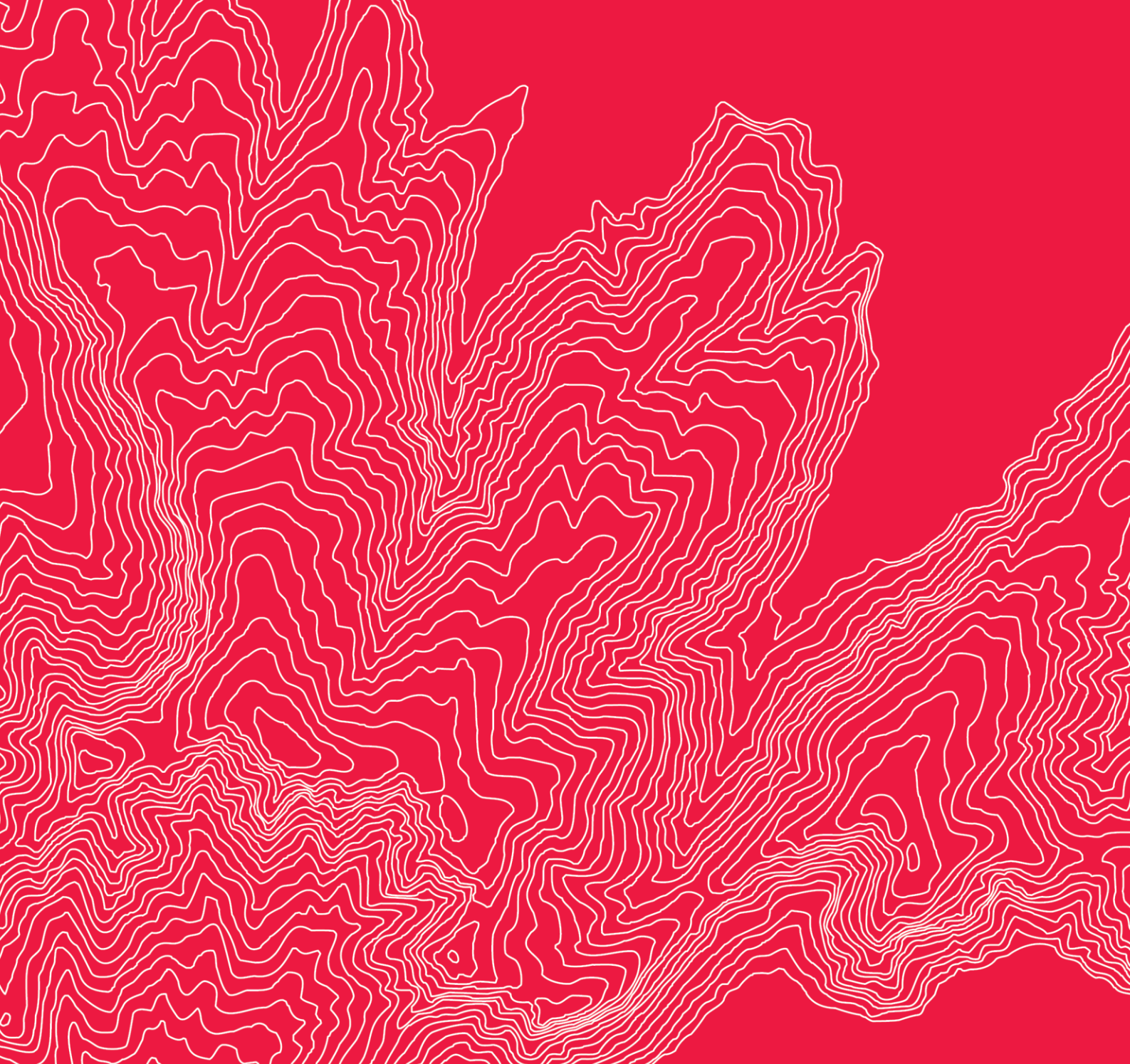




DSA Summary Report

VERSION A

**eliot
sinclair**

41 Weld Street, Hokitika

Prepared for Westland District Council
503051

DSA Summary

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Quality Control Certificate

Eliot Sinclair & Partners Limited
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Executive Summary

Purpose

Eliot Sinclair & Partners Limited (Eliot Sinclair) have reviewed the Detailed Seismic Assessment (DSA) of the Westland District Council building at 41 Weld Street, Hokitika, carried out by WSP and dated 26 November 2019. This report is a short summary for the Council. It covers only the parts of the building that scored below 34%NBS and explains the consequences if those elements reach their probable capacity.

