Determination 2016/003

Dispute about the issue of building consents and code compliance certificates and the building code compliance of building work for commercial buildings 2-5 at 2 Barry Hogan Place, Christchurch

Summary

This determination considers the authority's decision to issue building consents and code compliance certificates for commercial buildings with pre-cast concrete panels, and whether the building work complied with Clause B1 of the Building Code. The determination also discusses the difference between an alternative solution and industry practice.

1. The matter to be determined

1.1 This is a determination under Part 3 Subpart 1 of the Building Act 2004[1] ("the Act") made under due authorisation by me, John Gardiner, Manager Determinations and Assurance, Ministry of Business, Innovation and Employment ("the Ministry"), for and on behalf of the Chief Executive of the Ministry.

1.2 The parties to the determination are:

- Princess Lot 5, Body Corporate 398770 and Telegraph Hill Investments Ltd, who are the owners of the buildings 2 to 5 at 2 Barry Hogan Place, ("the applicants") acting through a lawyer ("the applicants' lawyer")
- Christchurch City Council ("the authority"), carrying out its duties as a territorial authority or building consent authority

1.3 I have also included Chartered Professional Engineers W. Lomax and G. Banks from the firm Structex Studio Ltd who undertook the building's structural design for the applicants ("the design engineers") as persons with an interest in this determination. The design engineers engaged a lawyer to act on their behalf ("the design engineers' lawyer").

1.4 This determination arises from the applicants' concerns about the compliance of their buildings with the Building Code (Schedule 1, Building Regulations 1992) when they were designed and built. The applicants stated the design engineers claimed the buildings were code compliant at the time of design but did not provide evidence to support this. The applicants subsequently commissioned a report from a consult engineering firm ("the consultant engineers") which identified what this firm considered to be critical structural weaknesses in the buildings' design.[2]

[1] The Building Act, Building Code, Verification Methods and Acceptable Solutions, past determinations and guidance documents issued by the Ministry are all available at www.building.govt.nz or by contacting the Ministry on ph: 0800 242 243.

[2] As described in the New Zealand Society for Earthquake Engineering (NZSEE) Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, June 2006

1.5 The applicants also queried whether the authority was correct to issue building consents and code compliance certificates for the building work.

1.6 The matters to be determined[3] are therefore:

- Matter One – whether the building work complied with the Building Code, in particular with Building Code Clause B1 Structure, at the time of design and construction, and
- Matter Two – whether the authority was correct to issue building consents and code compliance certificates for these buildings.

1.7 In making my decision, I have considered the submissions of the parties, the various reports provided by the parties and persons with an interest in these matters, the reports of the independent expert commissioned by the Ministry ("the expert") to advise on this dispute, and the other evidence in these matters.

1.8 I have not considered any other aspects of the Act or Building Code. The text of Building Code Clause B1, the versions of Verification Methods B1/VM1 and B1/VM4[4], and relevant extracts from standards referred to in this determination are included in Appendix A.

1.9 I understand contractual matters are at issue between the parties. I have ensured that all parties adhere to the requirements of natural justice throughout this determination and that they have been able to view all documentation provided to the Ministry. I note various correspondence has passed between the parties in relation to the 'final drawings' of the design consultant; however, I will not provide any further comment on these contractual matters.

2. The buildings

2.1 The buildings 2, 3, 4 and 5 ("the buildings") are part of the Workstation 55 office/warehouse complex built in 2007 on Lot 5, 2 Barry Hogan Place in the Christchurch suburb of Riccarton. The location of the buildings is shown in Figure 1 (note that building 1 is not covered by this determination).

Figure 1: the general layout of the buildings on Lot 5 (not to scale)

[3] Under sections 177(1)(a), 177(1)(b), 177(2)(a), 177(2)(d)

[4] A Verification Method for a Building Code clause provides a way to establish compliance with the requirements of that clause, via testing and/or calculations. B1/VM1 is a general method while B1/VM4 covers foundations.

2.2 The buildings are all two storeys high: Building 3 has two full levels while the others (buildings 2, 4 and 5) have partial first floors. The form of the building is what is known as modern light industrial construction, constructed predominantly from precast concrete and structural steel. The primary lateral load resisting and cladding elements are precast concrete wall panels of either 120mm or 150mm thickness. The panels are fixed by a combination of grouted vertical starters that extend from the foundations and horizontal reinforcing starters that extend from the base of the panel that are cast into the ground floor slab. The roof, that is comprised of structural steel beams, light gauge steel purlins and lightweight roof cladding, is supported by the wall panels.

The primary structure also contains structural steel frames that contribute to the support of the suspended concrete first floor.

2.3 Building 5 was prescribed as a sample building to be used in application of the evaluative framework, resulting in the report and peer review noted in paragraphs 7.1 and 7.2. Building 5 was described in this report as consisting of precast concrete panels and a structural steel roof and measuring around 70m long, 25m wide and 8.5m high. As noted in the previous paragraph, it has a partial first floor and this is constructed from reinforced concrete topping slab poured over precast hollow core concrete units. Building 5's external precast concrete panel walls at the perimeter have significant openings to provide entry access and windows.

2.4 I accept that this determination application has been lodged in respect of buildings 2-5 at 2 Barry Hogan Place. Table 1 below shows that in terms of comprising both double height and single height wall panels, building 5 is representative as a sample building, and is therefore used as such in this determination.

Table 1 Dimensions and constructed form of the buildings within the complex

Building Ref.	Dimensions (B×W)	Projected Areas Subdivided by Wall Panel Type			Other Features
		Double height (m^2)	Single height (m^2)	Total (m^2)	
2	63.3 x 24.8	533.2 (34%)	1036.6 (66%)	1569.8	Refer Structex plan S3.02
3	35.7 x 16.6	591.3 (100%)	0 (0%)	591.3	Refer Structex plan S3.03
4	48.2 x 24.8	679 (57%)	516.4 (43%)	1195.4	Refer Structex plan S3.04
5	69.8 x 24.8	454.6 (26%)	1276.4 (74%)	1731.0	Refer Structex plan S3.05

Explanatory Notes
(1) Single height is typically 7.2 m with no intermediate floor
(2) Double height is typically 2 @ 3.6 m with intermediation floor at half height
(3) Maximum height for all structures are deemed to be 7.2 m

3. Background

3.1 Buildings completed in 2007

3.1.1 The applicants engaged the design engineers to provide structural design and project management for these buildings. Two sets of structural drawings were supplied before building consent applications were submitted to the authority in mid-2006.

3.1.2 The authority issued the following building consents:

- Stage 1 – foundations and ground floor slabs on 28 August 2006

- Stage 2 – buildings 1, 2 and 3 on 20 October 2006, BC number 12066501
- Stage 3 – buildings 4 and 5 on 26 October 2006, BC number 13066501
- Stage 4 – siteworks on 28 August 2006, BC number 14066501

It is noted that no structural calculations were submitted in the consent applications.

3.1.3 The authority issued code compliance certificates for all buildings in December 2007.

3.2 Building assessment following the 2010/11 Canterbury earthquake sequence

3.2.1 Following the 2010/2011 Canterbury earthquake sequence, which caused significant damage to many Christchurch buildings, the applicants commissioned the consultant engineers to undertake a building assessment on the subject property. The consultant engineers stated there was no observed ground damage at the site and that the applicants' buildings had generally performed well in the recent earthquakes:

> … damage was limited to only minor cracking to some wall panels and some cracking to the wall panels at embedded bolts within the wall panels[5].

3.3 Further evaluation in 2012

3.3.1 In 2012 one of the applicants, Princess Lot 5[6], engaged the consultant engineers to prepare a detailed engineering evaluation ("DEE") report[7]. A copy of this report dated 15 October 2012 was provided to me by the applicants.

3.3.2 The report's purpose was to compare the buildings' earthquake resistance with current Building Code requirements for a new building constructed on the same site (expressed as a percentage of New Building Standard, %NBS,[8] and taking account of the May 2011 increase in Christchurch's seismic hazard factor from 0.22 to 0.3) and to identify any critical structural weaknesses that existed within the building.

3.3.3 In responding to this brief, the consultant engineers said they conducted a site visit which included invasive testing on one wall to determine the reinforcing mesh used. They used the 2006 sets of structural drawings supplied by Princess Lot 5[9] for their assessment but said that the original design calculations had not been made available.

3.3.4 The DEE report included a summary of seismic performance ratings for various structural elements within the building. The analysis identified components which exhibited %NBS ranging from 14% (for in-plane loads to concrete walls supporting first floor/ mezzanines) to 36% (for the first floor diaphragm[10]) based on an analysis prepared using a structural ductility factor [11] of 1.0 based on the use of the lesser ductility mesh reinforcement in the precast concrete wall panels. The DEE said that the calculated strength of the wall panels for out-of-plane bending loads is 22% NBS.

[5] As described in the later consultant engineers' report dated 15 October 2012
[6] An entity in the Latitude Group
[7] The Detailed Engineering Evaluation Procedure document was issued by the Structural Engineering Society and sets out methodologies for both initial and detailed quantitative building assessments
[8] For more about %NBS refer to the New Zealand Society for Earthquake Engineering (NZSEE) Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, June 2006 and corrigenda
[9] 'Lot 5, Princess St for Latitude Group Structural Drawings' by the design engineers, marked 'Consent' and dated 30 May 2006 and "Lot 5, Princess St for Latitude Group Precast Wall Panels' by the design engineers, marked 'Consent' and dated 30 May 2006
[10] Diaphragm is defined in NZS 4203:1992 as 'a horizontal or near horizontal system which acts to transmit lateral forces to the lateral force resisting elements.'
[11] 'Structural ductility factor' is defined in NZS 4203:1992 'as a numerical assessment of the ability of a structure to sustain cyclic inelastic displacements. Its value depends upon the structural form, the ductility of the material and structural damping characteristics.'

