MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HIKINA WHAKATUTUKI

Determination 2019/060

Regarding compliance with Building Code Clause B1 Structure of a multi-storey steel framed building at 230 High St, Christchurch

Summary

This determination considers the compliance of a recently-completed eight-storey steel framed building. Specific aspects of the completed frame had been brought to the notice of the authority, and the authority also questioned work carried out that varied from the approved consent. The determination considers the compliance of these matters in respect of Building Code Clause B1 Structure.

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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, Katie Gordon, Manager Determinations, Ministry of Business, Innovation and Employment ("the Ministry"), for and on behalf of the Chief Executive of the Ministry.

1.2 The parties to this determination are:

- Christchurch City Council carrying out its duties as a territorial authority or building consent authority, and which applied for the determination ("the authority")
- Rockwell E & C Ltd, the owner of the building ("the owner"), acting through an agent ("the agent").

[1] The Building Act and Building Code are available at www.legislation.govt.nz. The Building Code is contained in Schedule 1 of the Building Regulations 1992. Information about the Building Act and Building Code is available at www.building.govt.nz, as well as past determinations, compliance documents and guidance issued by the Ministry.

1.3 I consider the following to be persons with an interest:

- Seismotech Consulting Ltd, the engineering consultancy firm that designed the building's structure ("the design engineer")
- Miyamoto International NZ Ltd, the engineering firm that conducted a peer review of this design ("the peer reviewer")
- Aurecon New Zealand Ltd, an engineering and infrastructure advisory firm which approached the authority with concerns about the building's structure ("the independent engineers").

1.4 The determination is about an eight-storey steel-framed building in central Christchurch. During the building's construction the independent engineers wrote to the authority with a list of concerns about its structural design, prompting the authority to investigate further. These concerns included a modification made during construction to a brace on the building's ground level ("the modified brace").

1.5 The authority's application for determination asked me to determine whether the building's superstructure[2], both as designed and as constructed, complies with Building Code Clause B1 Structure[3]. The authority has since agreed to refine this to determining compliance with Clause B1 with respect to the specific concerns identified by the independent engineers. I note that some of those specific concerns relate to the compliance of the building's foundations.

1.6 Accordingly, the matter to be determined[4] is whether the building as designed and constructed complies with Clause B1 with respect to the specific concerns raised by the independent engineers in their letter to the authority of 13 December 2017.

1.7 I have not considered any other aspects of the Act or Building Code beyond those required to decide on the matters to be determined.

1.8 In making my decision I have considered the parties' submissions; the reports and a further response from the independent expert I engaged, which is a firm of chartered professional engineers with specialist expertise in structural engineering and in the seismic design and assessment of buildings ("the expert"); and the other evidence in this matter.

1.9 Appendices to this determination are:

- Appendix A – Extracts from the legislation, the Verification Method and relevant standards.
- Appendix B – The independent engineers' concerns, design engineer's response, and expert's conclusions.
- Appendix C – Responses to the expert's first report, and the technical meeting.
- Appendix D – Further submissions and the expert's response.

[2] The superstructure includes that part of the building constructed above ground including the structural steel frame. The superstructure does not include the building's foundation.

[3] References to clauses in this determination are to clauses of the Building Code and to sections are to sections of the Act, unless otherwise specified.

[4] Under section 177(1)(a) of the Act

1.10 Matters outside this determination

1.10.1 The matters for determination do not include any decision by the authority regarding whether or not to identify the building as potentially earthquake prone using the powers available to it under section 133AG(1)(b) of the Act[5], or whether to consider the building earthquake prone as this term is defined in section 133AB. I note that the Ministry has drawn the parties' attention to the expert's conclusions regarding the building's seismic performance. I also note that the expert's report does not identify the building as meeting the test of a dangerous building as this is defined under section 121(1).

1.10.2 Submissions on behalf of the owner have asked whether the determination should consider whether the building consent should have been issued for the building's superstructure, as well as how any non-compliances with the Building Code should be rectified. As the authority's decision to issue the building consent was not one of the particular matters applied for by the authority this is outside the scope of the determination. Similarly, the consideration of possible options for remedying the non-compliance is outside the matters I can determine.

1.11 Key terms and concepts

1.11.1 Terms used in this determination relating to a building's performance during an earthquake include:

- **Elastic/elasticity**: describes the ability of a material or structure to return to its original size and shape when a force is removed. Elastic buildings are categorised as buildings for which $\mu = 1$, where μ is the structural displacement ductility factor.
- **Ductile/ductility**: describes 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[6]. Buildings may be categorised as nominally ductile ($1 < \mu \leq 1.25$), limited ductile ($1.25 < \mu \leq 3$) or fully ductile ($\mu > 3$).
- **Overstrength**: the maximum strength that a member [e.g. a beam or column] or a connection can develop due to variations in material strengths, and strength gain due to strain hardening, if applicable[7].
- **Torsional sensitivity**: a term used to describe a building's susceptibility to twisting around its centre of stiffness during an earthquake; i.e. to moving excessively at its extremities.

1.11.2 Design methods and analysis tools referred to include:

- **Capacity design**: the design method in which elements of the primary horizontal earthquake action resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations [i.e. under severe seismic forces]. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained[8].

[5] Section 133AG:Territorial authority must identify potentially earthquake-prone buildings
[6] As defined in "The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments", July 2017, Section C6: Structural Steel Buildings ("The Seismic Assessment Guidelines"), available from www.eq-assess.org.nz. The building categories (elastic, nominally ductile, etc) are also taken from these guidelines.
[7] As defined in the Seismic Assessment Guidelines (at www.eq-assess.org.nz)
[8] As defined in New Zealand Standard NZS 1170.5:2004 Structural design actions, Part 5: Earthquake actions – New Zealand

- **A proprietary structural analysis model** ("the PSA model") used by engineers to establish a structure's modes and periods of vibration and, from this, the earthquake forces that the primary structure must be designed for. The analysis model also predicts the structure's displacement at various locations, and produces forces and bending moments in the members modelled.

1.11.3 Relevant design features include the following (also refer to Figure 1):

- **Eccentrically braced frame (EBF)**: a braced frame in which at least one end of each brace frames only into a beam in such a way that at least one stable, deformable link beam is formed in each beam if the elastic limit of the frame is exceeded. In this event, energy is dissipated through shear and/or flexural yielding in the link beams (termed the active link regions) with the bracing members and columns having sufficient capacity to remain essentially elastic[9].
- **Active link:** the short section of a collector beam that is designed and detailed to undergo stable shear and/or flexural yielding and energy dissipation when the elastic limit of the frame is exceeded.

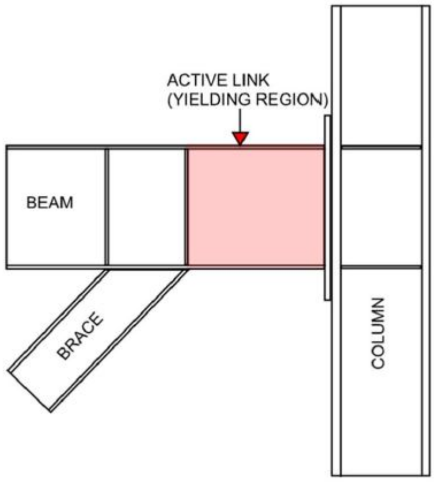

Figure 1: Example of an EBF showing the active link[10]

2. The building work

2.1 Building and location

2.1.1 The building is located at 230 High Street in central Christchurch, where it faces onto a busy shopping/pedestrian precinct. It is eight storeys high with a rectangular footprint. The building is relatively slender, occupying most of its narrow 232m^2 site. Its two neighbours on High Street are in close proximity: these are a two-storey commercial complex to the north and another new multi-storey building to the south.

2.1.2 The building was designed for retail use on the ground level, offices on the next five levels, and residential use on the top two levels. Construction was largely completed in 2018 but it is currently unoccupied.

[9] As defined in the Seismic Assessment Guidelines (atwww.eq-assess.org.nz)
[10] Figure 1 from the expert's response of 21 October 2019, which is described later in this determination

2.2 Site subsoil, foundation and concrete slab

2.2.1 The site is underlain by gravel and liquefiable silt and sandy soils, with the site subsoil class being D – "deep or soft soil" – under NZS 1170.5. The liquefiable soils were identified from a geotechnical investigation at a depth of between 3.5m and 5.5m, with gravel layers below this at a depth of approximately 5m to 10m.

2.2.2 The building's foundations comprise a 1.1m deep concrete raft slab on a 2m deep base of compacted gravel that was used to fill in a previous basement construction. Twenty-eight screw piles have been installed to a depth of 26.5-28.5m below ground level, and the shafts of these piles have been filled with 30 MPa-strength concrete.

2.3 The superstructure

2.3.1 The superstructure comprises steel framing for the support of gravity loads and to withstand lateral forces. There is a core comprising a lift shaft, single stairwell and services risers, which is located midway along the southern wall. The total floor area of the building is 1,540m^2, with floor to floor heights of 4.5m at ground level and 3.3m on other levels.

2.3.2 Floors are concrete topping on a proprietary timber infilled flooring system, with 200mm deep ribs spanning between the external walls to the north and south. The roof is steel cladding with lightweight steel purlins connected to steel beams. Exterior walls are timber studs with insulated metal panels.

2.3.3 The lateral load capacity is provided principally by eccentrically braced frames (EBFs) along the northern and southern walls (longitudinal Grids H, B1) and adjacent to the service core (transverse Grids 5, 6). Limited additional strength in the transverse direction is provided by two moment resisting frames on Grids 2 and 8. Refer to Figure 2.

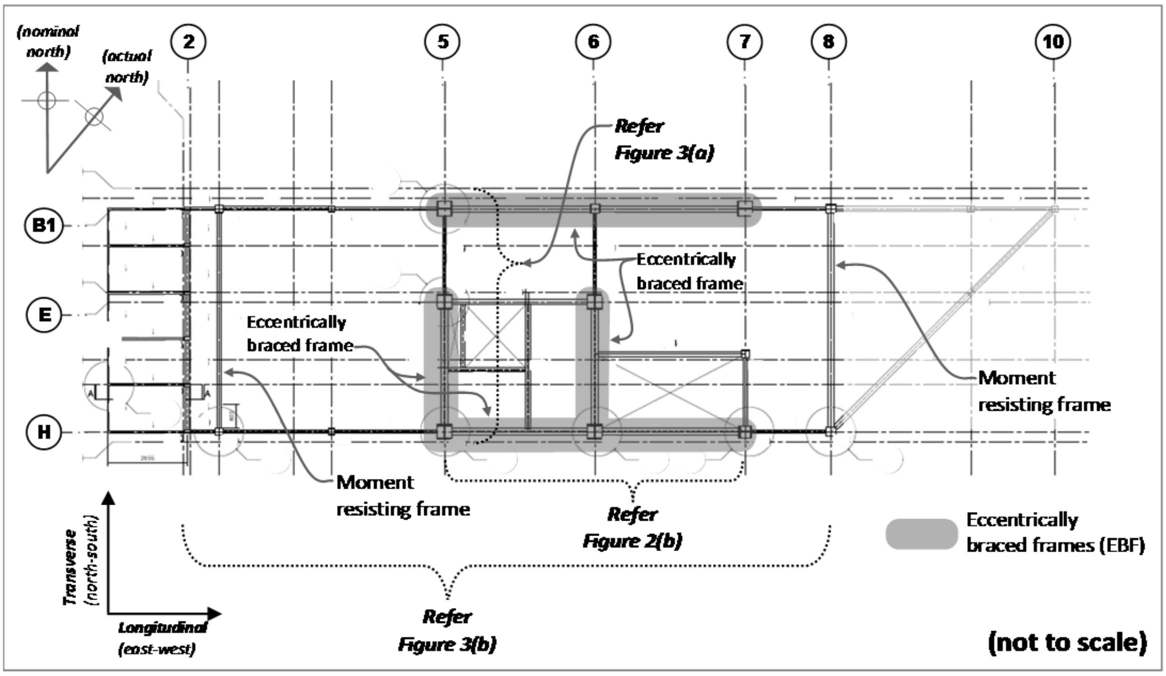

Figure 2: Building floor plan showing key gridlines - taken from building consent Drawing 100-S-016