Overall Result

The building scored 25%NBS (IL2), which corresponds to a Grade D building under the NZSEE grading scheme. Although the first-floor steel portal frames were assessed as being below 20%NBS, an alternative load path is provided by timber-framed walls, so the building score has not been reduced below 20%NBS. A Grade D building has a high earthquake risk, approximately 10 to 25 times greater than that of a similar new building. A score below 34%NBS means the building is classed as earthquake-prone under the Building (Earthquake-prone Buildings) Amendment Act 2016.

Parts of the Building Below 34%NBS

The following parts of the building scored below 34% NBS:

- **Steel Portal Frames at D1, D2 & D3 (25%NBS):** The seismic demand on the steel portal frames exceed their assessed capacity under seismic loads – however with the presence of timber framed walls throughout the first floor it is assumed that these will act as a secondary load path i.e. the building will continue to resist lateral loads beyond the yielding point of the steel portal frames. Parts of the roof in open areas could collapse in.
- **Panels to D2 (30%NBS):** The concrete panels (PC16, PC17, PC31 & PC32) to D2 have been assessed to be 30% NBS under out-of-plane seismic loads. Their failure could result in the panels falling in or out.
- **Diaphragms (25%NBS):** Diaphragm connections to D2 are limited by the pull-out capacity of the M12 TCM inserts. The pull out of the anchors could result in panels falling out and the floor structure collapsing due to loss of support.

Recommendation

We recommend the Council plans the strengthening of the above items. A high-level concept scheme for upgrading the building to achieve 34%NBS is presented in Section 5. We support the NZSEE recommendation to strengthen to as close to the standard of a new building as reasonably practicable, and at least 67%NBS. The decision on continued occupancy of the building remains with the owner and tenant..

1. Introduction

Eliot Sinclair & Partners Limited (Eliot Sinclair) have been engaged by Westland District Council to summarise how the Council building at 41 Weld Street, Hokitika (the Pakiwaitara Building), would perform in a design level earthquake (IL2). This report addresses only the building parts and elements assessed at below 34%NBS, and explains what may happen if the forces on these parts exceed what they are likely to withstand.

The building is given a score called %NBS (percentage of New Building Standard). The %NBS rating measures the ultimate seismic structural capacity of a building relative to the minimum required of a new building on the same site under the current New Zealand Building Code. The lower the score, the higher the risk to people inside. A building rated below 34%NBS is considered earthquake-prone under the Building (Earthquake-prone Buildings) Amendment Act 2016.

This summary is prepared for the Council as our client. The method, assumptions and results for the whole building are in the full Detailed Seismic Assessment (DSA) report by WSP(Reference - 6-WMAK1.01 / 005GG, Version A, 26 November 2019).

2. Building Overview

The Pakiwaitara Building has a footprint of approximately 1,242 m² and a floor area of approximately 2,246 m². Most of the building was constructed in 1993/94, with the original south-end section constructed at an earlier, unknown date..

For the purposes of this assessment, the building has been divided into 3 distinct areas: D1, D2 and D3, as shown in Figure 2 below.