Using Building 3 as an indicator, it said the quantitative assessment of that building indicated a seismic capacity less than 34% of the current NBS indicating a high risk category building under the Canterbury Earthquake Recovery Authority (CERA) guidelines[12]. Further, the DEE report stated the roof cross bracing did not have adequate ductility, and concluded that:

> Critical structural weaknesses have been identified to the precast concrete walls and roof plane bracing system…

3.3.5 The design engineers were asked for comment on the consultant engineers' report (in correspondence dated 2 October 2012 and 25 October 2012). In response, the design engineers stated they were satisfied the structural design of the buildings complied with the Building Code applicable at the time of design, and noted that building consents and code compliance certificates had been issued for these.

3.3.6 I received an application for determination on 27 November 2012.

4. The submissions

4.1 The parties provided a number of submissions in response to this application for a determination, the experts' reports and the evaluative framework (both of which are described below). These responses are summarised in Appendices B and D.

4.2 In brief, regarding the application:

- The applicants expressed concerns about the extent of their buildings' compliance with Building Code Clause B1 Structure when these were designed and built; they said the design engineers had not provided evidence to support their claims of compliance; and also queried whether the authority had correctly issued building consents and code compliance certificates for the building work.
- The authority outlined its considerations for deciding to issue the building consents and code compliance certificates on reasonable grounds, including the supply of producer statements and its judgement at the time that the design engineers had suitable qualifications and experience.
- The design engineers said the drawings submitted for the building consent were preliminary drawings, not the final versions, and, because of a commercial dispute between themselves and the applicant, they had not supplied these final versions to the applicants.

4.3 I acknowledge here that there are contractual disputes between some of those involved but note that these matters are outside the scope of the determination.

5. The expert's reports

5.1 As mentioned in paragraph 1.7, I engaged an independent expert to assist me. This expert is from an international consulting firm and has structural engineering expertise. The expert was asked to review the design and construction of the building work and to provide a report detailing to what extent these met the performance requirements of the Building Code at that time; particularly with regard to Clause B1 Structure.

5.2 The expert visited the site on 21 February 2013, observed the condition of the buildings and conducted scans to determine the wall panel reinforcing. The expert's draft report was circulated to the parties and persons with an interest on 9 September

[12] CERA's updated earthquake-prone building timelines, identified in the consultant engineers' report as draft guidelines dated 26 October 2011

2013 to allow for their response prior to issuing their final report on 18 October 2013.

5.3 Following receipt of further information from the authority, I asked the expert to review this. I received a second report (relating to Stage 1 of the development at the site) on 30 January 2014 and circulated this on 4 February 2014.

5.4 The expert also provided some additional clarification in a letter to me dated 8 May 2014, responding to points raised by the design engineers (on 11 March 2014). I circulated this further information on 27 May 2014.

5.5 The expert's reports and responses to these are summarised in Appendix B and Appendix C respectively. These detail the main technical issues raised; but in general terms:

- The expert found that in considering the building design against the design standards cited in B1/VM1 and BI/VM4 at the time (the design engineers having identified these Verification Methods as their means of compliance) there were areas where the requirements of these standards were not met. He could not verify to what extent the building construction was Code compliant at the time as the buildings had been completed for some years.
- In response, the design engineers said that:
  - (a) the applicable standards and Verification Methods were not the only means of compliance;
  - (b) the expert's assumption that the drawings and specifications were the as-built construction was inappropriate;
  - (c) the buildings were Code compliant at the time they were built.
- The consultant engineers said the PS1 producer statement clearly outlined that the design had been prepared in accordance with B1/VM1 and B1/VM4 and the authority was not notified of any alternative solutions being used to establish code compliance.

6. The technical hearing

6.1 On 3 June 2014 I held a technical hearing in Christchurch to clarify various matters raised by my expert's reports and the responses to these. The hearing was attended by:

- myself, accompanied by a referee engaged by the Chief Executive under section 187(2) of the Act, and also two officers of the Ministry
- a member of the Body Corporate on behalf of the applicants, and the applicants' lawyer
- the consultant engineers engaged by the applicants (two representatives)
- the authority (one representative)
- the design engineers (two representatives) and their lawyer
- my expert and another member of his engineering firm.

6.2 A summary of some key issues discussed at this hearing is included in Appendix C.

6.3 As a way forward, and given that the design engineers acknowledged some aspects of their building design either fell outside of the methodology set out in the

Verification Method B1/VM1 (the stated means of compliance) or had a different interpretation, I proposed the Ministry establish an evaluative framework that could be used to test these buildings to establish compliance (or otherwise) with Building Code clause B1: in other words, to provide an alternative solution if this could be successfully followed. This framework would be informed by the issues discussed at the hearing, including the additional information provided by the design engineers regarding some of the technical basis for their design.

7. The evaluative framework

7.1 Following the technical hearing, the Ministry developed a suitable evaluative framework ("the evaluative framework") which I submitted to the design engineers on 1 August 2014 and copied to the parties. In summary:

- The evaluative framework recommended using building 5 as a representative building for this structural analysis.
- It recommended treating some potential brittle hard-drawn welded wire (HRC) reinforced elements as secondary elements as per NZS 3101:1995 Clause 4.4.13 within the structure, checking that they deformed with primary structure in a prescribed/predetermined manner which did not lead to a loss of structural integrity overall, and checking these out for potential loss of structural integrity.[13]
- It defined structural actions and distortions as per the Building Code:
    - o resisting input ground motions with nominally ductile (μ=1.25) response under tributary loads, and
    - o having identifiable zones or components where inelastic deformation[14] can occur in preference to the HRC reinforced main panel which remains essentially elastic, and
    - o being able to displace horizontally via these inelastic hinges (for example 50% above the μ=1.25 calculated displacement).
- The evaluative framework considered that information was still required to be provided about the assemblage of panels (identifying primary and secondary panels in terms of location and response characteristics), and identifying primary and secondary structures and the nature of push-over mechanisms to be used to verify compliance.
- It anticipated that this primary/secondary structure would be subject to a pushover (or similar) analysis that was consistent with recognised displacement-based design methodology.
- It acknowledged that the applicant had (at the technical hearing) proposed a basis of design that relied on the results of The University of Canterbury test report[15] of similar HRC reinforced wall panels.
- It agreed that the analysis was to be undertaken on the basis of knowledge across the structural engineering profession in 2006, but not including, the now current

[13] The loss of structural integrity is defined as the loss of load carrying ability, with failure leading to rupture or collapse when subject to prescribed distortions.
[14] Elastically responding structure is defined in NZS 4203: 1992 as 'a structure designed and detailed in accordance with this Standard and the appropriate material standard so that a structural ductility factor of 1 to 1.25 is appropriate in assessing the ultimate limit state seismic actions.'
[15] J.I Restrepo, F.J Crisafulli and R. Park, "Earthquake resistance of structures: the design and construction of tilt-up reinforced concrete buildings", University of Canterbury Research Report 96-11, September 1996

provisions of the earthquake actions standard NZS 1170/5:2004 and the draft version of the concrete structures standard NZS 3101:2006.

- Reporting results from this evaluation should clearly identify:
  - the extent to which primary structure elements could be identified and distinguished from secondary elements
  - the experimental (or other) basis on which the claim for the primary structure achieving B1 compliance was based, and
  - confirmation that any failure of elements within the secondary structure did not lead to loss of integrity.
- The %NBS values likely to be obtained for building 5 (being a representative sample of other buildings in the complex) should be evaluated in longitudinal and traverse directions based on the analytical procedures applicable to the building as it was likely to have been constructed in 2006.

7.2 The design engineers accordingly conducted an evaluation of building 5 which was peer reviewed by a firm of structural engineers (“the peer review engineers”). I received an initial summary of their findings on 16 February 2015 and the completed findings on 17 April 2015. These were sent to all parties and persons with an interest.

7.3 I note that while the design engineers provided the raw data and calculations, the findings were collated and a report was provided by the peer review engineers as a primary technical response on the matter for the applicant.

7.4 Responses to the evaluative framework, including this analysis, are discussed in section 10 and summarised in Appendices B and D.

8. The first draft determination

8.1 On 11 August 2015 I issued the first draft determination (“first draft”) to the parties. The first draft concluded that the building work did comply with Clause B1 via an alternative solution; however the out-of-plane face loads needed further justification and the roof bracing was clearly non-compliant. Whilst it concluded that the authority had incorrectly exercised its powers in issuing the building consents and code compliance certificates, the first draft considered there was no reason to reverse the issue of the building consents, it did, however, reverse the code compliance certificates.

8.2 On 17 August 2015 the authority declined to accept the first draft. However it did accept the technical findings as they had not carried out any separate structural analysis themselves. The authority provided information that in 2012-2013 the owners had made applications for a series of building consents to structurally strengthen the building. They concluded that, in their view, the overturning of the original the code compliance certificates to be inappropriate in light of subsequent works.

8.3 On 26 August 2015 the peer review engineers responded to the first draft providing the additional justification sought in relation to the panel out of plane face loads and panel slenderness and the implications of the comments regarding the PS1 and the means of compliance. The comments of this report have been incorporated into Appendix B.

8.4 On 8 September 2015 the applicant's lawyer responded not accepting the first draft and providing a report from the consultant engineers. The main comments of this report have been incorporated into Appendix B.