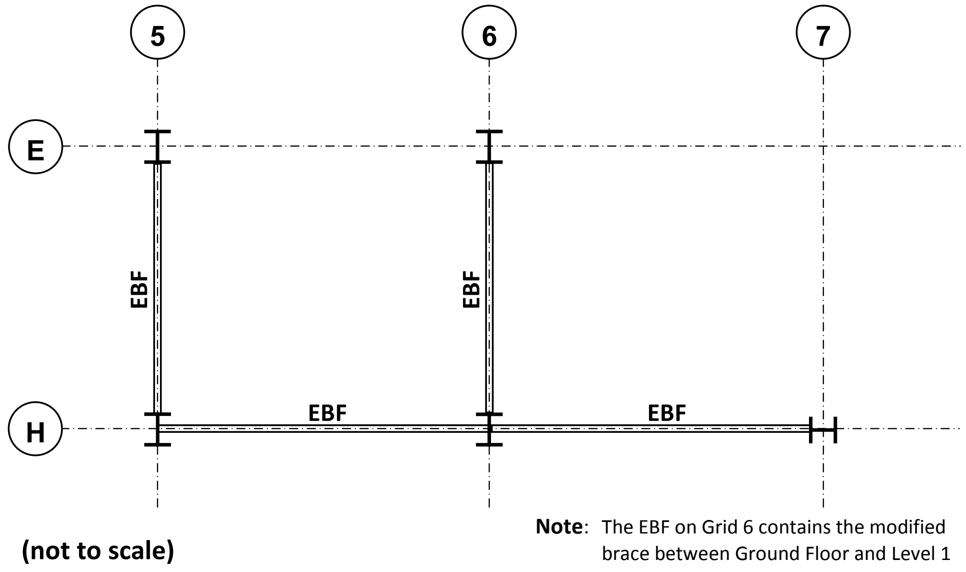

Figure 2b: Part floor plan showing EBFs on Grids 5, 6 and H

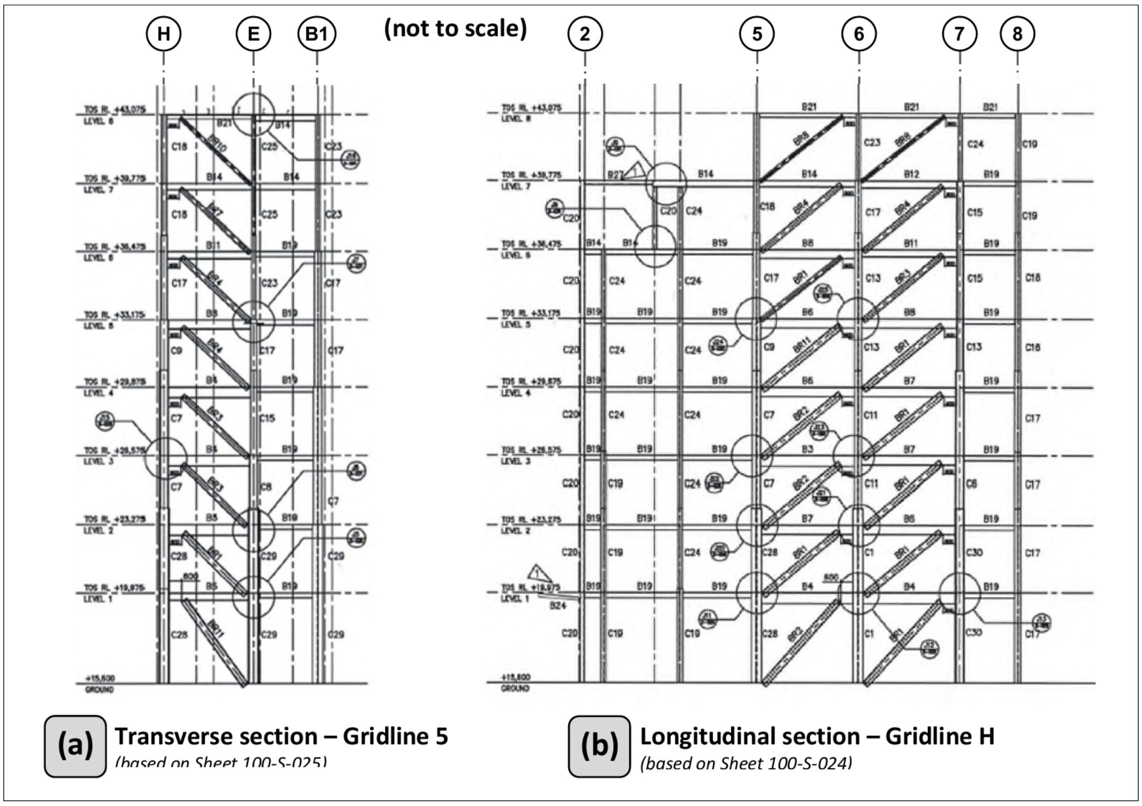

Figure 3: Frame elevations along Gridlines 5 and H – taken from building consent Drawings 100-S-024 and 100-S-025

2.4 The modified brace

2.4.1 During construction, the bracing of the EBF at ground level on Grid 6 next to the stairwell was modified to enable access to the stairwell (refer to Figure 4). This modification involved moving the upper section of the brace member further along the beam and adding a gusset section taken from a hot rolled section of steel, which I assume was to maintain the geometry of the active link adjacent to the column at Grid 6-H.

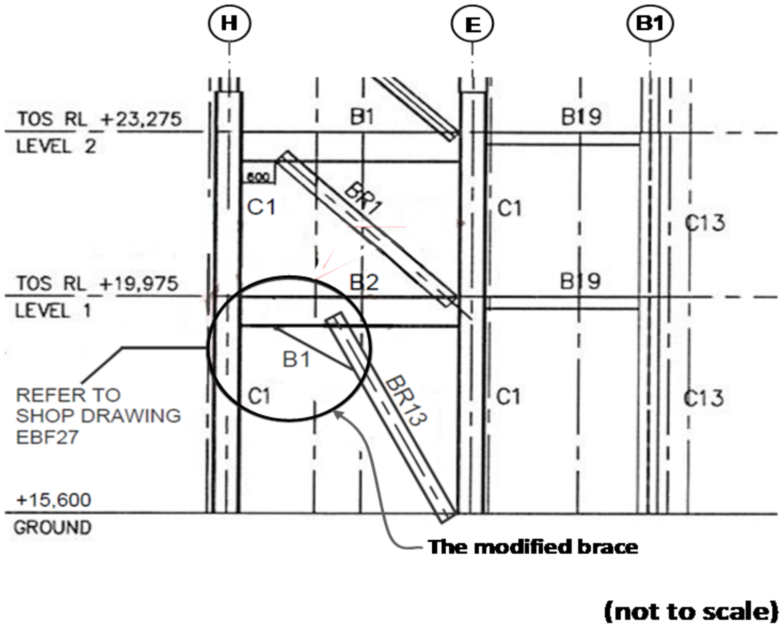

Figure 4: The modified brace on Grid 6 (circled) – taken from building consent Drawing 100-S-025 Rev 2

3. Background

3.1 Key events

3.1.1 The building's design and construction was consented in stages:

- Stage 1 "Foundations, concrete slab and services under slab" – the original building consent BCN 2015/8299 was issued on 19 November 2015 and the final code compliance certificate on 12 April 2018.
- Stage 2 "Steel superstructure only, excluding foundations" – building consent BCN 2016/2849 was issued on 15 August 2016, Amendment 1 to the consent on 2 February 2017, and Amendment 2 on 8 March 2018.
- Stage 3 "Building envelope and building services" – BCN 2015/12858 (I understand that a proposed Stage 4 was combined with this stage in late 2017).

3.1.2 Another engineering firm was responsible for the original foundation and slab design as well as for the structural design for the superstructure initially approved for the Stage 2 building consent. However, this originally consented superstructure design, which involved concentric bracing, was not used as the design engineer then became involved with the project and redesigned the building structure before work began. This new design was submitted as an application for Amendment 1 to the Stage 2 consent.

3.1.3 Key events leading up to the application for determination are summarised in the following table. The independent engineers' concerns about the building, the design engineer's response, and the authority's subsequent review are described in more detail in the paragraphs following the table.

Event	Date	Details
Stage 2 consent	15 Aug 2016	Stage 2 building consent issued for steel superstructure, excluding foundations **(**BCN 2016/2849) (this original design is superseded)
Amendment 1 to the Stage 2 consent	21 Dec 2016	Application for "Amendment to superstructure design", which involves a redesign by a new engineer (the design engineer) Application includes: • Relevant plans and specifications, including baseplate and slab layout plans (drawings 100-S-005, 006) • PS1 (producer statement – design) dated 19 December 2016 from the design engineer stating compliance with Clause B1 is in accordance with Verification Methods[11] B1/VM1 and B1/VM4; that the PS1 covers building work on drawings 100-S-000, 001 to 033; and that it is subject to site verification that "existing foundation and anchor bolts adequately constructed" • Design Features Report dated December 2016 prepared by the design engineer and reviewed by the peer reviewer • PS2 (producer statement – design review) dated 19 January 2017[12] from the peer reviewer for part of the work, being "superstructure peer review including the following documents: the drawings register referenced, the peer review register outlining all the items discussed, the Design Features Report"; the PS2 states the design was prepared in accordance with NZS 3404[13] and AS/NZS 1170 and with HERA Report R4-76[14] for the EBF design; it refers to drawings S000 to S033, and says the PS2 is subject to: "refer to comments in the attached drawings" • Also from the peer reviewer – a covering letter, peer review register dated 14 December 2016 and annotated drawings
	2 Feb 2017	Amendment 1 issued (BCN 2016/2849/A)
Authority inspections	28 Jun 2017	Inspections for pre pour floor (first suspended floor grids) and steel construction (Grids 5-10 on levels 1 and 2) – both passed
	3 Nov 2017	Final (superstructure) inspection – passed
Amendment 2 to the Stage 2 consent	29 Nov 2017	Application for "Amended structural details" (various design changes, some of which are to accommodate the cladding fixings)
	19 Dec 2017	Authority request for information (RFI): queries re some drawing details; says that no producer statements, design review log or other documents seem to be provided other than structural drawings revision 1
Independent engineers' concerns	13 Dec 2017	The independent engineers write to the authority with a list of concerns about the building's structural design (refer paragraph 3.2 and Appendix B)
Amendment 2 to the Stage 2 consent (continued)	23 Feb 2018	Design engineer's response to the authority's RFI includes: • PS1 dated 23 February 2018 from the design engineer for part of the work (for the amended structural details in Amendment 2, and with reference to drawings 100-S-100, 016, 022-025, 031 and 032), stating

[11] Acceptable Solutions and Verification Methods are produced by the Ministry and, if followed, must be accepted by a building consent authority as evidence of compliance with the Building Code.
[12] This was provided later to replace the PS2 that was dated 14 December 2016 and submitted with the original application, after the authority queried the date on that PS2.
[13] New Zealand Standard NZS 3404 Parts 1 and 2:1997 Steel Structures Standard; Amendment 2, October 2007: Amendment 2 was cited as a reference document to Clause B1 in 30 September 2010.
[14] New Zealand Heavy Engineering Research Association (HERA) Report R4-76 - Seismic Design Procedures for Steel Structures: 1995