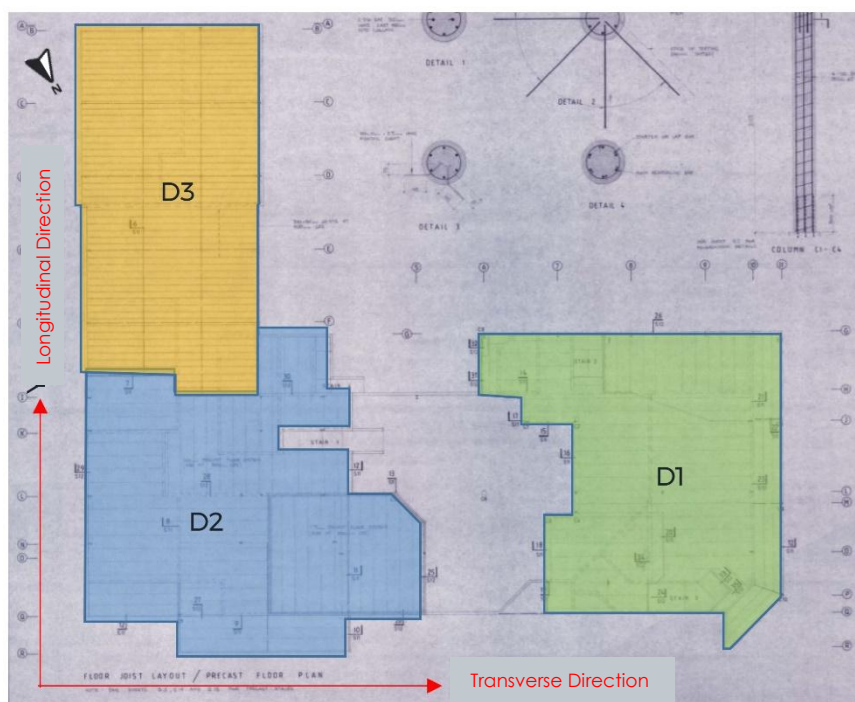


Figure 1. Floor plans indicating original construction (purple) vs later stage extensions (green)

The building is operated primarily as a commercial building, with retail, café, office and recreational activities run by various tenants. Based on these uses, the building is classified as Importance Level 2 (IL2) in accordance with AS/NZS 1170.0:2002.

2.1. Gravity Structure

Based on our review of the available documentation and a site visit to inspect the visible structure, we understand that the primary gravity load-resisting systems for this building comprise:

- **D1 & D2 Areas:** A Lightweight profiled metal roof cladding is supported on sawn timber purlins, which span between steel portal frames typically oriented in the east-west (transverse) direction. The south-eastern boundary wall is made of precast concrete panels that extend the full height of the building. Other first floor walls are timber-framed, and the ground floor walls are precast concrete panels. The first floor is a precast rib-and-infill floor system, with a large central atrium that divides the D1 and D2 areas. This suspended floor is supported on the precast concrete wall panels, as well as on some cast-in-situ concrete beams, steel beams and circular concrete columns. The ground floor is a reinforced concrete slab-on-grade. The foundations are reinforced concrete barrette piles, typically 2.0 m deep, with widths ranging from 500 mm to 900 mm.
- **D3 Area:** Two-storey steel portal frames, oriented in the east-west (transverse) direction, support both the roof and the first floor. The roof is lightweight profiled metal cladding on sawn timber purlins, which span between the steel portal frames. The first floor is timber particleboard flooring on sawn timber joists, spanning between steel beams, which in turn span between the two-storey portal frames. Part of the south-eastern boundary wall is made of precast concrete panels that extend the full height of the building. Timber-framed walls line the remainder of the upper floor, and the original concrete structure forms the ground floor walls. The ground floor is a reinforced concrete slab-on-grade. Parts of the original structure have been underpinned with shallow piles, while the remainder sits on shallow strip footings built integrally with the original concrete walls.

2.2. Lateral Structure

Based on our review of the available documentation and a site visit to inspect the visible structure, we understand that the primary lateral load-resisting system for this building comprises:

- **D1 & D2 Areas:** The lightweight roof, with steel tension-only bracing in some locations, acts as a flexible diaphragm. It transfers lateral loads to the steel portal frames in the longitudinal direction and to the timber walls in the transverse direction (refer Figure 2). The precast rib-and-infill floor acts as a rigid diaphragm at first floor level, transferring lateral loads to the precast concrete wall panels at ground level. Along the eastern wall, the panels extend from ground floor to roof height. The void in the middle of the building creates plan irregularity (hence the division into D1 and D2 for bracing analysis) and out-of-plane bending issues for the precast concrete wall panels. The panels are anchored to the concrete foundation by steel fishtail plates, 450 mm long and cast 325 mm into the foundation.
- **D3 Area:** In the "Existing Jade Workshop", the original part of the building, the lightweight roof acts as a flexible diaphragm, transferring lateral loads to the two-storey portal frames in the longitudinal direction. In the transverse direction, timber walls resist lateral forces at roof level, and the original concrete structure resists lateral forces at ground level. The first floor is timber and acts as a flexible diaphragm.

3. Seismic capacity of elements less than 34% NBS(IL2)

The results of the Detailed Seismic Assessment are reported as a %NBS. The %NBS value contains uncertainty because of the assumptions and simplifications made during the assessment. There are many variables in seismic assessment, and estimating the relative life safety risk for a particular building is uncertain. Among other factors, this uncertainty comes from the random nature of earthquakes, the complex response of buildings to earthquake shaking (particularly at the point of structural failure), the variability in construction quality, and the lack of accurate records of buildings' construction. Because of these factors, %NBS should be viewed as indicative of the engineer's confidence in the expected seismic performance of the building, rather than as an exact prediction¹.