9. Discussion: general considerations

9.1 As stated in paragraph 1.6, there are two matters to be determined which I will consider in turn:

- Matter One – whether the building work complied with the Building Code, in particular with Building Code Clause B1 Structure, at the time of design and construction, and
- Matter Two – whether the authority was correct to issue building consents and code compliance certificates for these buildings.

9.2 In order for me to form a view on these matters I must take into account, among my other considerations:

- the stated means of compliance on the building consent documentation submitted to the authority and the authority's subsequent reliance on this documentation, and
- the Building Code requirements in force at the time, as well as what the available means of compliance were at the time.

9.3 The relevant Verification Methods and some of the standards applicable at the time the buildings were designed and constructed are noted in Appendix A. The relevant loading standard was NZS 4203:1992 which identified two limit states: serviceability limit state[16] and ultimate limit state[17]. The relevant concrete structures standard in relation to Verification Method B1/VM1 was NZS 3101:1995 and steel structures standard was NZS 3404: 1997.

9.4 I note that although NZS 3101: 2006 was published by Standards New Zealand in March 2006 and NZS 1170.5 in December 2004 these standards were not cited in compliance documents until September 2010; that is after the applicants' buildings were designed and built. I also note that the technical publication "Seismic design aspects for tilt-up buildings" published as a paper in the journal of the Structural Engineering Society ("the SESOC guidance")[18] has been relied on by the design engineers and peer review engineers on this project, while the report by R. A. Poole, commissioned by the Department of Building and Housing[19] and relating to slender precast concrete walls,[20] ("the Poole report") published after the design of the subject building had been completed has also been drawn on as a justification.

9.5 The peer review engineers noted it was appropriate and best practice for design engineers to utilise current technical publication guidance in relation to a design that has generally been completed using the cited design standards (refer Appendix D). I accept the relevant standards at the time of the design had some limitations and that industry practices together with technical publications then available were relied upon by the design engineers.

[16] Serviceability limit state is defined in NZS 4203: 1992 as the condition 'reached when the building becomes unfit for its intended use through deformation, vibratory response, degradation or other physical aspects.'
[17] Ultimate limit state is defined in NZS 4203: 1992 as the condition 'reached when the building ruptures, becomes unstable or loses equilibrium.
[18] J.I Restrepo, F.J Crisafulli and R. Park "Seismic design aspects for tilt-up buildings", Structural Engineering Society, December 1996, SESOC Journal 2(9) pp 9-24.
[19] The predecessor organisation to the Ministry of Business, Innovation & Employment
[20] R.A. Poole "Report to Department of Building and Housing: Review of Design and Construction of Slender Precast Concrete Walls", August 2005.

9.6 Roadmap to Appendices B, C and D

9.6.1 Appendix B: Summaries of written submissions

- The application
- The experts' reports
- The evaluation framework

9.6.2 Appendix C: Expert's reports and technical hearing

- Experts' reports and responses
- Technical hearing – key discussion points

9.6.3 Appendix D: Evaluation framework (see also paragraph 10.5)

- Precast panels – ductility
- Precast panels – in-plane and out-of-plane face loads
- Precast panels – slenderness
- Precast panels – reinforcement
- Roof bracing

10. Whether the building work complied with the Building Code (Matter One)

10.1 In considering whether the building work complied with the Building Code, in particular with Clause B1 Structure, at the time of design and construction, I concluded from the evidence supplied to me and my expert's reports that the original building design did not in fact completely follow the Verification Methods stated on the producer statement supplied to the authority as the means of compliance. I note that the basis of the design adopted relies on a mechanism of energy dissipation, via the reinforcing starter bars connecting the wall panels and the foundations, that achieves a structural ductility factor of not less than 3.

10.2 Accordingly, following the technical hearing I supplied an evaluation framework to the design engineers that I believed might be used by them as the basis for an alternative solution: i.e. a means of demonstrating compliance with this Code clause by conducting a detailed structural analysis of the buildings that relied in part on the SESOC guidance referred to earlier in paragraph 9.4.

10.3 I acknowledge the applicants' view[21] that the evaluative framework does not provide a suitable approach to determining all the issues raised in their application. Their argument is that the design as actually prepared was understood to have been undertaken in accordance with B1/VM1 as the designer's PS1 has stated, and it was therefore inappropriate to check the buildings retrospectively by an alternative solution method. However, one of the matters I need to determine is whether or not these buildings complied with the Building Code at the time of design and construction, and the decision that I need to make in this regard is not limited by the narrow nature of the design engineer's PS1 attestation. Therefore despite the wording of the applicant's matters for consideration (refer Appendix B) I am limited to determining matters under section 177 of the Act and to do that I have had to utilise a method of assessing building code compliance retrospectively in this case.

[21] Letter from the applicant's lawyer 3 October 2014: refer to Appendix B for more detail.

10.4 My decision on whether or not the building work was compliant at the time of design and construction is based on the results of the design engineers' evaluation, noting that their calculations have been peer reviewed (refer Appendix D), and taking further expert advice and evidence into consideration.

10.5 Recognising that detailed technical issues are involved, I have ordered the discussion of these results under the following key headings (and ordered the summaries of submissions in Appendix D similarly for clarity):

- Issue A: The precast concrete wall panels
    - ductility
    - in-plane and out-of-plane face loads
    - slenderness
    - reinforcement
- Issue B: Roof bracing.

10.6 Issue A: The precast concrete wall panels

Ductility

10.6.1 The panel design has an acceptable ductile mechanism that does not rely on ductile behaviour from the precast panels themselves, or ductile behaviour from the HRC mesh. I consider the design standard at the time (NZS 3101:1995) in some circumstances allowed for lesser ductility HRC mesh to be used.

10.6.2 In reference to Amendment 3 of NZS 3101:1995 (refer Appendix A) the use of non-ductile HRC mesh is not prohibited, subject to yielding of reinforcement not occurring at ultimate limit state.

The expert referred to the limitations within Amendment 3 of NZS 3101:1995 which recognised the limitation of welded wire mesh fabric, e.g. HRC 663 mesh reinforcement, in terms of its lesser ductility[22] characteristic and imposed restrictions on its use. That amendment stated that its use would only be permitted where:

- welded wire mesh fabric had a uniform elongation of at least 10%; or
- lesser ductile welded wire mesh fabric regularly be used when:
    - yielding of reinforcement will not occur at the ultimate limit state; or
    - the consequence of yielding or rupture does not affect the (structural) integrity of the structure.

Ultimate limit state at the time was based on NZS 4203:1992 requirements, which correspond to seismic inputs of μ=1.25. Based on the evidence presented, I do not consider the HRC mesh will yield under in-plane loads at ultimate limit state under design conditions as defined by the loading standard NZS 4203:1992.

10.6.3 As noted by the peer review engineers, the application of the term 'ductility' has evolved since these buildings were designed and constructed. Within the design framework prescribed in NZS 4203:1992/ NZS 3101:1995, reinforcing detailing and earthquake response behaviour was assumed to μ=1.25, although in restricted circumstances. However, within the currently applicable design framework

[22] Ductility is defined under NZS 4203: 1992 as 'the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

prescribed by NZS 1170.5/NZS3101:2006, the corresponding earthquake response behaviour required demand to be calculated based on μ=1.0 with μ=1.25 being allowed for detailing which permitted nominally ductile behaviour. Referring to the use of lesser ductile HRC663 mesh in wall panels of this type, the peer review engineers quoted from the SESOC guidance in this regard:

> By using capacity design to ensure that walls away from the connection do not crack under any type of loading conditions, the use of brittle electro-welded mesh to reinforce precast concrete panels does not have to be prohibited nor discouraged.

In-plane and out-of-plane face loads

10.6.4 The forces and displacements induced in the precast concrete panels were analysed using standards of the day and found (with some exceptions) to comply.

10.6.5 The peer review engineers concluded that panels subject to in-plane seismic action do satisfy capacity design over strength requirements for flexure (except for panels numbered 196-203) where available capacity exceeded μ=1.25 and approached μ=1.0 limits. The peer review engineers found similar results for shear capacity. I consider the panel in-plane strength satisfies the SESOC guidance for an adequate seismic system based on analysis using μ=3.

10.6.6 In relation to analysis of out-of-plane seismic action the peer review engineers concluded the panels were compliant where nominally ductile (μ=1.25) face load accelerations were applied to panel along with p-delta effects. However, as noted at paragraph 10.7 the top reactions applied by the panels at the roof diaphragm level exceeded the capacity of that diaphragm.

10.6.7 I consider the design engineers need to provide further information in relation to the analysis used for the out-of-plane face loads in relation to the statement that NZS 4203: 1992 Clause 4.12 relating to parts[23] is 'overly conservative' as the in-"structure amplifications predicted" by that calculation method will not occur. There needs to be further information to justify that statement in relation to amplifications for the out-of-plane analysis that is clearly prescribed by the NZS 4203:1992 standard.

10.6.8 The design engineers also need to provide further evidence that full height panels that are not supported by a first floor diaphragm comply with out-of-plane face loads determined in accordance with NZS 4203: 1992 Clause 4.12. This is particularly relevant for panels with significant openings.

Slenderness

10.6.9 As noted above in paragraph 10.6.1, as the panels have met the requirements of the relevant design standard with regards to structural ductility under in-plane shear loads, I consider the increased slenderness limit under Clause 12.3.2.2 of NZS 3101:1995 is able to be justified.

10.6.10 In addition I note the supporting technical information in the Poole report that allows for a relaxation of the standard where analysis and test results show adequate strength and stability at the ultimate limit state, although the "rationale analysis" referred to in the standard has not been demonstrated by the design engineer. I consider this still to be a matter requiring confirmation in terms of compliance.

[23] A 'part' is in reference to an element which is not intended to participate in the overall resistance of the structure to lateral displacement under earthquake conditions in the direction being considered (as defined under NZS 4203: 1992).