Event	Date	Details
		compliance with Clauses B1 and B2 in accordance with NZS 1170, NZS 3101[15] and NZS 3404, and subject to site verification that "ground condition and steel base connections to be in accordance with Amendment 1 design" • PS2 dated 23 February 2018 from the peer reviewer for part of the work, being "superstructure peer review including the following documents: the drawings register referenced [listed as 100-S-000 and Rev 1 of 016, 022-025, 031-032], the peer review register outlining all the items discussed, calculations"; and stating that the PS2 is subject to: "refer to comments in the attached drawing S032" • also from the peer reviewer – annotated drawings and calculations, and a peer review register dated 16 January 2018
	8 Mar 2018	Amendment 2 issued (BCN 2016/2849/B)
Design engineer's response	21 Mar 2018	The design engineer responds to the authority about the independent engineers' concerns (refer paragraph 3.3 and Appendix B)
Inspection of the modified brace	12 Apr 2018	Authority inspection to identify the modified brace (site notice says a shop drawing for the modified brace was supplied and carried the design engineer's stamp of review and acceptance)
Application for code compliance certificate for Stage 2	4 May 2018	Owner applies for code compliance certificate, prompted by a reminder from the authority that the Stage 2 consent was issued nearly two years ago and the final inspection was passed Supporting documents include a PS4 (producer statement – construction review) for the superstructure from the design engineer
	10 May 2018	Authority writes an RFI[16], asking for: • assurance from the peer reviewer that it stands by the design change (the modified brace), which is not on the consented plans • the design engineer to include its instruction for this change in documents supporting the application, which would infer that it is covered by the PS4
The modified brace and response to design concerns	28 May 2018	Authority replies to the design engineer's response of 21 March 2018 (re the design concerns) and asks for: • acknowledgement and confirmation of the design engineer's response from the peer reviewer, and verification of the currency of the peer reviewer's PS2 • advice of any other departures from the consented plans aside from the modified brace, and also notes that no variation or amendment to the consent has been applied for
	25 Jun 2018	Peer reviewer contacts the authority, saying: • it had advised the owner and design engineer its PS2 did not cover the building as constructed: it did not accept the design changes were minor; qualifications in its original PS2 (which it said were outstanding issues that were raised and agreed to be closed out) had not been addressed; and there was a fundamental change to one EBF brace (i.e. the modified brace) • however, it was prepared to do a revised peer review based on the original design, the modified brace, and assessment of the qualifications in its original PS2, and it would do this in the context of the independent engineers' concerns

[15] New Zealand Standard NZS 3101 Part 1:2006 Concrete Structures Standard
[16] I note that the agent says this RFI was not received.

Event	Date	Details
Application for a minor variation	25 Jul 2018	Design engineer applies for a minor variation[17] to the consent for the modified brace; provides a covering letter; includes amended drawings, shop drawing and associated calculations
	7 Aug 2018	Design engineer supplies revised PS1 referencing the amended drawings to support this application (at the authority's request)
	10 Aug 2018	Authority advises the proposed minor variation will now be processed as an amendment to the consent "due to the complexity of the additional structural works"; asks the design engineer to submit an application form for this plus comment and an updated PS2 from the peer reviewer
Authority's design review	12 Aug 2018	A separate engineering firm engaged by the authority provides a high-level review of the design concerns and of the design engineer's response (refer paragraph 3.4)
Subsequent emails	17 Sep – 30 Oct 2018	Ongoing correspondence between the owner/agent and the authority questioning the delays in issuing the code compliance certificate and asking what the authority considers outstanding. Other discussion includes: • authority says an RFI was sent on 10 May 2018 and it is not aware of any response to this or application for an amendment • owner/agent disputes the RFI letter was sent; expresses concern that authority's responses have not referred to the "significant issues" that had arisen with the original plans, saying the authority had previously given this as the reason for not issuing the code compliance certificate • owner/agent considers the modified brace is a minor change, saying the building's bracing system is still fundamentally the same, it does not pose any other design concerns, and it is included in the design engineer's construction monitoring records and PS4
Application for determination	12 Nov 2018	The authority applies for a determination – this is accepted on 3 December 2018.

3.2 The independent engineers' concerns (13 December 2017)

3.2.1 On 13 December 2017 the independent engineers wrote to the authority with various concerns about the building's structure. They said that, from their observations of the building during construction and subsequent checks against the authority's property files, it appeared the following details "would not be able to withstand the capacity derived seismic actions and maintain building stability":

- a column splice on the east face, which the independent engineers considered was insufficient for the size of the column – in their view this posed an unintended structural weakness whereby the column splice would fail before the active link which was a potentially dangerous failure mechanism
- the modified brace on the ground level, which was required to support half of the building's base shear demand and which independent engineers said was "highly eccentric" – the independent engineers considered that this brace would buckle well before the active link and was also a potentially dangerous failure mechanism.

[17] A minor variation is defined in Regulation 3 of the Building (Minor Variations) Regulations 2009 as "a minor modification, addition, or variation to a building consent that does not deviate significantly from the plans and specifications to which the building consent relates".

3.2.2 The independent engineers listed another 11 items they considered "design defects" (when the structure was considered as a ductile frame) and which they said they had identified after further investigation, carrying out some calculations, and creating a simple PSA model. In their view, these defects were as follows (grouped, and in summary – refer to Appendix B for more details):

- the building was highly torsional and the seismic coefficients used in design appeared incorrect – the independent engineers considered the building's design underestimated seismic loads in the order of 25%
- the column on Grid 5-H was overloaded by about five times; and some of the connections on Grids 5 and 6 appeared insufficient to transfer load into the frames
- column demands appeared to be about ten times the maximum compression pile capacities; piles were offset from the EBF locations, raising concerns about raft punching shears; and the building's total overturning moment was about 110,000 kNm[18] but total restoring moment was only 37,000 kNm
- the EBF column hold down bolts were insufficient for shear and axial demands; all EBF active link connections into the columns' minor axis were weaker than the active link that was meant to yield; and the overstrength capacity of the active links did not allow for the concrete flooring cast with shear studs attaching it to these links
- EBF links against the stairwell did not appear to be restrained; and the precast stairs had no bursting/confinement at the top landing.

3.2.3 The independent engineers said their observations raised wider concerns about the building design, which they had not reviewed in detail. They had discussed their concerns with the design engineer but did not consider the response was adequate.

3.3 The design engineer's response (21 March 2018)

3.3.1 On 21 March 2018 the design engineer wrote to the authority regarding the independent engineers' concerns. The design engineer said they had discussed their responses with the peer reviewer and provided comments on each item raised by the independent engineers (refer Appendix B), a record of the peer review and associated calculations.

3.3.2 The design engineer said they considered the building's structural design was in accordance with New Zealand Standards and complied with the Building Code, and that the building's seismic design had been investigated at an early stage of the design process.

3.3.3 The design engineer assumed the independent engineers' concerns arose from those engineers taking an elastic approach based on nominal ductility of $\mu = 1.25$ (whereas the design engineer considered the building was designed for limited ductility of $\mu = 3$). The design engineer said this would lead to a big difference in design loads, and referred to the need to undertake capacity design. The design engineer also considered the EBFs were appropriately distributed and proportioned for the building given its relatively small floor plate.

[18] Kilonewton metre

3.4 The authority's design review (12 August 2018)

3.4.1 On 12 August 2018 a separate engineering firm engaged by the authority ("the consultants") wrote to the authority summarising their "high level review" of the independent engineers' concerns. The consultants said they had looked at the independent engineers' letter of 13 December 2017, the design engineer's response of 21 March 2018, and the consented drawings[19].

3.4.2 The consultants said they believed some of the independent engineers' concerns were valid. They gave three examples of items they considered did not comply with Clause B1:

- some EBF connections did not comply with NZS 3404 clause 12.11.3.7[20]
- columns and foundations were not designed in accordance with NZS 3404 clause 12.8.4 Concurrent actions on columns
- column hold-down details BP3 and BP6 were unable to resist the required earthquake loads as detailed in NZS 1170.5 and NZS 3404.

3.4.3 The consultants said other aspects of the building's structural design, including items not raised by the independent engineers, might not comply with the Building Code and in their opinion the building's structural design required a detailed review.

4. Initial submissions

4.1 The authority

4.1.1 The authority sent an application for determination on 12 November 2018, which was followed by a submission and background information on 28 November 2018. The authority's submission included the following:

- The authority said it had received credible information that gave rise to concerns about the building's structural compliance with Clause B1. The consultants' high-level review (refer paragraph 3.4) had confirmed there were design issues that should be considered further.
- The authority considered it had reasonable grounds to issue the various stages of building consent with regard to structure and noted that the building's structural design was carried out by a Chartered Professional Engineer, reviewed by two Chartered Professional Engineers employed by the peer reviewer, and received a regulatory review on the authority's behalf by another Chartered Professional Engineer.
- The design engineer had applied for a minor variation regarding the modified brace. However, the authority considered a formal amendment application would be more appropriate, even though this work had already been completed, given the level of assessment required. The authority had earlier advised that a minor variation or amendment would need to be applied for, and granted, before it could consider granting a code compliance certificate.
- If building work had been carried out in accordance with the consented documents the authority would be obliged to issue the code compliance certificate, notwithstanding any Building Code compliance concerns.

[19] Structural drawings by the design engineer stamped BCN/2016/2849 Amendment 1 (34 pages)
[20] Refer to Appendix A4.

4.1.2 With its submission the authority provided copies of information including its property files relating to the Stage 1 and Stage 2 building consents and relevant correspondence.

4.1.3 On 5 February 2019 the authority supplied copies of other relevant correspondence after searching its email files. This included the design engineer's response of 21 March 2018, which the agent had noticed was not included in the material that had been provided previously by the authority (refer paragraph 4.2.3).

4.2 The owner

4.2.1 On 13 December 2018 the owner's agent sent a submission, details of key events and copies of correspondence with the authority (by the design engineer on 21 March 2018, the owner on 17 September 2018, and the agent on 5 and 30 October 2018).

4.2.2 The agent said the authority's summary did not provide a full or accurate picture of events. The submission included the following points:

- The owner understood that the authority did not forward the design engineer's response and calculations of 21 March 2018 to the independent engineers or to its consultants. The design engineer understood they had satisfied the authority's concerns with this response as no formal response had been received. The lack of response had denied the design engineer the chance for an early review of its work and had prejudiced the owner.
- The authority had not given a formal reason for refusing to make a decision regarding the code compliance certificate. The owner was concerned that requiring an amendment to the consent for the modified brace rather than a minor variation (as the authority had initially suggested) had been done to delay the process. Further, the change made to the modified brace was necessary for fire safety reasons and the authority was aware of this.
- The owner accepted that the plans were reviewed to ensure the building's structural safety, but was critical of the process and time taken by the authority. The owner also considered the authority had refused to acknowledge the structural concerns with the building and, if the authority had wished to refuse on the grounds the building "was consented otherwise than in accordance with the Building Code" it should have done so in February 2018, not delayed until November 2018 when it applied for a determination.

4.2.3 On 31 January 2019 the agent sent copies of relevant information that had apparently been omitted from the authority's submission. This included emails and associated documents the design engineer sent to the authority on 21 March 2018 regarding the independent engineers' concerns and on 25 July 2018 with the application for a minor variation.

4.2.4 The agent said in an accompanying email that the authority was not correct to state it could not issue a minor variation regarding the modified brace because the peer reviewer did not provide confirmation. The agent said the authority had not asked for this, but had advised it would now only accept this change as an amendment. However, in the owner's view, this change should still have been covered by a minor variation.

4.2.5 On 20 March 2019 the agent provided shop drawings for the building structure. This was in response to the Ministry's request for the parties to confirm that the structural framing was constructed as described in the consented plans or, if not, to provide the shop drawings detailing the frame's manufacture.

5. The expert's reports

5.1 General

5.1.1 As described in paragraph 1.8 the Ministry engaged a firm of consulting engineers with specialist expertise in structural engineering ("the expert"). The personnel used are both Chartered Professional Engineers. The expert was engaged to:

- provide their opinion on the specific concerns raised by the independent engineers
- advise of any additional concerns they identified regarding compliance with Clause B1 of the building's steel framing or foundations, noting that they were not expected to do a full review of the structural design.

5.1.2 The expert was provided with copies of:

- the building files and other information submitted by the parties, and
- on 20 March 2019, the set of shop drawings referred to in paragraph 0 as an available method to confirm the as-built details of the building structure.

