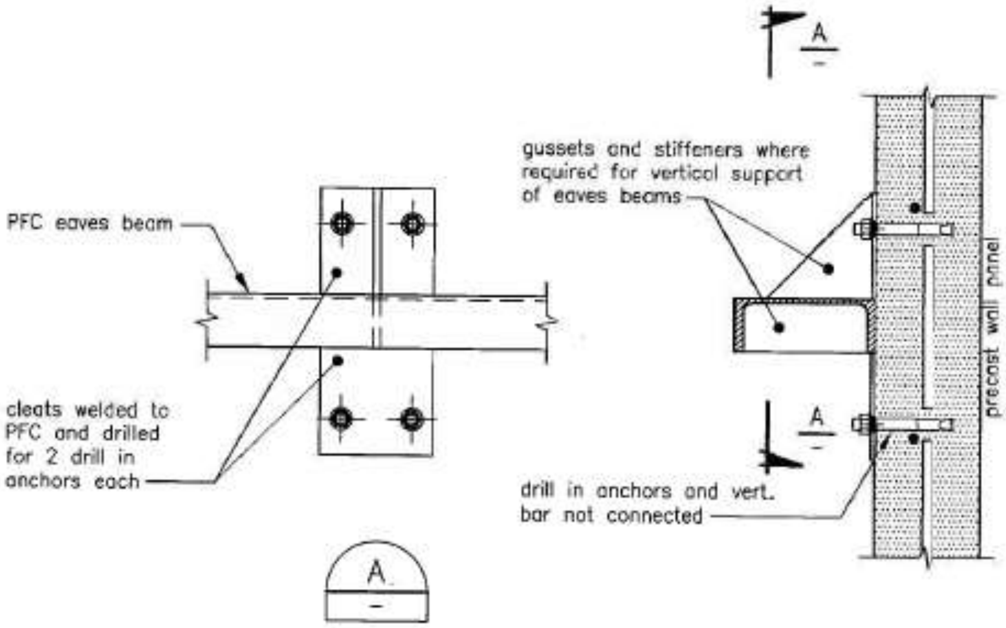


Figure 7
Eaves Beam Fixing
using Drill in Anchors
– Not Recommended

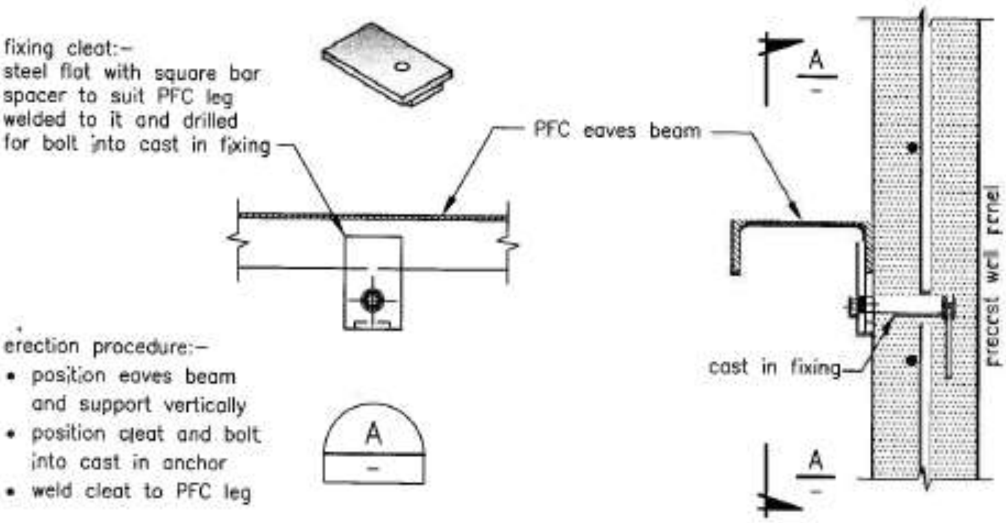


Figure 8
Eaves Beam Fixing
using Cast in Anchors
– Recommended

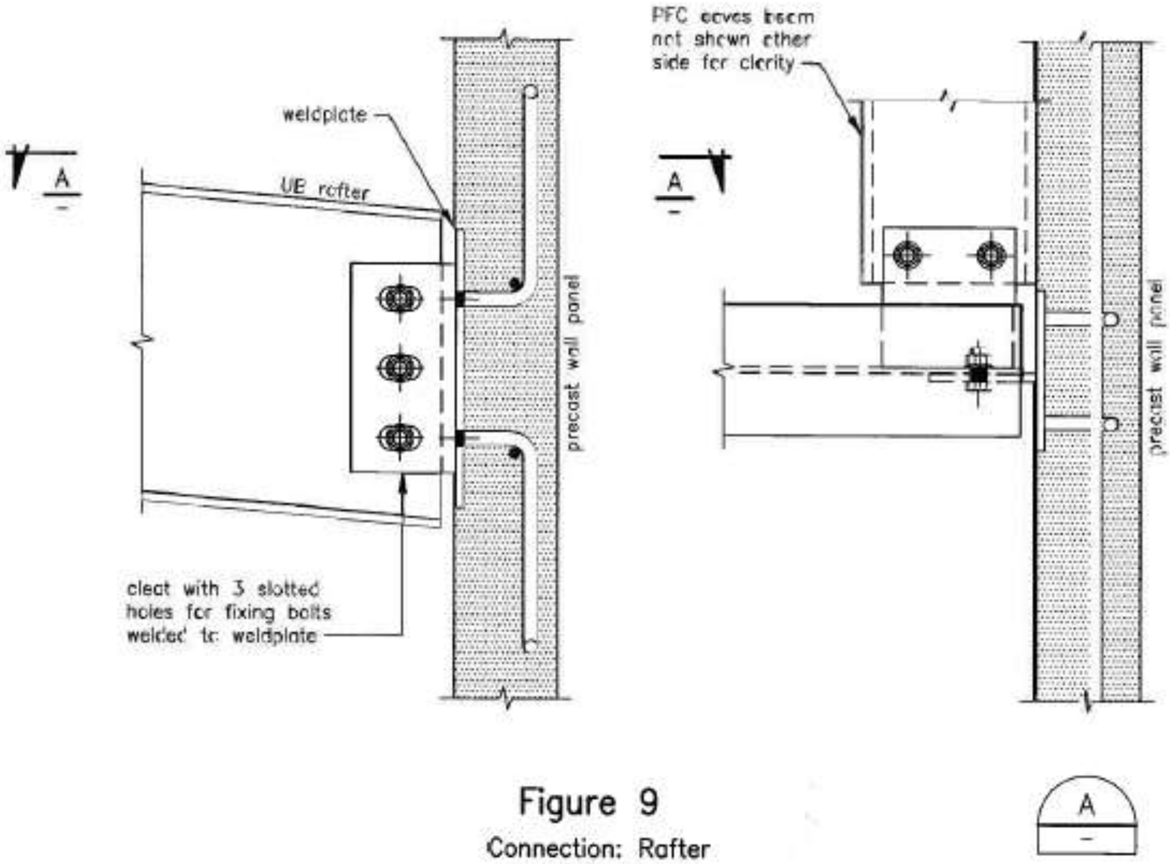


Figure 9
Connection: Rafter
to SPCW Panel

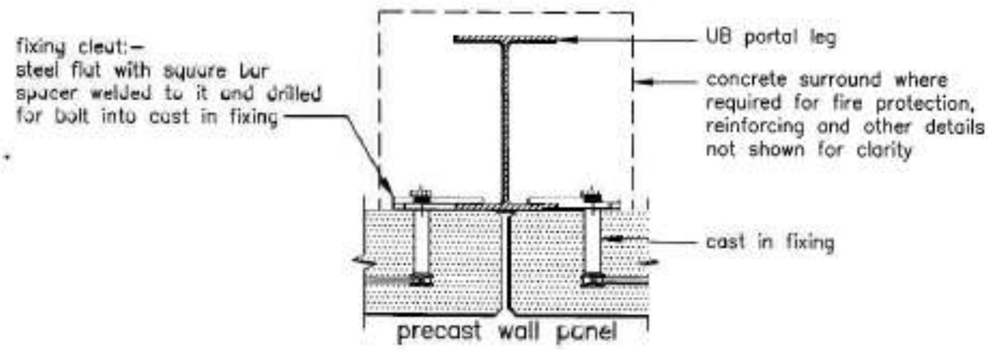


Figure 10
Type I Building: connection
panel to portal leg

APPENDIX II

BIA SURVEY – SLENDER PRECAST CONCRETE WALLS QUESTIONNAIRE

1. INTRODUCTION

Following the publication of John Scarry's Open Letter to IPENZ on the parlous state of the Structural Engineering Profession and Construction Industry, the BIA commissioned SKM to report on this letter and identify areas of serious concern. One of the technical topics identified as being of significant concern, as opposed to being of major concern, was Slender Precast Concrete Walls (SPCW) and their performance in a major earthquake. Generally these walls are tilt panels used in single-storey buildings although multi-storey precast panels and slender in situ wall panels were also mentioned.

Concern specifically about the slenderness of many of these panels within the industry and research community predates the Scarry Open Letter and has provoked several research projects.

This Survey is addressed at determining the following.

a. How are these buildings being designed, detailed and built?
b. Are there serious concerns about these building systems, components and details?
c. What should be done about these serious concerns?
d. Review recent research and recommend directions of future research?

The concerns relate mainly to the design of these panels and so the Survey will canvass the opinion of Structural Design Engineers, Territorial Authority Engineers and Research Engineers. However, it is also felt that the experience and opinions of Contractors and precast panel manufacturers will add a valuable practical dimension so they will also be approached.