Reinforcement

10.6.11 The peer review engineers found that by utilising the HRC mesh the panels (with some exceptions) complied with the minimum reinforcing requirements and said the panels were not expected to undergo ductile deformation. I accept this finding.

Conclusion

10.6.12 In addition to the above analysis, and based on my own considerations, I consider it likely that not all the panels in the buildings have the same profile and it is clear that some of the face load panels do not comply with Clause B1. I consider it is reasonable to extrapolate these findings in relation to isolated panels to the building as a whole, although I do not have any information on how many of these panels actually exist.

10.7 Issue B: Roof bracing

10.7.1 After reviewing the evidence made available to me I consider that the roof bracing, including the restraint of panels at the roof level, does not comply with the relevant requirements of the Building Code. The calculations from the design engineers conclude these are understrength at around 34% NBS, while the peer review engineers and consultant engineers have both noted the roof bracing is inadequate.

10.7.2 I agree with the statements from the peer review engineers regarding the insufficiency of the roof bracing and consider their concerns (summarised below) need to be addressed by the applicants:

- concerns about the stud anchors in the anchorage panels
- reliance on bolts bearing onto the thin ply of the purlin steel to transmit axial load into the purlins
- the fact that buckling restraint of the purlins is possibly insufficient to stop purlin pairs buckling together in parallel, and
- the inadequate load path from panels into the roof bracing lines, as shown on the drawings.

10.8 Conclusions

10.8.1 In summary, in relation to Matter One I consider:

- The buildings’ original design is not in accordance with B1/VM1 and uses evidence from technical publications to support aspects of the design which must be considered as an alternative solution.
- The alternative solution offered as part of the evaluative framework appears to satisfy the requirement of Building Code clause B1.
- The structural ductility and in-plane face loads and reinforcement of the wall panels is compliant with Building Code Clause B1, although the compliance of the panels in terms of stability and slenderness requires further consideration.
- The out-of-plane face loads require further justification to determine compliance with Clause B1, in particular for full height panels that are not supported by a first floor diaphragm.
- The roof bracing, including the restraint of panels at the roof level, does not comply with Clause B1.

11. The building consents and code compliance certificates (Matter Two)

11.1 The building consents

11.1.1 The building consent documents lodged with the authority for the applicants' buildings were based on a demonstrated compliance with Code Clause B1 Structure via Verification Methods B1/VM1 and B1/VM4.This verification was via a PS1 producer statements completed and signed by a professional engineer from the design engineering firm.[24]

11.1.2 As discussed in paragraph 10.8.1, I consider that the design engineers provided an alternative solution for compliance with Clause B1 rather than following B1/VM1 as stated. I therefore consider that the authority was misled by the design engineers in this regard. I acknowledge that industry practice often precedes changes to the relevant Acceptable Solution or Verification Method. However, in this case I consider there is enough deviation from the B1/VM1 that the designer's needed to highlight this on the Producer Statement and provide details as to what the deviations were.

11.1.3 Further, I consider that if the design engineers had correctly signalled they were pursuing an alternative solution as a means of compliance to the authority at building consent stage, the authority could then have decided what action it might have required as per its standard procedures; for example, a request for structural calculations or peer review of the building design might have followed.

11.1.4 In my view, the plans and specifications submitted to the authority as part of the building consent applications did not provide sufficient information to establish compliance with B1/VM1. Under section 49(1) of the Act the authority must grant a building consent if it is satisfied on reasonable grounds that the provisions of the Building Code would be met if the building work were properly completed in accordance with the plans and specifications that accompanied the application.

11.1.5 An authority is entitled to accept a producer statement at its discretion in the belief that the author of the producer statement is credible. However, the receipt of a producer statement does not lessen the authority's liability in establishing code compliance.[25]

11.1.6 In relation to producer statements, a recent High Court case has made the following comments which I consider of assistance in this matter:

> [115] It would not be appropriate for a territorial authority to accept any producer statement without question. The extent to which a particular producer statement should be relied on in considering whether code requirements had been met would depend on all relevant circumstances. These would include, for example, the skill, experience and reputation of the person providing the statement, the independence of the person in relation to the works, whether the person was a member of an independent professional body and subject to disciplinary sanction, the level of scrutiny undertaken and the basis for the opinion. The territorial authority would also need to consider any other information relevant to whether the works had been carried out to an appropriate standard and could be expected to meet code requirements. This would include the skill, experience and reputation of the party

[24] I note there were two relevant PS1 forms, one for Buildings 1-3 (Stage 2) and the other for Buildings 4 and 5 (Stage 3) both dated 30 May 2006.

[25] Determination 2010/096 Refusal to issue a code compliance certificate for fire repairs to a house *(Department of Building and Housing)* 18 October 2010 and Determination 2013/053 Regarding the refusal to issue a code compliance certificate due to the lack of a producer statement for drainage work to a house *(Ministry of Business, Innovation & Employment)* 17 September 2013.

> carrying out the works, the complexity of the works, the likely consequences of non-compliance and whether any concerns had arisen regarding the quality of the works. Ultimately, the territorial authority was only entitled to issue a code compliance certificate if it was satisfied on reasonable grounds that the building works complied.[26]

11.1.7 In this case I consider it reasonable for the Authority to have relied on the PS1 provided with the design. The author was a known Chartered Professional Engineer in the Canterbury area. However, as noted by the expert (refer Appendix C) there were no structural engineering calculations submitted as part of the plans and specifications. The authority was unable to check the accuracy or completeness of the structural design as stated on the PS1 (that is in accordance with B1/VM1 and VM4). In particular, in relation to the wall panels and the roof plane bracing, the full construction details were not available at the time the building consents were sought and issued, and that fact should have been apparent to the Authority at the time the consent was issued.

11.1.8 Therefore, despite being misinformed by the design engineers and relying on the PS1, I consider the authority incorrectly issued building consents for the applicants' buildings on the basis of insufficient information being provided with the building consent application.

11.2 The code compliance certificates

11.2.1 Under section 94 of the Act the authority must issue a code compliance certificate if it is satisfied on reasonable grounds that the building work complied with the building consent. The authority issued the code compliance certificates for these buildings in reliance on a PS4 (construction) producer statement and said they had ensured relevant inspections were carried out and records were completed.

11.2.2 As I consider that the authority incorrectly issued the building consents due to insufficient information, it is a necessary corollary that I consider the authority incorrectly issued the code compliance certificates. Although I acknowledge that in some circumstances it is reasonable to rely in a PS4, this reliance needs to be in-conjunction with the plans and specifications provided. As the building consent was incorrectly issued and compliance with the Building Code was not able to be established, based on the plans and specifications originally provided, it follows that the authority did not have reasonable grounds that the building work complied with the building consents.

11.3 Other matters

11.3.1 Given that the building consents as issued were based on B1/VM1 compliance, I note that the design engineer retained the option to verify the basis of consent via the alternative solution route at all times up until the code compliance certificates were applied for and/or issued. In this case, it is clear that the design engineer offered what he described as a B1/VM1 compliant design when in fact that was not the case. Error or not, there is, in my mind, no question as to whether the building consent as issued was valid. The works specified could (the design engineer might argue) be justified in terms of Building Code compliance even if evidence of that was to be provided retrospectively. However, there is an obligation on the design engineer to make an application for a building consent amendment where a major variation occurs, and this, in my view, would in this case cover those aspects which did not satisfy B1/VM1 requirements.

[26] Body Corporate 326421 & Others v Auckland Council [2015] NZHC 862

11.3.2 In my view, there was a clear obligation on the design engineer to correct this aspect of his design in a manner that the authority can then issue the code compliance certificates with confidence that the works described in the PS1 complied with the Building Code.

12. The decision

12.1 In accordance with section 188 of the Building Act 2004, I hereby determine that with respect to whether the building work complied with the Building Code, in particular with Clause B1 Structure, at the time of design and construction, and taking into account the conclusions reached at paragraph 10.8:

- The as-built construction partially complies with Clause B1 of the Building Code via an alternative solution route, with the exceptions to compliance being noted below.
- The out-of-plane face loads (refer paragraph 10.8.1) for some panels require further analysis to justify compliance. This particularly applies to full-height panels that are not supported by the first floor.
- The roof bracing (refer paragraph 10.7) does not comply and needs to be addressed.

12.2 With respect to whether the authority was correct to issue building consents and code compliance certificates for the applicants' buildings, I determine that the authority incorrectly exercised its powers of decision in this regard. In relation to the building consents issued, I consider that notwithstanding some building work that is not compliant with the Building Code, there is no justification for the building consents as issued to be reversed.

12.3 In addition, I consider the information provided by the authority relating to strengthening work on the buildings is such that it would be inappropriate to overturn the code compliance certificates.

Signed for and on behalf of the Chief Executive of the Ministry of Business, Innovation and Employment on 4 February 2016.

John Gardiner
Manager Determinations and Assurance

Appendix A: Building Code Clause B1 and relevant Verification Methods and standards

A.1 Clause B1 – Structure

B1.1 The objective of this provision is to:
- (a) safeguard people from injury caused by structural failure,
- (b) safeguard people from loss of amenity caused by structural behaviour, and
- (c) protect other property from physical damage caused by structural failure.

Functional requirement
B1.2 Buildings, building elements and sitework shall withstand the combination of loads that they are likely to experience during construction or alteration and throughout their lives.