5.1.3 The expert provided a report dated 12 April 2019 ("the expert's first report") on 16 April 2019 and I copied this to the parties. Responses to this report are summarised in paragraph 5.3 below.

5.1.4 On 19 June 2019 I convened a technical meeting to discuss aspects of the expert's first report (refer to paragraph 5.4). Following this meeting I received further information from the parties: photographs of the stairwell (from the agent); the information from the minor variation application; and a conference paper[21] which the design engineer said would help the expert understand how the structural system had been developed (from the design engineer).

5.1.5 I sent this information to the expert on 11 July 2019, instructing them to revise the report to take account of this and of any matters arising from the technical meeting that should be addressed.

5.1.6 The expert sent their final report dated 8 August 2019 ("the expert's final report") on 14 August 2019. I sent the expert's final report to the parties the next day and suggested they include any comments on this report with their responses to the draft of the determination.

5.1.7 I subsequently engaged the expert to respond to comments received on their final report. The expert's response is described in paragraph 6.3.

[21] Seismic design for steel framed building in Earthquake recovery area, Christchurch", a conference paper authored by the design engineer for the Australasian Structural Engineering Conference in Adelaide, Australia, September 2018

5.2 Key findings from the expert's final report

5.2.1 The expert's final report considered the independent engineers' 13 concerns, being (refer to Appendix B for further detail):

1. Column splice capacity
2. Modified brace (ground level, Grid 6)
3. Calculation of seismic loads, torsional stability
4. EBF column hold down bolts
5. Column on Grid 5-H
6. Pile capacity
7. Foundation uplift by overturning
8. Raft foundation (punching shear)
9. Load path into EBFs on Grids 5 and 6
10. Precast stairs at landing
11. EBF active link connections to minor axis of column
12. EBF links against the stairwell
13. Composite action with concrete flooring.

5.2.2 In summary the expert found that:

- Two items were in accordance with Verification Method B1/VM1 for Clause B1 – these were the load path into the EBFs on Grids 5 and 6 (item 9), and allowance for composite action between the floor slab and steel beams (item 13).
- The rest were not in accordance with B1/VM1. These related to: the column splice detail, the modified brace, calculation of seismic loads, the EBF column hold down bolts, the column on Grid 5-H, pile design, raft foundation (shear capacity), stair detailing at the landing and half landing levels, EBF active link connections to the minor axis of the columns, and the EBF against the stairwell.
- Regarding the foundation design with respect to building overturning stability, the expert concluded that the design engineer had not provided enough information to demonstrate that this was in accordance with B1/VM1.

5.2.3 The expert identified an additional item they considered was not in accordance with B1/VM1. This was the EBF beam/brace bolted connection on Grid 5-H at the opposite end to the active link on Grid H (at Grid 5-E), which they considered had insufficient capacity to transfer the brace load to the supporting column.

5.2.4 There was one substantive change from the expert's first report. That report concluded the stair detailing was in accordance with B1/VM1, but the expert revised their view on this item after receiving further information at and after the technical meeting.

5.2.5 In assessing the independent engineers' concerns, the expert said the building was torsional but at this stage they had not reviewed the impact of "the potentially large resultant displacements due to torsion [arising] at the building extremities". The expert considered further investigation was required to evaluate this displacement given the close proximity of neighbouring buildings.

5.2.6 The expert also described their assessment approach, which included reviewing:

- the consented drawings to identify any potential issues, and the shop drawings to confirm that the frame layout and connection details were constructed in accordance with the consented drawings
- each item raised by the independent engineers, carrying out calculations for key elements to the extent required to establish compliance or otherwise
- the independent engineers' simplified PSA model to the point that the expert was satisfied with the model's inputs; the design engineer's response to the authority of 21 March 2018; and the design engineer's and independent engineers' calculations to identify potential differences in approach.

5.3 Responses to the expert's first report

5.3.1 Responses to the expert's first report of 12 April 2019 included the following:

- the agent – acknowledged the expert's first report, asked who determined if there were any options to fix the building, and said the owner would prefer the Ministry to determine this aspect as part of its determination (17 April 2019)
- the authority – said it had reviewed the report and, on the information available, agreed with its findings (30 April 2019)
- the independent engineers – said they were in agreement with the report and considered the expert had an accurate understanding of the technical issues they had raised (24 May 2019)
- the design engineer made no response to the expert's first report.

5.3.2 The peer reviewer sent various responses to the expert's report as outlined below:

- a letter dated 3 May 2019 requesting the expert's calculations, the basis for the expert's conclusions and "other inputs" used to reach its conclusions, and asking for a meeting with the expert
- a letter dated 15 May 2019 in response to the expert's report (refer Appendix C1 for more details). Comments in this letter included:
    - o the statement that the scope of work for the peer review was limited to the superstructure only
    - o a query why the expert appeared to accept the independent engineers' PSA model but did not appear to have substantively reviewed the design engineer's PSA model
    - o responses to the expert's conclusions including that two of the items (the EBF active link connections, and the beam/brace bolted connection to the column minor axis) "were identified by [the peer reviewer] as conditions to the PS2", which needed to be acknowledged
- an email on 28 May 2019 providing copies of two PS2s issued in relation to the building superstructure (dated 19 January 2017 and 23 February 2018) and associated documentation. The peer reviewer said the "revised" PS2

incorporated architectural changes to the design and that both PS2s should be read together.

5.3.3 The correspondence referred to in paragraph 5.3.2 also included replies to specific questions from the Ministry to the peer reviewer in relation to:

- the reasons the peer reviewer considered the building work described in "the PS2" (*sic*) complied with the Building Code (Ministry email dated 7 May 2019)
- which PS2 was being referred to in the letter of 15 May 2019 as containing qualifications (Ministry email dated 17 May 2019).

5.3.4 In an email dated 7 June 2019 the Ministry advised that the "other inputs" used by the expert (refer paragraph 5.3.2, 1st bullet point) to reach their conclusions included "the extensive material provided with the application itself, the relevant standards and MBIE compliance documents, plus engineering knowledge and judgement".

5.4 The technical meeting

5.4.1 On 19 June 2019 I convened a technical meeting in Christchurch to discuss matters raised in the expert's first report. This meeting was held chiefly in response to the request by the peer reviewer referred to in paragraph 5.3.2. The meeting was attended by:

- myself accompanied by four advisors
- the two personnel responsible for the expert's report
- two officers of the authority
- the agent and an engineer engaged by the owner ("the owner's engineer")
- the design engineer
- four representatives of the peer reviewer.

5.4.2 Technical points of difference which emerged at the meeting mostly related to the magnitude of the tension and compression loads in the building's columns – and hence in the foundations – and to connection details, where there were different views about how the capacity design requirements had been met (or not met) in the building's design.

5.4.3 The expert also explained their assessment approach, while noting that they were asked to look at very specific design areas. The expert said they took a capacity design approach as the Design Features Report said the building was "limited ductile". Their first question while carrying out their assessment was whether the building could sustain a ductility of 3. The expert concluded that, with the detailing provided it could not, and accordingly they based their further evaluation on a ductility factor of 1.25.

5.4.4 Appendix C2 contains more details of the meeting's discussion.

6. The draft determination and further submissions

6.1 A draft of the determination was issued to the parties for comment on 28 August 2019.

6.2 Submissions on the draft determination and on the expert's final report

6.2.1 The following responses were received regarding the draft determination (issued 28 August 2019) and the expert's final report (issued 8 August 2019):

- The owner's agent (5 September 2019) – made no comment apart from correcting the description of material underlying the foundations.
- The authority (9 September 2019) – accepted the draft; said the authority had "reasonably relied on the opinion of appropriately qualified and experienced professionals in granting the building consent"; and said it had continued to maintain this position by accepting the draft "as it is based on the opinions of a further group of appropriately qualified and experienced professionals, and those opinions have been thoroughly discussed".
- The design engineer (9 September 2019) – disagreed with the draft's conclusions based on the expert's report; said the expert had ignored the design engineer's original concept and calculations and had instead reviewed the building as a nominally ductile structure; and considered that capacity design had been carried out. The design engineer also commented on specific findings in the expert's final report (refer to Appendix D for details).
- The independent engineers (9 September 2019) – said the draft accurately represented their design concerns, and said they deferred to the expert's view that two of the items they had identified were in accordance with B1/VM1. The independent engineers also considered further investigation was needed regarding the foundations and building stability under seismic actions.
- The peer reviewer advised it had no further comment to make (12 September 2019).

6.3 The expert's response

6.3.1 On my request the expert reviewed the design engineer's submission of 9 September 2019. The expert provided a response ("the expert's response"), dated 21 October 2019 which I copied to the parties on 22 October 2019. I also advised that the determination would now be finalised unless the parties requested another draft. No request for a further draft determination was made.

6.3.2 The expert's response included the following (refer to Appendix D for details):

- The expert confirmed it had reviewed the Design Features Report, structural drawings and calculations submitted for building consent, and also the requirements of HERA publication P4001:2013 'Seismic Design of Eccentrically Braced Frames (EBFs)' ("the HERA publication") as the design procedure in this publication was identified as a means of compliance[22].
- The expert continued to disagree with the design engineer's assertion that a limited ductile design had been achieved. The expert considered there were "obvious areas" where the requirements from the relevant Standards and

[22] The relevant HERA publications are discussed in paragraph 7.1.5 of this determination.

guidelines for detailing active links and capacity design of other elements had not been complied with.

- The expert also considered there were "significant departures" from the procedure in the HERA publication for design and detailing as a limited ductile structure, and considered there were a number of missing or incorrect calculations plus other issues in the calculation package submitted for building consent.
- The expert concluded that the building was not in accordance with B1/VM1 either as a limited ductile or nominally ductile design.

6.3.3 The expert provided two representative examples to support these views:

- **EBF hold downs/baseplate design connections**[23]: for comparison purposes the expert assumed there was adequate detailing for a limited ductility structure (as the design engineer had asserted). The expert calculated column loads based on the procedure in the HERA publication, then used these to assess the baseplate capacity. The expert said the design engineer had designed the baseplate on Grid 5-E[24] (refer to Figure 2b) for a tensile load of 1,929kN but this was not consistent with column loads in the calculation package issued for building consent, which indicated a column load of 8,196kN at Level 1 (being the "column away from link"). The expert also identified what it considered were a number of errors in the baseplate design connection calculations submitted for building consent.
- **Active links in weak axis connection**[25]: the expert said the HERA publication procedure required the bolted cleat to the column to have sufficient shear capacity to resist the overstrength shear demand imposed by the yielding active link in the beam. The expert's calculations demonstrated that the design engineer's design did not achieve this, as the expert concluded the cleat shear capacity (766kN) was less than the beam shear capacity (987kN) and significantly less than the beam overstrength shear capacity (1,530kN).

6.4 The design engineer's further submission

6.4.1 The design engineer replied on 28 October 2019, reiterated concerns expressed previously about the expert's approach to reviewing the building design, and also said they considered the expert's response had not been conducted appropriately for reasons including the "simple comparison" of building details with those in the HERA publication.

6.4.2 The design engineer said the ductile capacity of the active links should have been checked with actual loads transmitted through to the members and connections, and that the seismic loads were relatively low due to the building's small footprint. With regard to the pile capacities, the design engineer said documents were available concerning fill placement and compaction.

6.4.3 The design engineer also said their comments were of a general nature as there had not been enough time to prepare further information. The design engineer anticipated that most issues raised in the expert's response would be resolved once this

[23] Item 4 of the independent engineers' concerns, (as numbered at paragraph 5.2.1)

[24] The expert' report contains some minor typographical errors in respect of the baseplate at this grid reference and the corrected values and terms are used above. The corrected grid reference is Grid 5-E (in place of Grid 5), the corrected load values are 1,929kN and 8,196kN (in place of 2,000kN and 8,198kN respectively), and the additional column description is the "column away from link".

[25] Item 11 of the independent engineers' concerns, (as numbered at paragraph 5.2.1)

information was provided. The Ministry replied on 31 October 2019 asking if the design engineer intended to provide another submission and if so when this would be received: there was no response to this enquiry.