Table 1 lists the parts of the building that scored below 34%NBS(IL2). The concrete floor diaphragm connections at 25%NBS*, sets the rating for the whole building.

Table 1. Elements Below 34%NBS(IL2)

Building element	Direction of shaking	%NBS	What limits it
Steel Portal Frames to first floor of D1/D2	E-W	25%	Flexural/bending strength of Portal Frame Rafters
Panels to D1-D2	N-S	30%	Out of plane performance of concrete panels
Concrete Floor Diaphragms	Both	25%	Connections to shear walls

¹ July 2022 MBIE - Seismic Risk Guidance for Buildings

3.1. Steel Portal Frames at D1/D2/D3 (25%NBS)

Element description: Bracing in the east-west direction in areas D1, D2 and D3 is provided by steel portal frames.

Risk: If the seismic demand on these portal frames exceeds their assessed capacity, the portal beams and columns could fail in bending resulting in excessive displacements. In this scenario localised sections of the roof (D1, D2 and D3) and the first floor (D3) could collapse, mostly in open areas that have no timber-framed walls to hold the roof up. Some timber-framed elements may provide a secondary load path for the first-floor frames. However, because they were not designed for this purpose, their load-carrying capacity would be limited by their connections to the roof bracing system.

Basis of analysis: The analysis is based on the available existing building drawings. Because of the vertical irregularity, with a stiff ground floor and a lightweight upper floor, the upper structure is expected to sway more because of dynamic effects and higher-mode responses.