Performance
B1.3.1 Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives.
B1.3.2 Buildings, building elements and sitework shall have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during construction or alteration when the building is in use.
B1.3.3 Account shall be taken of all physical conditions likely to affect the stability of buildings, building elements and sitework, including:
(a) self-weight,
(b) imposed gravity loads arising from use,
(c) temperature,
(d) earth pressure,
(e) water and other liquids,
(f) earthquake,
(g) snow,
(h) wind,
(i) fire,
(j) impact,
(k) explosion,
(l) reversing or fluctuating effects,
(m) differential movement,
(n) vegetation,
(o) adverse effects due to insufficient separation from other buildings,
(p) influence of equipment, services, non-structural elements and contents,
(q) time dependent effects including creep and shrinkage, and
(r) removal of support.
B1.3.4 Due allowance shall be made for:
(a) the consequences of failure,
(b) the intended use of the building,
(c) effects of uncertainties resulting from construction activities, or the sequence in which construction activities occur,
(d) variation in the properties of materials and the characteristics of the site, and
(e) accuracy limitations inherent in the methods used to predict the stability of buildings.
B1.3.5 The demolition of buildings shall be carried out in a way that avoids the likelihood of premature collapse.
B1.3.6 Sitework, where necessary, shall be carried out to:
(a) provide stability for construction on the site, and
(b) avoid the likelihood of damage to other property.
B1.3.7 Any sitework and associated supports shall take account of the effects of:
(a) changes in ground water level,
(b) water, weather and vegetation, and
(c) ground loss and slumping.

A.2 The relevant Verification Methods and standards referred to in this determination

Verification Methods

Amendment 6 of the compliance documents for Code Clause B1 took effect from 1 March 2005. This included:

- **Verification Method B1/VM1 (General),** which referenced NZS 4203: 1992, NZS 3101:1995 and NZS 3404:1997 (with modifications) as the means of compliance for loadings, concrete and steel respectively**,** and
- **Verification Method B1/VM4** (Foundations).

Standards

NZS 4203:1992 Standard for General Structural Design and Design Loading for Buildings (known as the loading standard)

This standard was written in limit state format. It stipulates the level of loading which was required to be imposed on a building when assessing its ability to satisfy a particular limit state and also stipulates a means of calculating the distribution of loading elements of the building.

> Clause 4.12 Requirements for parts
>
> > 4.12.1.1 All structures, including permanent non-structural components and their connections, and the connections for permanent services equipment supported by structures, shall be designed for the seismic forces specified herein. The value of the risk factor for the parts shall be as provided in table 4.12.1…

NZS 3404:1997 (known as the steel structures standard)

No clauses are relevant to the current decision.

NZS 3101: 1995 including Amendments 1, 2, 3 (known as the concrete structures standard)

> Clause 3.4.1
>
> > …
> >
> > members shall be designed for the ultimate limit state by providing strength and ductility and ensuring stability, as appropriate, in accordance with the relevant requirements of 3.4.2 to 3.4.4 …
>
> Clause 4.4.10 Elastically responding structures
>
> > Structures assumed in the terms of NZS 4203 to remain essentially elastic under the actions of appropriate gravity loads and seismic forces corresponding to the ultimate limit state in accordance with 4.4.1.1(c), shall be designed so as to satisfy the following criteria:
> >
> > (a) When the structural system is such that under seismic actions larger than anticipated, energy dissipating mechanisms could develop in the same form as required by 4.4.5 to 4.4.9 for ductile structures or those of limited ductility, the selected structure is exempt from the additional seismic requirements of this standard.
> >
> > (b) When energy dissipation would be possible only in a form not admitted in ductile structural systems or those of limited ductility, the relevant plastic mechanism or mechanisms shall be identified. Members of mechanisms so identified shall be detailed in accordance with the additional seismic requirements of this standard.

Clause 4.4.13

The interaction of all structural and non-structural elements which, due to seismic displacements, may affect the response of the structure or the performance of non-structural elements, shall be taken into account in the design of that structure.

Clause 7.3.1.2 Amendment 3:

Welded wire fabric shall have a uniform elongation, as defined as AS/NZS 4671, of at least 10%

Lesser ductile welded fabric may be used where:

(a) the yielding of reinforcement will not occur at the ultimate limit state: or

(b) the consequences of yielding or rupture does not affect the structural integrity of the structure

Clause 7.3.7.3

7.3.7.3

The development length, L_d, in tension may be determined from:

$$L_d = \frac{\alpha_b}{\alpha_c \alpha_d} L_{db} \geq 300 \text{ mm} \quad \text{(Eq. 7-3)}$$

with α_b, α_c and α_d being defined as follows:

(a) Reinforcement provided in a flexural member (not subjected to seismic forces nor required for temperature or shrinkage in restrained members) in excess of that required:

$$\alpha_b = A_{sr} / A_{sp}$$

(b) When cover to bars in excess of 1.5 d_b or clear distance between adjacent bars in excess of 1.5 d_b is provided:

$$\alpha_c = 1 + 0.5\left(\frac{c_m}{d_b} - 1.5\right) \quad \text{(Eq. 7-4)}$$

with the limitation of $1.0 \leq \alpha_c \leq 1.5$

where c_m = the lesser of the concrete cover or the clear distance between bars

(c) When transverse reinforcement with at least 3 bars, spaced less than $8d_b$, transverse to the bar being developed, and outside it, are provided within L_d:

$$\alpha_d = 1 + \sqrt{\left(\frac{A_{tr}}{s}\right)\left(\frac{f_{yt}}{80 n_L d_b}\right)} \quad \text{(Eq. 7-5)}$$

with the limitation of $1.0 \leq \alpha_d \leq 1.5$

Transverse reinforcement used for shear, flexure or temperature may be included in A_{tr}.

Clause 7.3.31 Wall reinforcement

Clause 12.3.2.2

Walls shall not be less than 100 mm thick for the uppermost 4m of wall height and for each successive 7.5m downward (or fraction thereof) minimum thickness shall be increased by 25mm. Bearing walls for two-storey buildings may be 100mm thick for the total wall height, provided that the compression stress over the gross area of the wall due to the ultimate limit state axial load does not exceed 0.2 *f'c.*

Clause 12.3.2.7

Limits to thickness and quantity of reinforcement required by 12.3.2 and 7.3.31 respectively may be waived where, instead of the empirical rules of 12.3.6, rational analysis or test shows adequate strength and stability at the ultimate limit state.

Clause 12.4.3.3

The diameter of the bars used in any part of a ductile wall shall not exceed one tenth of the thickness of the wall.

Appendix B: Summary of written submissions and other key documentation

The following table outlines the submissions and other key documents received under the headings:

- The application
- The expert's reports –refer Appendix C for a summary of these reports and the subsequent technical hearing
- The evaluative framework. –refer Appendix D for more detail of these responses.
- The first draft determination

For ease of reference, the following descriptions used in this determination are repeated here:

- the applicants, acting through their lawyer ("the applicants' lawyer") and the authority are parties to the determination
- "the design engineers" are persons with an interest who designed the buildings
- "the consultant engineers" were engaged by the applicants to independently assess the building design
- "the peer review engineers" were engaged by the design engineers to review their analysis of the building design using the Ministry's evaluation framework.

The application		
27 November 2012 Applicants' lawyer	Written submission, along with copies of: the DEE report from the consultant engineer; building consent drawings for BC12066501 for Lot 5 Princess Street dated 30 May 2006; producer statement construction ("PS4") by the design engineers; producer statement design ("PS1") by the design engineers; practical completion certificate; code compliance certificate issued by the authority dated 20 December 2007; various correspondence between the design engineers, the applicants' lawyer and the consultant engineers	Written submission seeking me to determine whether the following aspects of the buildings complied with Clause B1 Structure: 1. the minimum reinforcing content in the precast concrete wall panels (whether this complied with the relevant standard, NZS 3101: 1995) 2. the use of non-ductile mesh to reinforce the concrete wall panels (whether this complied with the relevant standard, NZS 3101: 1995) 3. the strength (capacity, shear and bending) of the concrete wall panels for in-plane and out-of-plane loads (whether this complied with the relevant standards, NZS 4203:1992 and NZS 3101:1995) 4. the precast wall panels met the ductility demand or curvature ductility requirements, strain limitations and the like of the relevant standards at the time of design 5. the precast wall panels and buildings, in terms of whether these met the Building Code's structural performance requirements 6. if all of, or elements of, the above complied with the verification method B1/VM1, which was stated on the PS1 producer statement for engineering works as the means of compliance, and 7. if the roof bracing capacity and connections complied with the relevant standards at the time of design. The applicants also asked me for a determination regarding the authority's decision to issue building consents and code compliance certificates for this work as shown in the

		structural drawings prepared by the design engineers and stamped ‘Consent’ by the authority on 30 May 2006.
4 February 2013 Design engineers’ lawyer	Submission	A copy of the complaint lodged by the applicants (and subsequently dismissed) with the Institution of Professional Engineers (IPENZ) about the conduct of the design engineers. The design engineers’ lawyer discussed the background of the commercial dispute between the parties in relation to the final drawings not being the final drawings for the site and unpaid fees. The design engineers’ lawyer would make the drawings available to the Chief Executive for the purposes of this determination.
13 February 2013 Applicants’ lawyer	Submission	The applicants were unaware of the existence of a ‘final version’ of the drawings. The commercial dispute was acknowledged.
10 May 2013 Applicants’ lawyer	Submission	The applicants’ lawyer provided the consent drawings currently in possession of the applicants.
21 February 2014 Authority	Written submission from Authority, along with copies of: Building Consent documents for all Stages (1-4); and the authority’s complete property file for the buildings (on computer disc)	In relation to its issue of building consents: • Structural calculations were not generally required by the authority but were requested on a case by case basis where they were considered necessary to verify compliance. • A senior structural engineer assessed the buildings in this case. The construction was “relatively simple” and modest in size using methods that were, and are, common. • The design engineers had suitable qualifications and experience. The assurance in particular regard to the PS1 was that this was accepted as providing reasonable grounds to believe the building would comply with Clause B1 if completed in accordance with the consented documents. • A peer review or detailed analysis was not considered necessary in this case. In relation to its issue of code compliance certificates: • The detailed engineering inspections of a specific design building were beyond the expected capabilities of a building inspector. The inspector ensured the relevant inspections were carried out by the engineer and records completed. • The PS4 was considered to be reasonable grounds that the buildings complied with the consented documents, and therefore with Clause B1. The PS4 was supported by a PS3 completed by those carrying out the construction. The authority noted there was no indication in the documents that the construction was not in accordance with the consented documents. The determination application suggested the drawings used for construction were not the consented documents but the authority was not made aware of this at the time.
The expert’s reports		
25 September 2013 Consultant engineers	Response to draft report and questions to the expert	• The expert’s report stated the mesh did not comply; therefore the precast concrete wall elements should be designed for a ductility of 1.0 not 1.25. • While the shear strength of the walls was checked in