7. Discussion

7.1 Compliance with Clause B1 via Verification Method B1/VM1

7.1.1 The matter to be determined is whether the building as designed and constructed complies with Clause B1 with respect to the independent engineers' concerns.

7.1.2 One way to demonstrate compliance with Clause B1 is by following a relevant Acceptable Solution or Verification Method[26], and section 19 of the Act says a building consent authority must accept either document as a means of establishing compliance with the Building Code. Verification Method B1/VM1 is generally used for buildings requiring specific design.

7.1.3 For this particular building, B1/VM1 comprises the AS/NZS 1170 loadings in conjunction with the relevant material standards, which include NZS 3404 for steel and NZS 3101 for concrete, as modified by the citation for this Verification Method.

7.1.4 The design engineer identified B1/VM1 (and also B1/VM4 Foundations, which primarily concerns soil capacity available within the building foundations) as the means of compliance with Clause B1 in the PS1 dated 19 December 2016 and submitted for Amendment 1 to the Stage 2 building consent. A subsequent PS1 (dated 23 February 2018 and submitted for Amendment 2) lists AS/NZS 1170, NZS 3101 and NZS 3404 – all of which are cited by B1/VM1 – as the "approved documents" for compliance with Clause B1.

7.1.5 I note that the peer reviewer's PS2s of 19 January 2017 and 23 February 2018 both list NZS 1170 and NZS 3404 as the means of compliance, as well as HERA Report R4-76 which is referenced as the basis for an alternative solution[27] proposal for the EBF design. It appears from section 1.4 of the Design Features Report that Corrigendum 1:2013 of R4-76 (HERA Publication P4001:2013)[28] was used, but this is not specifically stated.

7.1.6 The expert has concluded that in the case of two of the items identified by the independent engineers (items 9 and 13, refer paragraph 5.2.2), compliance with Clause B1 can be established through B1/VM1.

7.1.7 However, it is the expert's view that the building structure has not been designed or constructed in accordance with B1/VM1 with respect to almost all the items identified by the independent engineers (refer paragraph 5.2.1); i.e. the selected building elements the expert was asked to review. These include the modified brace, which has not been approved by the authority either as a minor variation or as an amendment to the consented design.

7.1.8 I accept the expert's findings in this regard and, as no other comprehensive justification for means of compliance with Clause B1 has been provided, I now turn to consider whether the structural design could comply with this clause directly; i.e. as an alternative solution.

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[26] Under section 22 of the Act

[27] An alternative solution is all or part of a building design that demonstrates compliance with the Building Code, but differs completely or partially from the Acceptable Solutions or Verification Methods.

[28] New Zealand Heavy Engineering Research Association (HERA) P4001 - Seismic design of eccentrically braced frames - 2013

7.2 Compliance with Clause B1 directly

7.2.1 Clause B1 requires buildings to withstand the combination of loads they are likely to experience during construction or alteration and throughout their lives. The clause's performance requirements include:

- "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" (Clause B1.3.1)
- "Account shall be taken of all physical conditions likely to affect the stability of buildings, building elements and sitework" (Clause B1.3.3).

7.2.2 These conditions include self-weight, imposed gravity loads arising from use, adverse effects due to insufficient separation from other buildings, and a range of natural events such as wind and earthquake.

7.2.3 The loadings and load combinations to be used in the design of buildings are presented in the AS/NZS 1170 suite of standards. The associated materials standards have been developed to provide buildings whose resistance to the loadings specified in AS/NZS 1170 exceed the prescribed demands by a margin that is deemed sufficient in statistically reliable terms, in accordance with the philosophy and principles of ISO 2394:1998 General principles on reliability for structures.

7.2.4 The design engineer considers the building is designed as a limited ductile structure with earthquake energy dissipated through the EBFs' active links. However, the expert considers this is not the case because the building structure (on account of its design and detailing) is only capable of a nominally ductile response when evaluated in terms of accepted design methodologies (such as those set out in HERA publication P4001:2013, and which give effect to AS/NZS 1170 and NZS 3404 for the design and detailing of EBFs); and further that various errors have been made in its design. The expert also considered that there were significant departures from the procedure set out in the HERA publication, even though this was referred to (in the PS2s) as an alternative solution proposal for compliance with Clause B1.

7.2.5 Drawing on the expert's findings and analysis, I make the following observations and conclusions:

- Due to the lateral resistance being concentrated about the central core area, the building is torsionally sensitive, and hence classified as irregular. An incorrect combination of modes was used from the original analysis using the PSA model, resulting in the design lateral forces being too low.
- The building is not capable of a ductile response given various design aspects including: detailing of the EBF active links into several columns; as the columns at Grids 5-H and 6-H are unable to support the overstrength actions of the active links from both directions; and as the EBF column splices generally are inadequate.
- As its overturning resistance is inadequate for either the actions from the active links at overstrength or the higher forces from a nominally ductile response, the building is also likely to be unstable at design level earthquake loading for a number of reasons. These include: insufficient capacity of the EBF column hold down details (base plates and bolts); as the raft slab has insufficient shear strength to transfer overstrength actions into the piles; and as the pile capacities cannot sustain the column forces in compression under overstrength actions.

- As a result, there is inadequate strength of some members and of the overall structure at ultimate limit state design loads, leading to possible rupture or instability.

7.2.6 Accordingly, I do not consider that the parts of the building's structure originally identified by the independent engineers can be shown by alternative methods to comply with the requirements of Clause B1. This is because Clause B1.3.1 in particular, which requires the building to have "a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing", has not been satisfied.

7.2.7 I note the expert concluded that two of the items identified by the independent engineers were in accordance with B1/VM1. However, given the significant issues with the building structure generally, and noting that the expert has identified an additional concern even though they did not undertake a comprehensive structural review, I do not consider that the compliance of these two items can be regarded as sufficient to offset the effects of other non-compliances or lead me to a conclusion that the building complies with Clause B1.

7.3 Further comments on EBF design

7.3.1 The design engineer has expressed the view that the expert did not assess the building in terms of its intended design as a limited ductile structure. While I consider that the expert has made it clear this was not the case, I provide the following overview for the benefit of parties.

7.3.2 In general terms, EBFs may be designed as ductile elements (e.g. $\mu = 3$ for limited ductile elements) or as nominally ductile elements ($\mu = 1.25$).

7.3.3 If EBFs are designed as limited ductile elements, to ensure that flexural yielding can only occur in specifically designated locations:

- there must be designated active links that are proportioned and detailed in accordance with prescribed requirements, and
- columns and foundations must be designed to resist the overstrength seismic loads from these active links.

7.3.4 If EBFs are designed as nominally ductile elements, all those elements must be designed for the significantly higher lateral loads that are associated with the structure's lower overall ductility.

7.3.5 This means, therefore, if an EBF does not meet the detailing and overstrength requirements of a limited ductile element it must be designed for the higher loads associated with a nominally ductile structure.

7.3.6 In the case of the building, the expert's review established a number of locations in selected EBFs that did not meet the design and detailing requirements associated with the limited ductile design intended by the design engineer.

7.3.7 The expert then evaluated these EBFs against the requirements for nominally ductile elements. However, they found that the increased strength requirements for such elements could not be met.

7.3.8 This in turn led the expert to conclude that key items such as column splices, column hold down bolts and baseplates, and the concrete foundations did not meet the requirements of either limited ductile or nominally ductile elements.

7.4 Summary

7.4.1 For the reasons outlined in this determination, I conclude that the building as designed and constructed does not comply with Clause B1 with respect to 10 out of 13 of the independent engineers' concerns. The building complies with Clause B1 with respect to items 9 and 13 raised by the expert, by way of B1/VM1. I have insufficient information to determine whether the building complies with Clause B1 with respect to item 7 raised by the expert (foundation uplift by overturning).

7.4.2 I emphasise that this determination is focused on the specific concerns of the independent engineers, and has not involved a comprehensive review of the building's foundations and superstructure. I also draw the parties' attention to the expert's comments at paragraph 0 regarding the need for further investigation to evaluate the potential for displacement due to torsion arising at the building's extremities, given the close proximity of neighbouring buildings.

8. The decision

8.1 In accordance with section 188 of the Building Act 2004 I hereby determine that the building as designed and constructed does not comply with Clause B1 with respect to the independent engineers' concerns, as expressed in their letter to the authority of 13 December 2017. Of the 13 items raised; ten items are not compliant, two items are compliant, and I have insufficient information to establish compliance with respect to one item.

Signed for and on behalf of the Chief Executive of the Ministry of Business, Innovation and Employment on 4 December 2019.

Katie Gordon
Manager Determinations

Appendix A: Extracts from the legislation, the Verification Method and relevant standards

A1 New Zealand Building Code (Schedule 1 to the Building Regulations, 1992)

Clause B1 Structure

Objective

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.

…

B1.3.3 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.

A2 Verification Method B1/VM1

1.0 General

1.0.1 The Standards cited in this Verification Method provide a means for the design of structures to meet the performance requirements of New Zealand Building Code Clause B1 Structure. For any particular building or building design, the Verification Method shall consist of AS/NZS 1170 used in conjunction with the relevant cited material standards as modified by this Verification Method.

1.0.2 Modifications to the Standards, necessary for compliance with the New Zealand Building Code, are given against the relevant clause number of each Standard.

1.0.3 Citation of Standards in this Verification Method is subject to the following conditions.

…

(d) Where AS/NZS 1170 is used in combination with other Standards cited in this Verification Method and there are incompatibilities with these other Standards, then the underlying philosophy, general approach, currency of information and methods of AS/NZS 1170 are to take precedence.

…

2.0 Structural Design Actions Standards

2.1 The requirements of the AS/NZS 1170 suite of Standards are to be complied with. These comprise:

AS/NZS 1170.0: 2002 including Amendments 1, 2, 3, 4 and 5
AS/NZS 1170.1: 2002 including Amendments 1 and 2
AS/NZS 1170.2: 2011 including Amendments 1, 2, 3, 4 and 5
AS/NZS 1170.3: 2003 including Amendment 1,
and NZS 1170.5: 2004.

2.2 The requirements of AS/NZS 1170 are subject to the following modifications.

2.2.1 Material Standards Where AS/NZS 1170 calls for the use of appropriate material Standards, only those material Standards referenced in this Verification Method B1/VM1 are included. Use of other Standards with AS/NZS 1170 must be treated as an alternative means of verification.

…

3.0 Concrete

3.1 NZS 3101: Part 1 subject to the following modifications:

…

5.0 Steel

5.1 NZS 3404: Part 1 subject to the following modifications:

…

A3 Australian/New Zealand Standard AS/NZS 1170.0:2002 (Amendment Nos 1, 2, 3, 4 and 5)

The Preface to this standard includes:

> This Standard is based on the philosophy and principles set out in ISO 2394:1998, *General principles on reliability for structures*. ISO 2394 is written specifically as a guide for the preparation of national Standards covering the design of structures. It includes methods for establishing and calibrating reliability based limit states design Standards.

A4 New Zealand Standard NZS 3404: Part 1: 1997

12.11 Design of eccentrically braced framed seismic-resisting systems

12.11.3 Design requirements for EBF frames and components

12.11.3.7

The following requirements shall apply:

(a) No part of the brace to beam connection shall extend into the web area of an active link;

(b) The minimum clear length of active link, e_{min}, shall be not less than the beam depth, d_b.

Appendix B: The independent engineers' concerns, design engineer's response, and expert's conclusions

The following table summarises the independent engineers' concerns, the design engineer's response, and the expert's conclusions in their final report (which included an additional item identified by the expert). Item numbering matches that used in paragraph 5.2.1.