Russell Poole, ex Director of Holmes Consulting Group, and resident in Hong Kong from 1992 until 2002, will conduct the Survey, initially during June and July 2004 and will be reporting to Dr David Hopkins at the BIA.

2. SURVEY METHODOLOGY

The Questionnaire below will be followed by an interview, generally face to face but in some cases by telephone. A response to the Questionnaire will be appreciated, especially if it is feasible to respond in advance of the interview.

Initial interviews in Christchurch on June 24th and 25th have provoked minor alterations to this Questionnaire including the deletion of references to in situ walls on the basis that very slender walls are most unlikely to be built in situ.

3. QUESTIONNAIRE

As noted above this Survey is principally aimed at the design profession, so if any of the questions below are not relevant to you, please ignore them.

a. Identify typical structural systems employing precast walls being used in your area?
b. What loading usually determines the design of the SPCWs?
c. What minimum thickness do you consider practical, and what aspect ratio limits do you follow?
d. Do you have concerns about the behaviour of any of these systems, their components, or connection details, in a major earthquake?
e. What typical details are in use for connecting:
   i) panel to floor
   ii) panel to portal
   iii) panel to panel?
f. Have you encountered any multi-storey wall panels or panels in multi-storey buildings that were of concern to you, and hence should be included in this Survey?
g. Do you, or your staff, have regular opportunities to observe construction of SPCWs, especially precast panels for single-storey buildings, and have those observations raised any particular concerns?
h. Are there other aspects of these panels that concern you, e.g. Fire resistance, weathertightness, connection robustness, reinforcing cover, tolerances, temperature movements, acoustic properties, thermal properties?
i. Priorities for research to improve design practices for SPCWs?

Russell Poole
28th June 2004

APPENDIX III

LIST OF SURVEY PARTICIPANTS

Designers:

Alan Reay	Managing Director, Alan Reay Consultants Limited, Christchurch
Dick Cusiel	Director, Lovell Smith and Cusiel Limited, Christchurch
Ray Patton	Director, Clendon, Burns and Park Limited, Wellington
Kevin Spring	Principal, OSA Silvester Clark Limited, Wellington
Stuart George	Director, Buller George Engineers Limited, Auckland
Bob McGuigan	Director, MSC Consulting Group Limited, Auckland
Stephen Rod	Director, Chris Howell and Associates Limited, Auckland
Warren Clarke	Director, Warren Clarke Consulting Engineers, Hamilton

Territorial Authority Engineers:

John M. Taylor	Senior Building Control Engineer, Christchurch City Council
Harry Adam	Manager, Land and Buildings, Civil Design Services Limited, Lower Hutt
Bill Vautier	Senior Structural Engineer, Auckland City Council
Khen Lee Tan	Senior Structural Engineer, GMD Ltd, acting for Manukau City Council

Research Engineers:

Dr Andy Buchanan	Professor of Civil Engineering, University of Canterbury
Dene Cook	Project Manager, Cement and Concrete Association, Christchurch
Graeme Beattie	Principal Engineer, BRANZ Limited, Judgeford, Wellington

Contractors and Suppliers:

Scott Watson	Director, Naylor Love Limited, Christchurch
Mike Phillips	Director, Ebert Construction Limited, Wellington
Rodger Bradford	Managing Director, Bradford Construction Limited, Ashburton
Alan Martin	Director, Ebert Construction Limited, Auckland
Derek Lawley	General Manager, Pryda Reid Limited, Auckland
Len McSaveney	Market Development Manager, Golden Bay Cement Limited, Auckland

Appendix IV

CCANZ Design Method for SPCWs

This design method has been developed by CCANZ and was first presented at a series of Seminars in May 2005. A preliminary edition of the Information Bulletin IB82: Slender Concrete Walls was distributed at the Seminars and copies are available from CCANZ. The method conforms with the requirements of DZ3101 in particular sections 4.7 and 4.8, Fire requirements for walls, and sections 11.3.5, Dimensional limitations for stability of walls.

The bulk of IB82 is devoted to the rational assessment of cantilever wall behaviour subject to an assumed, and probably conservative, standard fire. P△ effects are included. The method is extended to establish the reaction force in a propped cantilever. This represents the system in Type II and III SPCW buildings provided by the eaves beam.

The method then checks for wind and seismic face loads and checks in plane behaviour in accordance with DZ3101.

The hand calculations of this method are protracted but tables are provided that greatly simplify the fire loading calculations and CCANZ is preparing a computer programme for the method incorporating all the above checks.

IB82 does not establish maximum slenderness ratios so as a design starting point the unpublished BRANZ Design Guide values of 50 for Grade 300E and 60 for Grade 500E with 1% central reinforcing are suggested for cantilever walls. For propped cantilever walls corresponding values of 60 and 70 would be reasonable starting points. Note DZ3101 establishes a maximum slenderness ratio of 75. Further experience of these new design methods will allow refinement of these figures.