		building three, the critical walls to this building did not have window openings as in the other buildings where the walls supported a suspended concrete floor. Therefore, the critical wall section for shear was adjacent to these openings. • The capacity of the base of the wall panels and connection to the floor slab should be checked.
20 November 2013 Design engineers' lawyer	Initial response to the expert's report	Letter asking for time for the design engineers to provide a detailed technical review of the expert's reports, and providing an indication of what they saw as key issues.
11 and 13 March 2014 Design engineers	Technical submission (report) from the design engineers in response to the expert's reports	• The design engineers maintained the view the buildings were code compliant at the time they were built. • They stressed that the applicable standards and Verification Methods were not the only means of achieving compliance with the Building Code. • The assumption that the drawing and specifications reviewed by the expert were the as-built construction was inappropriate. They said the design was modified via site instructions during the course of construction, and they recognised some areas in which the buildings did not conform to the drawings. • The design engineers noted that the buildings had performed 'very well' in the Canterbury earthquakes of 2010/11 and the type of mesh reinforcing used in them had been adopted in hundreds of other concrete buildings in Christchurch. They considered many of the issues the expert identified arose from a mistaken interpretation of the applicable standards, saying that: o NZS 3101:1995 Clause 3.4.1 was misinterpreted as mandating that members had a ductile response in all circumstances. However, this was incorrect and did not provide the full context for the clause. o Clause 3.8.4.1 of B1/VM1 had nothing to do with mesh; therefore they did not agree that the Verification Method did not permit the use of mesh in wall panels or diaphragms that are part of the seismic resisting structure. o In relation to medium wall thickness and slenderness the prevailing industry practice and literature accepted H/t ratios of above 30. Therefore, in this case 1/56 was within the acceptable range. o The Verification Method did not specify when ties are required; only where they are 'definitely not required'. • The design engineers noted that not all the expert's calculations were made available and therefore they could not comment on all these calculations (for example, the roof bracing calculations).
19 March 2014 Applicants' lawyer	Submission	Letter stating that the design engineers and the expert should not directly confer.
28 April 2014 Consultant engineers	Technical submission in response to the design engineers' technical submission of 11 March 2014	• Alternative solutions must demonstrate compliance with the performance requirements of the Building Code by providing comparison compliance documentation, testing and research, and/or peer reviewing to an acceptable level generally guided by

		industry standard or the authority's policy. • In this case, the PS1 producer statement from the design engineers clearly outlined the design was prepared in accordance with B1/VM1 and B1/VM4. The authority was not notified of any alternative solutions to prove code compliance. • In relation to specific matters: o Non-ductile welded wire mesh did not meet the general conditions or additional seismic conditions of NZS 3101:1995 for all three ductility states (ductile, limited ductile or elastically responding) as there is no provision in the code for non-ductile mesh. There is limited ability to bypass ductility for any primary structural element or system, even for elastically responding structures. o Elastically responding structures still required yielding of the reinforcing and needed to meet the minimum general requirements of the concrete standard. o The non-ductile mesh round bars did not meet the requirements of Clause 3.8.4.1 of B1/VM1 due to the development of stress issues in round bars. The consultant engineers agreed with the expert's interpretation of this clause. o In relation to wall slenderness the consultant engineers agreed with the expert that in order to waive the requirements of clauses 12.3.2 and 7.3.31 of NZS 3101: 1995 the reinforcing must comply with the requirements of NZS 3101:1995. In this case the mesh did not comply. • The consultant engineer identified areas where further investigation was needed by the design engineers: for example ties, roof plane bracing and vertical wall reinforcement.
The evaluative framework		
3 October 2014 Applicants' lawyers	Submission	The applicants' lawyers said the proposed evaluative framework did not address the first two issues raised in the application for determination [refer to submission of 27 November 2012 at top of this table]. They said compliance (or non-compliance) of the use of non-ductile mesh reinforcing in the precast concrete wall panels with the relevant standards in place at the time of design needs to be determined before any further evaluation of the building takes place. The consultant engineers had provided an opinion on this matter. They also said the evaluative framework contradicts issue 6. The PS1 issued for the building by the design engineers stated that the design was undertaken in accordance with B1/VM1. It is not appropriate to check the buildings retrospectively by an alternative solution method. An alternative solution would require a peer review to be undertaken at the time of design – this did not occur.
5 February 2015 Applicants' lawyers	Submission	Letter objecting to delays to design engineers' assessment

13 February 2015 Design engineers' lawyer	Submission of findings 'to date' including a four page summary assessment and calculations from the design engineers	Brief summary of findings to date and statement that this assessment had been carried out in response to the Ministry's 'scoping outline' (evaluative framework). The design engineers recognised the significance of these issues and would undertake some additional work as well as obtain an external peer review.
17 April 2015 Design engineers and peer review engineers	Technical submission from the design engineers including the report from the peer review engineers [Refer Appendix D for more details of this response]	The design engineers undertook the structural design of the building primarily in accordance with the Verification Method B1/VM1. As often occurs, limited aspects of the design were undertaken in accordance with accepted industry practice beyond the requirements of the standards at the time of design. It is relevant to note that such practices were later confirmed as good practice with their inclusion in a subsequent updating of the design standard. The peer review engineers said their report found the panel design had an acceptable ductile mechanism that did not require ductile behaviour from the precast concrete panels themselves, and hence did not require ductile behaviour from the HRC mesh (and the design standard at the time allowed the use of this mesh under these circumstances). The panels (with some exceptions) complied with the minimum reinforcement requirements and the panel design (with some exceptions) was compliant under seismic actions. The panels had not been provided with adequate restraint at roof level and the roof bracing did not have adequate capacity.
16 June 2015 Consultant engineers	Technical submission in response to the peer review report of 17 April 2015 and the design engineers' calculations of 13 February 2015 [Refer Appendix D for more details of this response]	• The consultant engineers consider the Verification Methods are the only basis that can be used as a means for testing compliance with the Building Code in this case, as the PS1 clearly outlines that the design has been prepared in accordance with B1/VM1 and B1/VM4. • They also considers the adoption of an innovative design process does not allow a designer to deviate from prescribed Verification Methods without a sufficient review and qualification process as an alternative solution. • Review of the design methodology of building 5 has identified that the building has been designed and detailed with brittle failure mechanisms. Yielding of these mechanisms at the ultimate limit state will affect the structural integrity and stability. Therefore, the design does not meet the underlying fundamentals of seismic design of the Building Code or the engineering profession, as there are no dependable yielding mechanisms.
The first draft determination		
26 August 2015 Peer review engineers	Technical submission in response to the first draft determination	• In relation to the panel out-of-plane face loads, the design engineers provided the calculations for calculating the horizontal seismic coefficient for parts in NZS 4203 stating conservative assumptions are made to simplify the calculations. Conservatively C = 1.02 however more accurate parameters calculate C=0.39. • The design engineers stated that the calculations show the panels are more governed by ground accelerations than by accelerations induced through the seismic response of the building. • The design engineer provided particular calculations

		for Panel 186 for re-analysis which showed the reduced ultimate limit state capacity of the panel is adequate under a horizontal seismic coefficient of C=0.83 • In relation to panel slenderness, the use of slenderness ratios in excess of the code nominated values was industry practice at the time the design was undertaken. The Poole report records the Ministry was aware of industry practice. • In reference to the first draft's comments about the design engineers misleading the authority in issuing the PS1 with a stated means of compliance being B1/VM1, the design included aspects of current practice outside of the verification method. The Verification Method requires engineering judgement, it is inherent in structural design for the results of research and innovative design to be incorporated as part of standard design practice as it takes some time for the Ministry to cite the Standards that incorporate the advances. • The Ministry needs to consider how innovative design may progress for the benefit of the public if working outside a Verification Method uses aspects of current practice.
8 September 2015 Consultant engineers	Technical submission in response to the first draft determination and the design engineer's submission above.	• The ductile mechanism of the panel design requires further consideration. • The detailing of the concrete panels and the subsequent capacity checking foes not meet the SESOC publication guidance and therefore the requirements of an alternative solution. • The panels do not meet the requirements of NZS3101:1995 as the starter bar lengths do not meet the development length requirements and a brittle bond failure will occur. • The consultant engineers have concerns with the Ministry treating the SESOC publication as industry guidance. • A relaxation of the slenderness limits in NZS 3101:1995 is inappropriate. • The panels do not comply with the minimum reinforcing requirements of NS 3101:1995. • The panels do not have sufficient capacity to support the design loads and therefore do not meet the minimum requirements of the Building Code Clause B1. • The consultant engineers agree with the first draft in relation to the comments regarding reliance with the PS1.
16 September 2015 Peer Review engineers	Technical submission in response to the consultant engineer's submission of 8 September 2015 noted above.	• The consultant engineers challenged the standing of the SESOC publication; the paper was written by the chair of the Concrete Design Committee that wrote NZS3101:1995. • The SESOC guidance included calculation examples but was not written in a prescriptive manner specifying detailing. • The practices applied in the SESOC guidance were confirmed as good practice with their inclusion in a subsequent updating of the Design Standard • The peer review engineers are of the understanding a