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
1. Column splice capacity	• column splices not sufficient for size of columns • this poses an unintended structural weakness as splice would fail before the active link (a potentially dangerous failure mechanism)	• columns connected by bolts, all connections designed with Steel Construction New Zealand's steel connection guide, in accordance with NZS 3404 • provided calculations for critical splices at Grids 5 and H on the third level; said combined action of tension and bending moment applied to each splice and is satisfied	**NOT in accordance with B1/VM1** • reviewed a column splice on Grid 5-H at Level 2; calculated tensile demand on the column as approx. 8.0 MN[29]; said NZS 3404 requires the minimum tensile load that a column splice in a limited ductile frame is to be designed for is 50% of capacity of the smaller member (about 5.2 MN in this case) • calculated the combined tensile capacity of the splice joint's web and flange plates as 5.1 MN and the total shear capacity of its bolts as approx. 5.5 MN • had not considered moments and shear forces transferred though the splice at this stage, but said the combined actions would reduce the tensile capacity of the spliced joint
2. Modified brace (ground level, Grid 6)	• brace is highly eccentric and required to support 50% of the building's base shear demand – it would buckle well before the active link (a potentially dangerous failure mechanism)	• load transferred through the stiffened brace at the top, where a bending moment is applied as additional force and combined with the axial force • brace designed to support this combined action; additional stiffener is provided at the base	**NOT in accordance with B1/VM1** • said beam and brace have sufficient capacity for the revised layout, but load path changed so moment in the active link zone is considerably higher • this results in higher moments in the column and beam/column connection, which do not appear to have been checked in the frame redesign • based on the expert's calculations the supporting column on Grid 6-H and the beam/column connection

[29] Meganewton

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
			do not have sufficient capacity to support the resultant forces
3. Calculation of seismic loads, torsional stability	• their PSA model shows the building is highly torsional • appears that incorrect seismic coefficients used in design (the period for the first and second modes used to determine seismic coefficients instead of the second and third modes), so consider seismic loads underestimated in design in the order of 25% • design engineer did not appear to consider torsional stability requirements of NZS 1170.5 clause 4.5.2.4	• considers structure has design ductility of 3 with four bracing lines in transverse direction and stiff bracings in perpendicular direction • considered not to cause inelastic torsion; however, irregularity requirement in NZS 1170.5 clause 4.5 was considered in the design • two analyses (ESA[30] and MRSA[31]) carried out and had equal base shear, used scaling factor of 1 for seismic forces in the MRSA; considers building meets NZS 1170.5 clause 4.5.2.4 as sufficient resistance for both translation and torsion	**NOT in accordance with B1/VM1** • the building is torsional, with the first two modes being a combination of torsion and translation (both in the transverse direction). NZS 1170.5 allows the designer to use the higher period for these two mode shapes when calculating the base shears in the transverse direction. The third mode period would then be used for calculating the base shears in the longitudinal direction • based on the independent engineers' PSA model, this corresponds to a fundamental period of 0.84 seconds (transverse) and 0.4 seconds (longitudinal) • the design engineer has calculated the building base shears using fundamental periods of 0.84 seconds (transverse) and 0.56 seconds (longitudinal); and appears to have used the second mode period (in the transverse direction) for calculating the base shear in the longitudinal direction, this results in the design engineer's base shear being approx. 25% less than the required design level load in the longitudinal direction • the building is classified as irregular due to the torsional sensitivity. An MRSA can be carried out to determine the resultant frame actions but the scaling factor "k" (NZS 1170.5 clause 5.2.2.2) is to be a minimum of 1.0, with the base shear calculated using the modal analysis not being less than that calculated using ESA • the design engineer carried out modal analysis in accordance with NZS 1170.5 but the base shear calculated in the longitudinal direction is 25% less than required

30 Equivalent static analysis
31 Modal response spectrum analysis

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
			The expert also said: • they had not reviewed the impact of potentially large resultant displacements due to torsion at the building extremities, but said this required further investigation given the close proximity of neighbouring buildings • the independent engineers had referred to torsional stability requirements in NZS 1170.5 that were contained in 2017 amendments to the standard and not yet covered by B1/VM1
4. EBF column hold down bolts	• EBF column hold downs not sufficient for shear and axial demands	• EBF column hold downs designed for two major load cases, tension and compression (said shear combined with compression but ignored in tension due to minimal force) • uplift resisted by hold down bolts and the chemical bolts (described standards these were designed in accordance with); shear resisted by anchor bolts and the shear key under the baseplates • hold down bolts connected with EBF baseplates by bolt couplers in all but two cases, where larger bolts are used	**NOT in accordance with B1/VM1** • reviewed column baseplate on Grid 5-H; column supports combined directional actions as is common to braced frames in both the transverse and longitudinal directions, so the column actions should be calculated using the combined actions of the braced frames in both directions; the braced frame actions are based on the summation of overstrength capacity of the frame active link zones at each level • calculated total tensile load at baseplate level as approx. 14.5 MN but combined bolt tensile capacity as approx. 6.0 MN – i.e. significantly less than (about 40% of) the demand • baseplate hold-down capacity potentially reduced further by other factors: location of bolts not symmetric about the column centreline; baseplate had insufficient bending capacity to utilise the full tensile capacity of the outer bolts; and location of the brace on the baseplate was eccentric to the centre of the column

Item	Independent engineers’ concerns (letter 13 December 2017)	Design engineer’s response (letter 21 March 2018)	Expert’s conclusions (final report 8 August 2019)
5. Column on Grid 5-H	• Column on Grid 5-H roughly five times overloaded when considering overstrength shear forces of active links in both directions • braces also noded eccentrically into the columns (does not show up in PSA model analysis) and these additional loads must be accounted for in the column design	• structure designed for limited ductility so does not need checking for concurrency of the seismic force because of ductile behaviour occurring at the link • actions therefore considered to act separately along each of the two directions (NZS 1170.5 clause 5.3.1.1(a)); active links attached to columns in weak axis designed for inelastic deformation specified in NZS 3404 clause 12.11.3.3.3 • calculations showed columns in Grid H have less load than in Grid 5; the perimeter EBFs in Grids B1 and H relatively stiff for the small floor area so there was less demand • regarding the eccentrically noded braces, additional bending moment and shear to the columns was considered in the design	**NOT in accordance with B1/VM1** • this column’s compressive capacity is approx. 12.5 MN, but the expert calculated the axial compressive demand as approx. 14.5 MN (i.e. more than the capacity) • column’s capacity reduced further when consider combined actions of axial load and bending moments • the expert did not consider bending at this stage, but said moments were generated in both directions because of the transverse braced frame active link zone next to the column, and as the longitudinal braced frame’s brace connection was eccentric to the column/beam centrelines
6. Pile capacity	• drawings note maximum compression pile capacities as 1555kN but column demands appear about 10 times this • drawings do not indicate uplift capacity available	• extra force in an ultimate seismic event would be distributed through the mass concrete (and supporting by neighbouring piles) • this is a conservative approach ignoring the soil bearing capacity, which is sufficient to support this compression, and the design also considered an earthquake with liquefaction • piles provided to support compression only; no uplift occurs in this 1.1 m deep, heavy foundation (refer comments in next row)	**NOT in accordance with B1/VM1** • design engineer’s calculations indicate that pile capacities are exceeded in a number of locations • column reactions used in the design were not provided but, based on reactions used in the slab shear capacity check (refer comments on ‘Raft foundation’ below), it is unlikely that frame overstrength values were used so the number of piles exceeding their capacity is likely to be greater than the design engineer indicates • pile demands further exacerbated by potential rocking action (refer comments on ‘Foundation uplift’ below)

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
7. Foundation uplift by overturning	• building's total overturning moment demand approx. 110,000 kNm (@ μ = 1.25) but total restoring moment only 37,000 kNm in transverse direction • building foundations uplift, but implication on subgrade and building displacement not considered in design	• maximum overturning moment of about 120,000 kN-m conservatively based on ductility of 1; this is resisted by the foundation, which is considered strong enough to support the stress distributed from the uplift point load • reinforced concrete beam sufficient to support the negative moment at the lifting point and distribute stress over the foundation slab • re the uplift load, gravel compaction has the bearing pressure at the end of the slab; re the building displacement, the foundation is 1.1m deep and considered sufficient to resist lateral displacement by the soil passive pressure at the perimeter and lift pit	**Design calculations submitted do not demonstrate that the building foundations are in accordance with B1/VM1** • based on the overstrength capacity of the EBF active link zones, the frame overturning demands exceed total building restoring load in the transverse direction • calculations provided by the design engineer only appear to consider the design actions in one direction (the more favourable one) and do not address the resultant rocking action and subsequent effects on the piles, foundation and superstructure
8. Raft foundation (punching shear)	• piles offset from EBF locations, raising concerns about raft punching shears	• considers foundation and baseplates designed to be adequate for punching shear • baseplate 1,060mm x 1,000mm, which is large enough to distribute the shear to the concrete • calculations show no extra reinforcement required for the deep concrete slab, but there is HD16 reinforcing steel at 200mm centres along its edge	**NOT in accordance with B1/VM1** • the column on Grid 5-H has a base reaction of approx. 14.5 MN (refer 'Column on Grid 5-H' above) • the design engineer calculates the concrete shear capacity as approx. 8.6 MN but this assumes axial load transferred across the full baseplate area. Expert considers this is incorrect as the column and brace are offset from the centre of the plate by 250mm, and the plate has insufficient capacity to distribute the bearing loads to the plate's extremities • expert calculated the shear capacity as approx. 7.2 MN; i.e. significantly less than the demand

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
9. Load path into EBFs on Grids 5 and 6	• load path into EBFs on Grids 5 and 6 directly adjacent to large penetrations • connection between the drag members appears insufficient to transfer load into the frames	• lateral load transferred through the slab diaphragm action such as 100 mm thick reinforced concrete slab and shear studs • provided typical calculation for the shear studs; said all factors (e.g. P-delta[32], overstrength) were considered to determine the slab's elastic action • stress analysis showed maximum stress on slab adjacent to the two penetrations for the lift and stairs; the SE82[33] mesh was adequate to support the maximum tension	**IS in accordance with B1/VM1** • reviewed potential seismic load path at typical floor level; calculated the combined capacity of the slab and beam to transfer loads to frames on Grids 5 and 6 as approx. 410 kN and 510 kN respectively • said additional load can be transferred through the section of slab adjacent to the beams in the non-voided region, but impact of this not considered at this stage • used pseudo-ESA to determine the diaphragm forces – calculated maximum floor diaphragm load as approx. 800 kN in the transverse direction. The maximum load distribution between frames, taking account of torsion, was 50% (400 kN) and 65% (520 kN) to the frames on Grids 5 and 6 respectively
10. Precast stairs at landing	• precast stairs have no bursting/confinement at the top landing	• continuous rebars to be placed over top landing with 150° bending at the stair/landing corner • described layout of longitudinal and transverse bars, and provided shop drawings; considered no extra confinement necessary	**Stair detailing at landing and half landing levels NOT in accordance with B1/VM1** • shop drawings do not indicate bursting/confinement steel at corners of the stair landings but, while recommended, it is not a requirement • however, photos of as-built stairs provided after the technical meeting show the seismic movement joint at the half landing level was not installed as shown on the original structural drawings • this means the stairs are likely to experience larger forces than originally designed for as they are not isolated from the inter-storey seismic drifts, and are likely to exceed their design capacity before the building reaches its full design level lateral displacement

32 P-delta effects include gravity induced loads acting on a laterally displaced building.
33 A grade of ductile steel reinforcing mesh