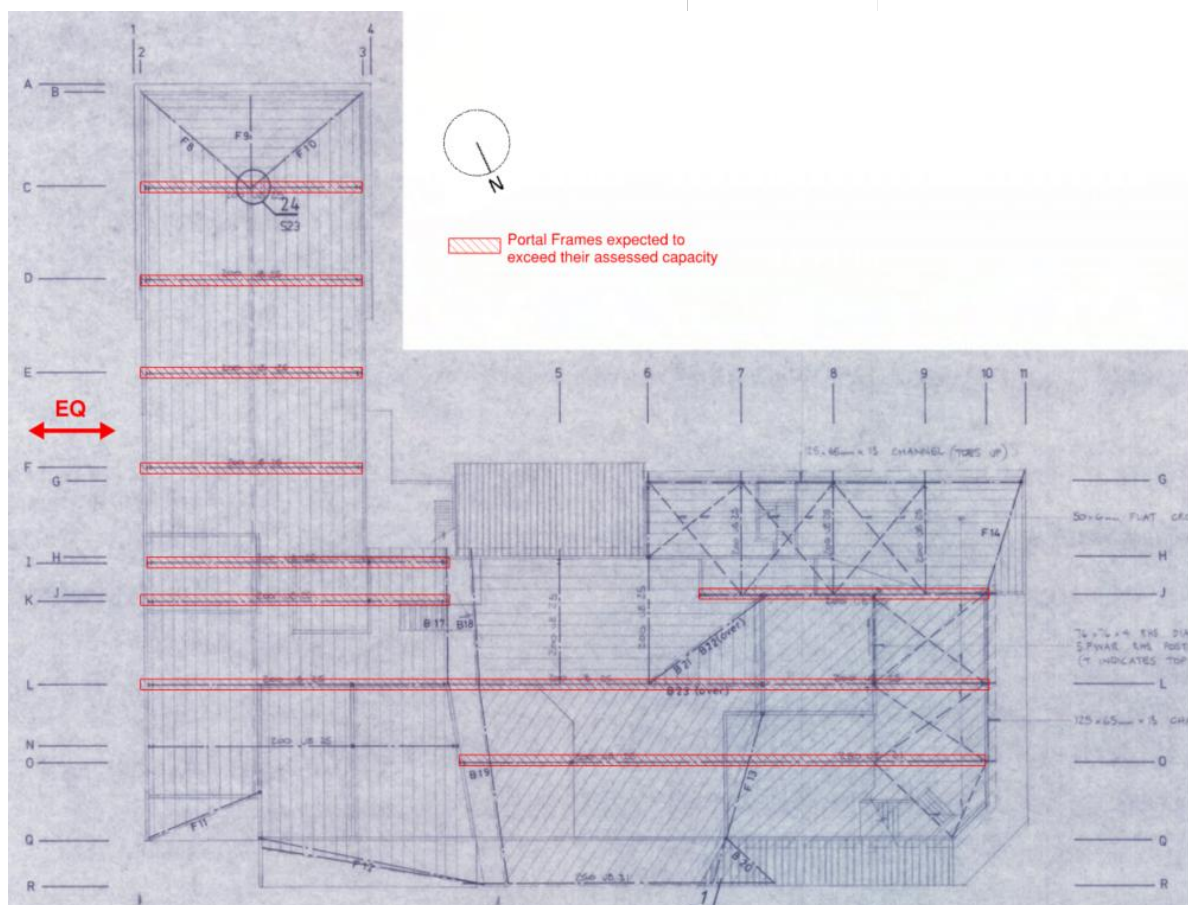


Figure 2. Steel portal frames rated lower than 34% NBS

3.2. Concrete Panels to D1/D2 (30%NBS)

Element description: Panels P16-P17 and P31-P32 (Out-of-plane performance)

Risk: Because there is no floor structure between areas D1 and D2, these panels span further than the other panels, but do not appear to have been detailed to withstand the additional out-of-plane seismic loads. If a panel bends and fails, part of the panel, and possibly some of the connected roof structure, could collapse.

Basis of analysis: The %NBS rating depends on variations in concrete strength, reinforcement spacing and similar factors. However, the lack of a robust load path to stabilise the panels under out-of-plane loads, combined with the consequences of failure, is considered to justify a rating of less than 34%NBS.