		version of the drawings were provided to the authority at the time of the Building Consent application, and it was not common practice for the authority to require structural calculations. It is agreed changes such as significant increases in the floor area would be considered a major variation from the consent documents. • It is unlikely a peer review would have required more background to the design concept given industry practice and the SESOC guidance • The design engineers did not consider their design as an alternative solution; as often occurs limited aspects of the design were undertaken in accordance with industry practice which was not formally cited under B1/VM1. It is 'virtually impossible' to build using only the Acceptable Solutions and Verification Methods. The Ministry must decide if all design work utilising industry guidance not formally cited under the Building Code to be an alternative solution and what implications this would have for future and past building consents issued.
8 October 2015 Consultant engineer's	Technical clarification submission in response to the peer review engineer's report of 16 September 2015	• The publication *(Seismic Design Aspects for Tilt-up Buildings 1996)* was <u>not</u> standard industry practice in 2006 and was not adopted into any further design codes, amendments or any further publications. • The adopted methodology considers cracked wall sections for out-of-plane actions and uncracked sections for in-plane actions. The design of the concrete panels cannot consider both cracked and uncracked behaviour at the ultimate limit state for the same system. • The SESOC publication (a journal publication not a guidance paper) is not considered standard design under the Building Code currently, nor in 2006. At the time NZS3101:1995 was the required standard. • The calculations provided by the peer review engineer are not applicable to all panels in the building (for example panel 186). The consultant engineer considers some of the panels would fail if the calculations provided were adopted, therefore the building as a whole would not comply with the Building Code. • The consultant engineer considered the calculations to be incorrect as: o they have not proved or provided evidence that the bar stress from the base starters can be developed into the panel across the joint interface so compatibility of force cannot be obtained o the calculation method does not meet the requirements of the Building Code or other technical publication o the panel would have to crack to transfer bar stress into the panel reinforcement, which would then not comply with the uncracked methodology of the SESOC publication • The failure of one element of the design defines non-compliance of the structure under the Building Code. • The peer review engineer has not provided any further evidence in relation to the capacity of the connection

		of the cantilever action at the panel base. The design does not comply with the Building Code for the building as a whole. • The peer review engineer argued the SESOC publication complies with B1/VM1 due to the testing carried out as part of the research for the publication. The testing was not verified for compliance with B1. • The consultant engineer considered the determination must be site specific and therefore should consider all buildings at the site not just building number 5.
14 October 2015 Peer review engineers	Submission in response to the consultant engineer's submission of 8 October 2015	• The peer review engineer stated the design did not consider the cracked and uncracked behaviour in the same system but merely noted the critical action is governed by whether a panel is connected to the mezzanine floor. • Out-of-plane actions are most relevant for panels that do not have the mezzanine floor. Panel in-plane action resists the seismic weight of the panels themselves, and the seismic weight of the roof. These in-plane actions are insufficient to yield the mesh when subjected to elastic seismic demands. If the concrete has already cracked from prior out-of-plane behaviour, there is still a satisfactory mechanism to resist in-plane seismic actions. Calculations demonstrating the mesh does not yield were provided. • In-plane actions are most relevant for panels that are connected to the mezzanine floor. These provide the seismic lateral load resistance for the weight of the panels themselves, the weight of the mezzanine floor and the weight of the roof and supported by mezzanine floor for the out-of-plane actions. These are insufficient to crack the concrete when subjected to elastic seismic demands. Calculations were provided
29 October 2015 Consultant engineers	Submission in response to the peer review engineer's submission of 14 October 2015	• NZS3101:1995 does not allow consideration of the tensile strength of concrete for flexural calculations at the ultimate limit state. • It was not acceptable in the industry at the time to rely on tensile strength of concrete at the ultimate limit state. • The uncracked methodology does not consider cracking that would occur due to other loading conditions such as thermal cracking, shrinkage cracking and cracking during construction. No evidence has been provided to prove that the panels would not crack under these loading conditions, cracking will occur. • No evidence of the panel testing and inspection regime that would have been required during the construction of the building was provided to ensure the uncracked methodology 'holds true'. • The building would be considered a brittle structure (based on the adopted uncracked methodology) under NZZ4203:1992 • The peer review calculations for panels without mezzanine rely on the non-ductile reinforcing for in plane actions as the calculations show cracking for out-of-plane loading. • The building design did not meet any requirements of

		the SESOC publication, not B1/VM1 and was not in accordance with industry practice at the time of design. The buildings therefore did not comply with the Building Code.

Appendix C: The expert's reports and the technical hearing

The following tables include brief summaries of:

- the expert's reports and his response to the design engineers' submission on these ("the expert's response"), and
- some key points from the technical hearing held 3 June 2014, as this discussion was structured around responses to the expert's reports.

The expert's reports
Building design • Two producer statements (PS1s) for the buildings stated that the designs were prepared in accordance with B1/VM1 and B1/VM4. • In considering the building design against NZS 4203:1992 and NZS 3101:1995 (standards cited in these Verification Methods at the time the buildings were designed), the expert concluded: o Minimum requirements for horizontal shear reinforcement were not met, even where steel was provided in addition to HRC mesh. o None of the 150mm thick panels complied with minimum reinforcing requirements. o The HRC mesh could not be considered as contributing to member strength where ductility was required. o Ductility requirements were not met as a result on dependence of non-ductile HRC mesh for reinforcing. o Connections between the diaphragm and the walls did not meet NZS 3101:1995. o Wall panels had about two thirds of the strength required to resist out-of-plane seismic loads. o There was no evidence that roof bracing capacity and connections complied with the relevant standards at the time of design. The bracing provided did not have adequate strength.
The building consents and code compliance certificates • Under section 49 of the Act, part of being satisfied on 'reasonable grounds' that the provisions of the Building Code would be met could be a reliance on the competence of the designer. In this case a signed PS1 from the design engineer could have met this test. • The files did not contain any structural calculations. The expert assumed none had been submitted and the consent could not have been used as a check on the accuracy or completeness of the structural design. • Full construction details for the roof plane bracing were not added to the drawings until after the consents were issued. A detailed assessment of the complete structure could not have been carried out at the time the consents were sought. • The drawings submitted with the building consent documents did not comply with the relevant design standards in a number of respects. • In issuing the code compliance certificate the authority has relied on the PS4.
Building construction • The expert was unable (apart from a visual inspection and verification of a sample of the wall panel reinforcing steel) to verify to what extent the building construction was code compliant as the buildings had been completed for some years.
The expert's response
(Letter dated 8 May 2014) Further points made, in reply to the design engineers' submission of 11 March 2014, included: • Why the expert's review was limited to consideration of compliance with design standards referenced in the verification methods (these were the terms of the brief; and as the PS1 stated the design had been prepared in compliance with these verification methods) • The expert was limited to assess the construction as distinct from the design and documentation of the buildings as they were not able to observe construction or verify hidden details. The expert was limited to the documentation provided.

- The use of hard drawn mesh was not explicitly confirmed however strongly implied.
- NZS 3101 Clause 12.3.2.7 allowed a waiver of the limits of thickness and quantity of reinforcement where rational analysis or test shows adequate strength and stability at the ultimate limit state. There was no evidence that such methods had been applied in this case.
- The expert accepts the design engineer’s view that ties were not required around the concentrations of longitudinal bars.
- The PS1 made reference to B1/VM1, VM4 and the PS4 do not make reference to specific design requirements. It is noted that in appropriate spaces on the PS4 form for listing building consent amendments there are no changes recorded to the original documentation. The only standards referenced in B1/VM1 and VM4 as the declared design basis for the building.

The technical hearing: some key discussion points

General

- The consultant engineers said if the alternative solution approach had been signalled as a basis of verification (instead of the Verification Methods noted on the PS1 producer statements) the BCA could have decided what, if any, further peer review if might have required, as per its standard procedures.

Ductility

- The discussion considered whether the reinforcing mesh used in the buildings was ‘non-ductile’.
- The design engineers considered the mesh has limited ductility but not ‘no’ ductility and therefore complied with the relevant standard. They said stated Amendment 3 to Clause 7.3.1.2 of NZS 3101:1995 did not preclude the use of mesh, and limited ductility strength mesh could be used in some circumstances. The mesh does exhibit some yielding; therefore a μ=1.25 was allowed to be used. They also noted there was a recognition the buildings (post-Canterbury earthquake sequence) had other elements that contributed to the whole system.
- The consultant engineers disagreed, referring to Amendment 3 to Clause 7.3.1.2 and saying that ultimate limit state is the point at which one tries to stop collapse. Yielding would occur at the members at ultimate limit state. The ductility factor was μ=1.0.

It was noted it was necessary to distinguish between the primary structure and secondary elements.

Face loads

- The consultant engineers considered there was a high concentration of steel that would not allow for the panels to yield and said the minimum requirements of the Building Code had not been met.
- The design engineers considered plain round bars were not appropriate.

Slenderness

- The expert considered the wall thickness and slenderness had not met the Verification Method relevant at the time and this could be an alternative solution.
- There was some discussion relating to the fact that not all the wall panels were the same. In particular, the horizontal panels above the cantilevered floor areas were an area of concern for the expert.
- The discussion relating to wall slenderness focussed on whether the mesh was accepted to be non-ductile or had limited ductility (as noted above).