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
11. EBF active link connections to minor axis of column	• all EBF active link connections into the columns' minor axis are weaker than the active link which is meant to yield	• EBFs with the columns' weak axis placed along Grid H, and part of stiff lateral resisting frames in the perimeter walls • connections adequate for these columns as force demands are much less than for frames in the transverse direction • inelastic requirements for connection to the weak axis also considered in the design	**NOT in accordance with B1/VM1** • the EBF active link connection consists of cleats welded to the column and bolted to the beam flanges and web; the bolts are located within the potential yielding region of the active link zone; there are no additional plates at the bolt locations to prevent plate shear failure • EBFs connected to the minor column axis are considered non-ductile so must be designed for the higher seismic load demands required for a structure limited to elastic response (μ = 1.0) • the expert reviewed the capacity of connection cleats to the Level 1 transverse beam at Grid 6-H; concluded these were likely to yield before the active link zone, so the EBFs could not achieve the assumed design ductility of μ = 3 and must be designed for the higher seismic load demands of an elastic responding structure (μ = 1.0) • expert calculated the web cleat shear capacity for the Level 1 beam at approx. 900 kN; shear demand based on overstrength capacity of the active link beam is approx. 1,400 kN, and shear demand for the elastic responding frame is approx. 2,500 kN • in both cases the shear capacity of the cleat is less than the shear demands
12. EBF links against the stairwell	• EBF links against the stairwell do not appear to be restrained	• checked the torsional deflections and appear to exceed the 4mm required • 100x50x6 RHS[34] bracings will be welded to the link and floor beam diagonally	**NOT in accordance with B1/VM1** • not apparent from the drawings how EBF restrained beside the stair core • the frame active link zone requires restraint to enable it to fully yield; apparent lack of restraint reduces the frame's allowable ductility to nominally ductile (μ=1.25)

34 Rolled hollow section

Item	Independent engineers' concerns (letter 13 December 2017)	Design engineer's response (letter 21 March 2018)	Expert's conclusions (final report 8 August 2019)
			but it was designed based on seismic demands for a structure capable of limited ductile behaviour (μ=3.0) • the EBF does not have sufficient capacity to resist the higher demands required
13. Composite action with concrete flooring	• overstrength capacity of the active links does not account for the concrete flooring cast with shear studs attaching it to these links, which causes higher overstrength demands to be taken into the structure	• composite action with concrete slab was not considered in the seismic design but designed as the non-composite active links with overstrength factor of 1.3 • links have a maximum three studs on the flange in 600 mm span, with high stress compared to the collecting beam; these studs are not sufficient to resist the concrete slab's compressive force so no composite action is assumed in accordance with NZS 3404 clause 13.4.6.1	**IS in accordance with B1/VM1** • as there are shear studs on the tops of beams within the active link zone, composite action (between the floor slab and steel beams) must be considered and the higher overstrength factor used in the design • from the calculations provided by the design engineer, appears that the higher overstrength factor has been used in the design • conclusion: the allowance for composite action has been considered and the overstrength factor used in the design complies with B1/VM1
Additional: Beam/brace bolted connection to column minor axis (Grid 5-E)	(N/A - additional item noted by the expert)	(N/A - additional item noted by the expert)	**NOT in accordance with B1/VM1** • just as the connection in the EBF active zone on Grid 6-H has inadequate capacity to transfer beam shear loads to the columns (refer 'EBF active link connections') the connection at the other end of the beam on Grid 5-H also has insufficient capacity to transfer the brace load to the supporting column

Appendix C: Responses to the expert's first report, and the technical meeting

C1 Peer reviewer's response 15 May 2019

The peer reviewer's response to the expert's first report is summarised below.
Other responses to this report are described in paragraph 5.3.2.

Peer reviewer's comments on the expert's first report of 12 April 2019 (letter 15 May 2019)	
General	The design engineer's letter to the authority on 21 March 2018 gave substantive responses that the expert should have considered. However, while the design engineer's letter suggested the responses were made with discussion and comment from the peer reviewer, the peer reviewer had not been party to the letter's content.
	The expert referred to the independent engineers' PSA model but not to any substantive review of the design engineers' model (which the peer review had relied on). There appeared to be discrepancies between these models; e.g. the expert accepted section properties in the former, but these did not seem to match the corresponding elements specified in the design documentation. The peer reviewer could not review the report properly without the expert's calculations.
Calculation of seismic loads, and EBF column hold down bolts	The expert had relied upon the independent engineers' PSA model for its findings, but a high level review of this and comparison with the consented drawings indicated that the section properties of many of the elements used for creating the model did not match the section properties of the corresponding elements specified in the drawings. The design engineer had created its PSA model for understanding the dynamic properties of the building structure and estimating member actions; once the latter were available it followed a capacity design approach for proportioning of members, which was reflected in the consented drawings. There was also a difference in floor level mass between the two models (overall, this was 12% less in the independent engineers' model). As the two models used different section properties for many elements and mass distribution was different, the implications were: • the fundamental periods were not fully representative so could not be compared • the sequence of modes was different, and the expert should consider the design engineer's model • once the periods of different modes changed, the mass participation of the different models would also change • the building period would be slightly larger than estimated by the independent engineer's model, even if the model's stiffness would have been the same. The fundamental period and modes of behaviour of the structure had a direct impact on how the seismic coefficient is determined and its magnitude. The peer reviewer reviewed the design engineer's model and calculation of the seismic coefficient "which complied with Building Code requirements". In the absence of a "true" model with the correct sections, including changes made during construction and capacity design, any comment or conclusion on the estimated fundamental period and resulting seismic coefficient was arbitrary.
Column on Grid 5-H	The independent engineers' comments and the expert's conclusions were based on nominally ductile loads, but the building was designed for limited ductile loads and had clearly defined seismic resisting elements in two different directions, so NZS 1170.5 clause 5.3.1.1 applied.
Foundation uplift by overturning	The building was designed for a ductility of $\mu = 3$ (not $\mu = 1.25$) so the overturning needed to be checked for this and with an appropriate overstrength factor. It was not clear which ductility the expert had assumed.

Peer reviewer's comments on the expert's first report of 12 April 2019 (letter 15 May 2019)	
EBF links against the stairwell	If this aspect was non-compliant, remediation appeared relatively easy.
Other items	**Items the peer reviewer considered outside scope:** The peer review and PS2 was limited to the building's superstructure so the peer reviewer said it was unable to comment on the expert's conclusions regarding the foundation's raft punching shear ("foundation system including foundation raft was not part of the [peer reviewer's] peer review"). The modified brace was also outside the scope of its review.
	Items with associated conditions on the PS2*: Two other items in the expert's first report had been identified as conditions to the peer reviewer's PS2, so no further comment was provided. These were the EBF active link connections into the columns' minor axis (paragraph 5.2.1, item 11) , and the additional item noted by the expert (the beam/brace bolted connection to the column minor axis, refer paragraph 5.2.3). * I note that the date of the PS2 referred to was not provided. The peer reviewer subsequently provided some clarification of this on 28 May 2019 (refer paragraph 5.3.2).
	Items not commented on The peer reviewer did not comment on the column splice capacity or on those items the expert's first report concluded were in accordance with B1/VM1.

C2 Technical meeting 19 June 2019

Some discussion points from the technical meeting held to discuss the expert's first report (refer paragraph 5.4) are summarised below.

Discussion points from the technical meeting 19 June 2019 included:	
Expert	Explained their assessment approach, noting that they were asked to look at very specific design areas; said they took a capacity design approach as the Design Features Report said the building was "limited ductile"; also said the first question (when reviewing) was whether the building could sustain a ductility of 3 – the expert concluded the detailing could not, and therefore based their further evaluation on a ductility factor of 1.25; said their assessment was also based on the section capacities and less on the actual PSA model outputs; said the connections were the limiting factors in many cases, and that column hold-down details were very eccentric. Also said they had limited information regarding the piles; and noted that the lateral resistance provided by the EBFs in the transverse direction was concentrated into the centre of the building but relied on the whole mass of the building to prevent overturning.

Discussion points from the technical meeting 19 June 2019 included:	
Design engineer	Questioned the expert's approach; maintained that the building was ductile; said had used the PSA model for the member design and had followed NZS 1170.5 and HERA guidelines. Regarding the piles: described the foundations and said that during construction concrete columns were connected to the original basement slab and the piles were installed to the previous design (i.e. as designed by the original engineering firm involved with the project, which was responsible for the substructure). Regarding the total overturning moment: considered had demonstrated Building Code compliance through calculations; considered that the building was narrow transversely but longitudinally very stiff, so stress longitudinally was very small compared to the transverse direction. Still did not consider the modified brace was a substantial modification to the design. Also noted there was a minor error in the shop drawings for the stairs and landing detail (regarding the construction joint) which could affect one of the conclusions in the expert's first report.
Peer reviewer	Said the scope of the peer review was for the building superstructure only; wanted to make sure the expert's review considered the original design basis and had taken the design engineer's response to the design concerns into consideration (i.e. the design engineer's letter of 21 March 2018).

Appendix D: Further submissions and the expert's response

The following table summarises: the design engineer's submission on the draft determination and on the expert's final report; and the expert's response to that submission. Other submissions on the draft determination are outlined in paragraph 6.2. Item numbering matches that used in paragraph 5.2.1 (note: the design engineer did not comment on items 9 or 13, or on the additional item identified in the expert's final report).

Topic/item	Design engineer's submission (9 September 2019)	Expert's response (21 October 2019)
General conclusions	The design engineer said they had carried out capacity design "with a procedure rigorously established with an accepted design methodology", and this was presented in the building consent documents. Considered the original concept was ignored by the expert (and other reviewers) who considered the structure nominally ductile, and also said the expert reviewed connection details conservatively, carrying out calculations partially based on the capacities of the steel sections. Disagreed with the draft's conclusions based on the expert's final report. Provided further feedback on specific items in that report (refer to rows below).	General comments on their overall approach: The expert did not agree a μ = 3 design had been achieved as contended by the design engineer, saying there were "obvious areas" where established requirements for detailing active links for EBFs and capacity design of other elements, as required by NZS 1170.5 and NZS 3404, had not been complied with. These included: • underestimating capacity design actions for columns/column splices/column base plates, beams and braces • inadequate transfer of shear to the column for the active link connecting to the column in the column minor direction • non-compliant detailing of the shear panel in the same active link • no recognition of changes in active link sizing from design to construction. The expert said they reviewed HERA Publication P4001:2013 'Seismic Design of Eccentrically Braced Frames (EBFs)' ("the HERA publication") against the relevant Standards, as the design engineer indicated the design was intended to follow the procedure in this publication. However, the expert did not believe the publication included any concessions to justify the deficiencies they had identified. That left two options: detailed consideration of the problems relating to achieving a μ= 3 design by correctly applying capacity design; or considering whether the building could achieve nominally ductile (μ = 1.25) requirements. The expert said the deficiencies that were recognised led to the latter approach – it was not a requirement of their review. The expert concluded the building was not in accordance with B1/VM1 for either μ = 3 or μ = 1.25. They said they could see from the calculations presented that the design (errors and omissions accepted) initially proceeded correctly, and in general accordance with the HERA procedure, to deliver a μ = 3 structure. However, they said the latter steps of the procedure that was adopted departed significantly from those recommended in the HERA publication.

Topic/item	Design engineer’s submission (9 September 2019)	Expert’s response (21 October 2019)
Comments on analysis and modelling	The design engineer considered the MRSA had been interpreted incorrectly; regarding torsional sensitivity, said the structural irregularity had been determined by NZS 1170 clause 4.5 and considered in the design; said there was a significant disagreement in the results from the two PSA models; for the ductile design an elastic model was set up first and then capacity design was carried out; the ductile structure could not be reviewed solely by an elastic analysis; considered design lateral loads were correct “as specified in the standards”.	General comments on analysis: The expert said they did not have the design engineer’s modal analysis to review but believed, from the calculations available, that the way this analysis had been interpreted and applied in the design engineer’s calculations was incorrect. Also identified other concerns arising from comparing the design engineer’s calculations with their own, including the: • identification of primary translational modes in each direction from the PSA analysis • level of accidental eccentricity adopted • consideration of biaxial actions on common columns in braced frames in two directions. In response to the design engineer: The expert did not disagree with the analysis method but questioned the application of the results. The expert inferred that the design engineer used the second fundamental period instead of the third to calculate the minimum base shear requirement for the longitudinal direction, which (as noted in the expert’s final report) gave a 25% lower design value for minimum strength than the loadings code potentially required. Commented again on the design engineer stating that the design was based on the HERA publication; said the frame design did not conform to this publication’s detailing requirements for a limited ductile EBF so calculating active links based on μ = 3 loads was not appropriate.
Comments on the expert’s review	Considered the expert’s conclusions were conservative and based on a “high-level review”, and that the expert’s section-based calculations led to quite large loads on the members compared to the actual loads driven in the analysis. Said the EBFs were originally designed in accordance with NZS 3404 (especially clause 12.11), and that the HERA design procedure “was adopted to avoid any controversy”. Said no engineers involved in the determination had commented on the calculations for the ductile structure, and it was inadequate to state the structure was non-ductile without a thorough review of these.	The expert said they had carried out a detailed review of the structural drawings issued for building consent and of the Design Features Report. Based on this and the expert’s own calculations they had established that the design, as detailed, did not comply with B1/VM1. The expert listed items they considered did not conform to the requirements of the HERA procedure or with NZS 1170 or NZS 3404. These included that: • a capacity design approach had not been adequately carried out for the design of the column splice, bracing connections and column baseplate • concurrent actions had not been considered • there were bolted joints in the yielding region • the tapered flange plates did not protect the bolted shear cleat or prevent it from being considered as located within the active link region • the clear length of the active link was not greater than the depth of the active link beam.

Topic/item	Design engineer's submission (9 September 2019)	Expert's response (21 October 2019)
		The expert also listed observations from their review of the calculation package submitted for building consent and said that, based on these observations, the existing calculation package fell short of confirming the design was in accordance with B1/VM1. The expert's observations included the following: • A load takedown for the building was not provided, so it was unclear how the seismic mass was calculated. • The analysis results were stated in the calculations with minimal computer output confirming the maximum shear force, axial load and bending moment values and the members these related to. The expert said it was difficult to assess from the information provided whether the values provided were the critical values, and it was also unclear whether global torsional requirements ('accidental eccentricity' requirement) had been allowed for in the analysis. Comparisons with values obtained from the independent engineers' analyses suggested that the accidental eccentricity requirement may not have been included in the member actions provided in the calculations. • The building periods were simply stated for each direction and there was no summary analysis table indicating periods, mass participation etc, which made it difficult to confirm whether the correct seismic load had been derived. • No calculations were provided for the design of the column splices or of certain spliced beam details. In addition, the calculations did not include the design of required restraints for the EBF adjacent to the stairwell with no natural restraint between columns. • The combined actions on common EBF columns were not considered. • The displacement calculation for the frame in the transverse direction incorrectly added the P-delta value to the lateral displacement (instead of multiplying the two). • The shear calculation for the link members used the effective depth between flanges to calculate the shear capacity (instead of the overall beam depth), so the overstrength capacity was underestimated. • The maximum overstrength value was incorrectly calculated in the longitudinal direction and was 3.0, not 2.4 as used in the calculations. • The baseplate design actions did not match the column design loads. The calculations indicated the axial design loads for the EBF columns at ground level were in the order of +/-11,000 kN (transverse) and +/-7,700 kN (longitudinal) whereas the baseplates had been designed for axial loads in the

Topic/item	Design engineer's submission (9 September 2019)	Expert's response (21 October 2019)
		order of +/-3,000 kN and +/-1,000 kN respectively. • The baseplate loads had been determined using the reactions out of the PSA model multiplied by an overstrength factor, but this factor did not appear to account for the actual provided active link overstrength capacity so was incorrect. The overstrength factor had also been applied incorrectly, as it should have been applied directly to the seismic reaction (the expert said the design engineer had applied this factor to the combined seismic reaction plus gravity load reactions, resulting in a significantly lower tensile demand being used to determine baseplate loads). • The hold down anchor bolts on Grid 6 had insufficient capacity to resist the tensile loads stated in the design engineer's calculations (although, as previously noted, these were incorrect), so the design engineer proposed sharing the load with the frame on Grid 5. However, the expert said the frame on Grid 5 also had insufficient capacity to resist the design loads "let alone support the additional load requirements from Grid 6". In addition, an adequate structural mechanism had not been provided to share the load. • The baseplate design did not include the columns and braces eccentric to the hold down bolt layout and did not take account of the cast-in hold down bolts offset below the plate; an offset which led to eccentricities that significantly reduced the bolts' load carrying capacity.
The expert's report: background and scope	Said did not have a chance to review the findings of the authority's consultants (refer paragraphs 3.4.1 to 3.4.3) until the determination was processed. Questioned the scope and nature of the expert's review and said it seemed to skip some procedures such as reviewing the design engineer's original calculations and PSA model, and a "full assessment" would clarify why different results had been achieved than the independent engineers' model.	(no comment)
1. Column splice capacity	Said the expert had calculated the tensile load conservatively based on the section capacity; the design engineer considered this a "significant error" in reviewing the steel members; the design engineer calculated a tensile load of 2 MN for the column in question; the splice used connection details from the	Considered that loads used to design the frame connections were not derived using the procedure in the HERA publication, did not conform with capacity design methodology, and did not account for combined actions on common columns in orthogonal frames. Said the connection had insufficient capacity to support the tensile demands plus the combined biaxial bending moments. Calculations submitted by the design engineer for the column splice design referred to a

Topic/item	Design engineer's submission (9 September 2019)	Expert's response (21 October 2019)
	HERA publication and represented 75% of the axial capacity (more than the 50% required).	standard connection detail which was not suitable for this column splice joint as it did not take account of biaxial actions.
2. Modified brace	Considered this detail was in accordance with B1/VM1; said analysis showed stress change (from original design) was mainly in the collector beam and brace; and the original section of the column was still satisfactory with the beam induced load.	Said the beam was 700 mm deep with a 450 mm clear yielding region, which did not conform with the HERA publication's requirements (i.e. that the yielding region must be as long as the beam depth). The revised beam size had insufficient capacity to support the original design loads and, as noted in the expert's final report, the amended bracing layout resulted in significantly higher actions in the beam, column and connection that had not been accounted for in the design.
3. Calculation of seismic loads, torsional stability	Considered the expert's conclusions were based on a review of the independent engineers' PSA model but that was not correct or appropriate; the design engineer's PSA model had been established for capacity design (with μ = 3.0) and confirmed a plastic hinge would develop at the active links; torsional sensitivity was determined by NZS 1170 clause 4.5 Structural irregularity, not the selected modes of the analysis.	Said as frame detailing did not conform to the HERA publication's design requirements for a limited ductile structure (μ = 3) the appropriate way to assess the building was to assume available ductility was limited to the range μ = 1.0 to 1.25. The only way limited ductile behaviour could be assumed would be to ensure the detailing met the limited ductility requirements in B1/VM1. Said as they did not have a copy of the design engineer's PSA model they could not comment on the validity of the analysis. However, minimum design base shear demands should be based on the first response mode for each direction irrespective of the level of ductility used in the design. The expert's review indicated that the design engineer appeared to have based minimum base shear demands on the second mode (which was for the transverse direction) rather than the third mode which was the first response mode for the longitudinal direction, leading to a demand 25% lower than required by NZS 1170.5.
4. EBF column hold down bolts	Considered the expert had calculated excessively high loads based on section capacities and for an elastic structure; whereas the design engineer considered the structure ductile and bi-directional action was not significant.	For comparison purposes, the expert: calculated column loads based on the HERA publication procedure assuming the detailing adequately allowed for μ =3; combined these loads for the two orthogonal seismic resisting systems, and used them to assess the baseplate capacity. The expert noted the capacity design requirements of NZS 1170.5 (clause 5.6.1) including additional requirements in the appropriate material standard, NZS 3404, which were for consideration of combined actions for the design of common columns of orthogonal braced frames. The HERA publication also required common EBF columns to be designed for concurrent actions but the expert said this was not done. Said the design engineer had designed the baseplate on Grid 5-E for a tensile load of 1,929 kN but this was not consistent with column loads in the calculation package issued for building consent, which indicated that the column load at Level 1 (column away from link) was 8,196 kN. Considered there were a number of errors in the

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		baseplate design connection calculations submitted for building consent, including: • the overstrength factor did not account for the overstrength capacity of the bracing system • the reactions from the PSA analysis should have been amplified by a factor of 2.3 when designing the columns, baseplates and foundations at ground level. The design engineer had applied an overstrength factor of 1.5 to reactions obtained from their PSA analysis to design the baseplates • the hold-down capacity relied on load sharing between neighbouring frames (on Grids 5 and 6) but the combined capacity of the column hold down bolts was significantly less than the combined tensile demand, and there was no reliable structural mechanism for sharing the load between these frames.
5. Column on Grid 5-H	Said the expert reviewed the axial load based on section capacity and considered combined actions, resulting in what the design engineer considered excessively high loads.	Did not agree with the design engineer that EBFs were designed in accordance with the HERA publication, as it requires designing for combined actions. Acknowledged that the columns were designed assuming a limited ductile (μ =3) analysis was acceptable but, as a number of the EBF connection details did not conform with the HERA publication requirements, the analysis should be limited to nominally ductile.
6. Pile capacity	Said the expert considered all axial loads solely supported by the piles and had ignored the raft foundation system with a compacted base course, which made the piles supplementary. The foundation design had been reviewed and the slab appeared to support a large axial load transferred from one column by spanning between adjacent piles; column axial loads were also distributed across all piles.	Said no evidence was provided to demonstrate that fill was compacted to a level suitable to provide adequate support for the building, so the expert considered this was supported solely on the piles. Said column reaction loads used by the design engineer did not comply with the HERA publication requirements and were significantly lower than those required by the relevant design standards.
7. Foundation uplift by overturning	Considered the expert's review of the design engineer's calculations misleading: said had provided calculations for the least favourable (transverse) direction and considered these showed the raft slab sufficient to support the uplift and adequately reinforced to distribute stress through the section; these calculations were conservative and ignored soil structure interaction at the bottom of the raft slab and friction of the slab with the original basement wall columns along the perimeter, which would provide	The expert provided the figures they had used for estimating the building's overall stability and said as overturning moment was greater than resisting moment there was "a potential stability issue". Also noted that their overall stability calculation assumed the building's full weight but this was unlikely (as EBFs were centrally located and the raft slab did not appear designed to cantilever beyond the frames to pick up the weight of the building ends). Therefore, they considered the restoring moment was likely to be lower than they had calculated.

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	further resistance to overturning.	
8. Raft foundation (punching shear)	Said baseplates were designed for the uplift load and were adequate to support the punching shear; the shear reinforcement of the raft slab provided further capacity.	Said loads used by the design engineer to check for punching were incorrect; punching shear check did not correctly account for side columns being close to the edge of the raft slab; and column bases were recessed 200 mm below the concrete surface so it was incorrect to assume the full 1.1 m depth when calculating punching shear capacity.
10. Precast stairs at landing	Considered the stair detailing compliant; seismic movement was designed for via a sliding connection (a slotted plate) and installed on site as designed.	Described features of the stairwell as-built, based on the photographs provided after the technical meeting. Concluded the stair sliding requirements had not been provided as designed and detailed.
11. EBF active link connections to minor axis of column	Considered the expert's conclusions were based on a high-level review; also considered the expert had incorrectly calculated the capacity of, and demand on, the web cleats; said analysis also showed member forces of the EBFs in the longitudinal direction were much less than in the transverse direction and, subjected to these forces, a strong column was achieved with the wide flanged section in the weak axis.	Said the active link was a shear yielding mechanism, so yielding occurred in the beam web region, and the flange width did not contribute to the beam shear capacity. The HERA publication stated that bolted connections were prohibited in the yielding region for a limited ductile frame. As the splice detail provided did not conform with this, the frame design must be limited to μ = 1.25. Said the HERA publication also required the active link connection to be designed to resist the overstrength capacity of the yielding region, but the cleat provided had insufficient shear capacity to support the overstrength shear capacity of the beam (the expert provided calculations to support this view, which concluded that "The cleat shear capacity (766 kN) is less than the beam shear capacity (987 kN) and significantly less than the beam overstrength shear capacity (1,530 kN)". Said as the cleated connection did not conform with capacity design requirements the frame must be limited to μ = 1.25.
12. EBF links against stairwell	Said site investigation would determine if additional restraints were needed.	Said that, based on the information currently available to the expert, the EBF adjacent to the stair was not restrained between columns.