Reinforcement

- The design engineers described their interpretation of the relevant standard (NZS 3101:1995) and how their design philosophy related to that. A cross section of the parallel bars and panels was drawn.
- The design engineers did not consider the yielding of a bar would affect structural integrity (referring to the phrase ‘consequences of yielding or rupture’ under Clause 3.4.1 of NZS 3101:1995). The consultant engineers disagreed.
- The design engineers considered there were no plain round bars and Clause 3.8.4.1 of B1/VM1 did not apply to welded wire mesh.

Appendix D: The evaluative framework

The following table summarises responses to the evaluative framework relevant to the buildings' compliance with the Building Code (Matter One); in particular, the analysis by the design engineers (reported by the peer review engineers) of one of the applicants' buildings using this framework, and the consultant engineers' response and the parties responses to the draft determination analysis of the evaluative framework. Headings parallel those used in section 10.

Issue A: The precast concrete panels – ductility		
Peer review engineers and design engineers	17 April 2015	The peer review engineers found that the panel design had an acceptable ductile mechanism that does not require ductile behaviour from the precast concrete panels themselves, and therefore did not require ductile behaviour from the HRC mesh. • In relation to Amendment 3 of Clause 7.3.1.2 of NZS 3101:1995 the peer review engineers considered the paragraph at issue covered the use of lesser ductile mesh, allowing for the use where a) the yielding of reinforcement will not occur at the ultimate limit state **or** (emphasis added) b) the consequence of yielding or rupture does not affect the structural integrity of the structure. • The peer review engineers considered the hard drawn welded plain wire HRC mesh would not yield under the design ultimate limit state earthquake. The ductile behaviour is achieved at the footing to the panel joint in accordance with the SESOC guidance at the time.
Consultant engineers	16 June 2015	• The use of mesh does not meet NZS 3101:1995 as it does not comply with the ductility requirements. The Verification Method disallows the use of non-ductile wire mesh. • The in-plane design of the concrete walls for building 5 has been based on ductile behaviour of the base starters at the panel-foundation joint. The consultant engineers do not consider the precast panels exhibit ductile behaviour.
Consultant engineers	8 September 2015	• The detailing of the concrete panels and subsequent capacity checking does not meet the requirements of the SESOC guidance and therefore does not meet the requirements of an alternative solution if accepted, which is unlikely without further testing.
Issue A: The precast concrete panels– in-plane and out-of-plane face loads		
Consultant engineers	16 June 2015	• The calculated in-plane bending is 14% and in-plane shear load is 25% NBS using a ductility of 1.0. The calculated strength of the wall panels for out-of-plane bending loads is 22% NBS. • The only way to achieve compliance of the in-plane response of the concrete walls is to prove that no yielding of the non-ductile mesh reinforcement will occur at ultimate limit state (μ=1 design actions). The design engineers' calculations have conducted the panel assessment based on μ=1.25 design actions. Yielding will occur at ultimate limit state which will compromise the structural integrity and stability.
Peer review engineers	17 April 2015	• The in-plane seismic response of the precast panels in the

		areas of the structure with the first floor level using the equivalent static force method were modelled as this area was most critical. The results of the modelling showed the design satisfies requirements for an adequate seismic restraint system, and the panels are shown to be protected from ductile deformation. • The critical panels for out-of-plane seismic actions were analysed. The results showed for panels supporting and restrained by the first floor, the displacements and P-delta effects are negligible. • The wall panels were assessed and established that the panel yield strength has 'superior strength' to the over-strength capacity of the starter bar plastic hinge system, and the strains induced in the panel reinforcement are within acceptable levels. Yielding of the reinforcement in the panel will therefore not occur. • The peer review engineers identified two panels as exceptions to this. However, they said their: "reduced nominal flexural strength was greater than the nominally elastic μ=1.25". • The panels have been designed with the base fixed, the top propped and an intermediate height propped at the first floor level when applicable. Out-of-plane accelerations have been assessed using NZS 4203:1992 Section 4.12 "Requirements for parts", which in this case is conservative and has the "significant effect" of double accelerations.
Consultant engineers	8 September 2015	• The peer review engineers provided design calculations for a concrete panel that is considered most critical for out-of-plane stability; the panel is noted as being supported at the roof and ground level only. The consultant engineers consider the assessment relies on the panels acting as a propped cantilever, however as the roof diaphragm does not have sufficient capacity to support the loading of the panels they cannot be assessed in isolation to the rest of the building structure. • No assessment has been made on the capacity of the cantilever action at the panel base. • The assessment assumes centrally located mesh and bar reinforcement, whereas the mesh cannot be centrally located as it would clash with the drossbach ducts for the starter bars. • If the panels were assessed as fixed base only with reinforcement that is not centrally located they would have insufficient capacity when assessed using the correct design assumptions.
Peer review engineers	16 September 2015	• The panels are considered too narrow to form thermal cracks, the longer panels (inside) will only undergo minimal temperature change. It is noted the SESOC guidance was not written in a prescriptive manner specifying detailing. • The starter bars of the design are within grouted drossbach ducts which provide confinement, and a larger surface area to transfer tensile forces to the surrounding concrete and mesh, allowing their full tensile capacity to be developed. • The panels that are most affected by out of plane actions are those that are not connected to the first floor, while the panels that work hard in plane are those that support the first floor. • The failure of some individual panels to meet minimum reinforcement requirements should be regarded as a

		localised non-compliance of the detailing, not the structural design as a whole. • The assessment does not assume centrally located mesh and bar reinforcement, the calculations of panel strength used the actual locations of the bars, for out-of-plane actions the bars were modelled as being either side of the mesh.
Issue A: The precast concrete panels – slenderness		
Peer review engineers	17 April 2015	NZS 3101:1995 Clause 12.3.2.2 sets minimum thickness for walls depending on height. The expert recorded that many panels in the building are over 7.7m in height and 120mm thick. Therefore the complying minimum thickness is 125mm for 7.7m high walls. • The review reported to the Ministry's predecessor the Department of Building and Housing on August 2005 by R. Poole[27] identified that H/t ratios of 67 are common and that these often exceeded 80 for industrial buildings (of similar type to the applicants' buildings). The Poole report allows relaxation of the standard where analysis and test results show adequate strength and stability at the ultimate state. • The peer review engineers consider the assessment of the equations through NZS 3101:2006 confirms the wall thickness and slenderness are acceptable. • It is noted that on all 120mm panels the design engineers' drawings specified the outer layer of reinforcing bars and ties were to be galvanised. This indicates that durability at this location was considered in the design, and addressed.
Consultant engineers	16 June 2015	The roof panels would be required to have a minimum wall thickness of 124mm to comply with NZS 3101:1995 but these panels are typically 120mm thick. The consultant engineers consider the increased slenderness limit should only be applied if the concrete walls meet the minimum requirements of the design standard in regards to minimum reinforcement content, ductility requirements and support conditions.
Peer review engineers	26 August 2015	The use of slenderness ratios in excess of the code nominated values was industry practice at the time the design was undertaken, the Poole Report records that the Ministry was well aware of the industry practice.
Issue A: The precast concrete panels – reinforcement		
Peer review engineers	17 April 2015	Clause 12.4.3.3 of NZS 3101:1995 in relation to longitudinal reinforcement states that the diameter of the bars used in any part of a ductile wall shall not exceed one tenth the thickness of the wall. The peer review engineers consider this clause relates to longitudinal reinforcement within the wall and does not apply to HD16 starters bars, which are a connection of the panels to the foundation beams. They noted the strain demands on the bars were very low, the upper section of the starter bars were confined within the drossbach ducts and, as such, buckling failure was prevented and the lower sections were within a thicker footing. The peer review engineers also quoted from Practice Advisory 3 issued in June 2005 by the Department of Building and Housing.[28]

[27] Report to Department of Building and Housing: Review of Design and Construction of Slender Precast Concrete Walls, August 2005, R.A Poole

[28] Practice Advisory 3 *'Beware of Limitations, Cold-worked wire mesh"* Department of Building and Housing June 2005.

Consultant engineers	16 June 2015	• There is insufficient cover to the reinforcement bars that pass outside of the ducts for the specified 25MPa concrete strength under NZS 3101:1995. • The starter bar length is insufficient to develop the tensile strength of the bar into the mesh reinforcement and does not comply with Clause 7.3.7.3 of NZS 3101:1995.
Consultant engineers	8 September 2015	• The draft determination considered the panels complied with the minimum reinforcing requirement with some exceptions; the consultant engineer considered the exceptions mean the panels do not comply with the Building Code.
Issue B: Roof bracing		
Peer review engineers	17 April 2015	The panels have not been provided with adequate restraint at roof level, and the roof bracing does not have adequate capacity. • The loading estimates showed that the weight of the precast concrete panels acting at the roof level was much larger than the steelwork roofing weight. The calculations indicate the roof bracing may have been able to support the roof if the panels had been self-supporting, but the panels require top restraint and therefore rely upon this bracing. • The peer review engineers noted the expert had raised this as a concern, and the calculations provided by the design engineers confirmed this was the case. The design engineers assessed the roof bracing as being understrength at 34%. • The peer review engineers were concerned with the point connections producing stress concentrations in the panel, and the restraint of the panels consisting only of purlin connections with stud anchors.
Consultant engineers	16 June 2015	The design engineers’ calculations indicate the roof bracing is understrength at 34% NBS. The consultant engineers consider the roof plane bracing and associated connections did not meet the requirements of the Building Code at the time of design.