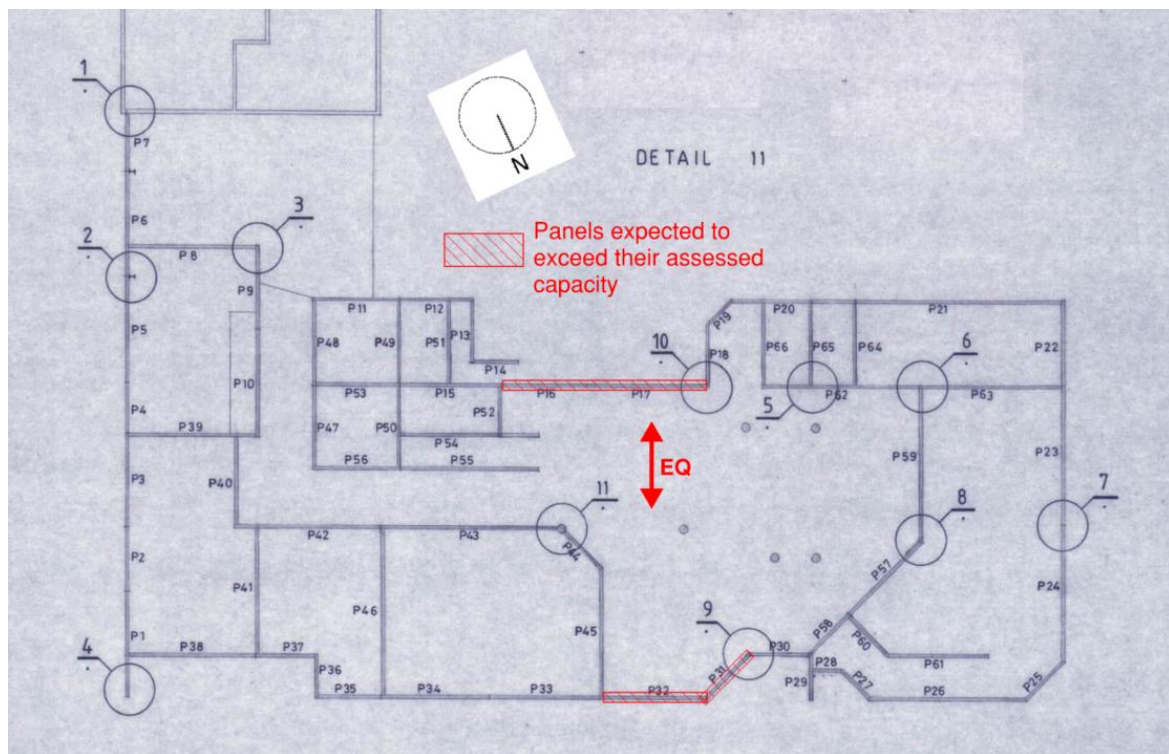


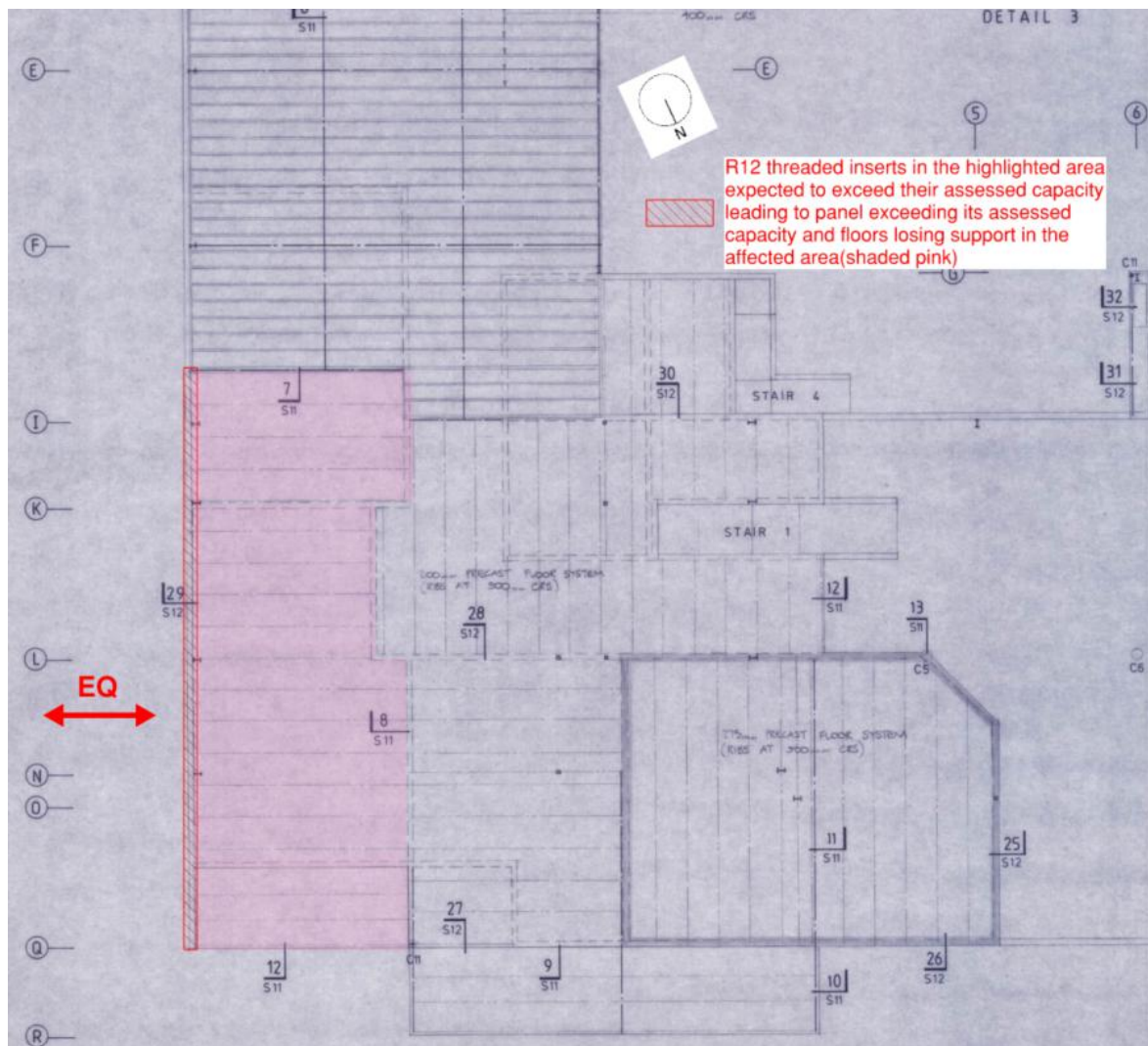
Figure 3. Concrete panels rated as less than 34% NBS under out-of-plane loads

3.3. Concrete floor diaphragms (25%NBS)

Element description: Along Grid line 2 (refer to Figure 4 below), M12 threaded inserts have been used to tie the floor diaphragm into the external panel shear walls. The pull-out strength of the inserts is rated at 25%NBS.

Risk: If the ties connecting the floor structure to the supporting panels pull out, the Stahlton rib-and-infill floors could lose their support and collapse. The external double-height panels could also fail in bending without the ties securing them to the first floor. In addition, in most areas the current support provided to the Stahlton ribs does not meet the minimum seating length required for a new building.

Basis of analysis: We have based our analysis of the pull-out strength on available structural drawings, however the drawings do not indicate the exact spacing of TCM R12 threaded Bar starters. Further investigation on site with a rebar locator could potentially affect the reported rating.



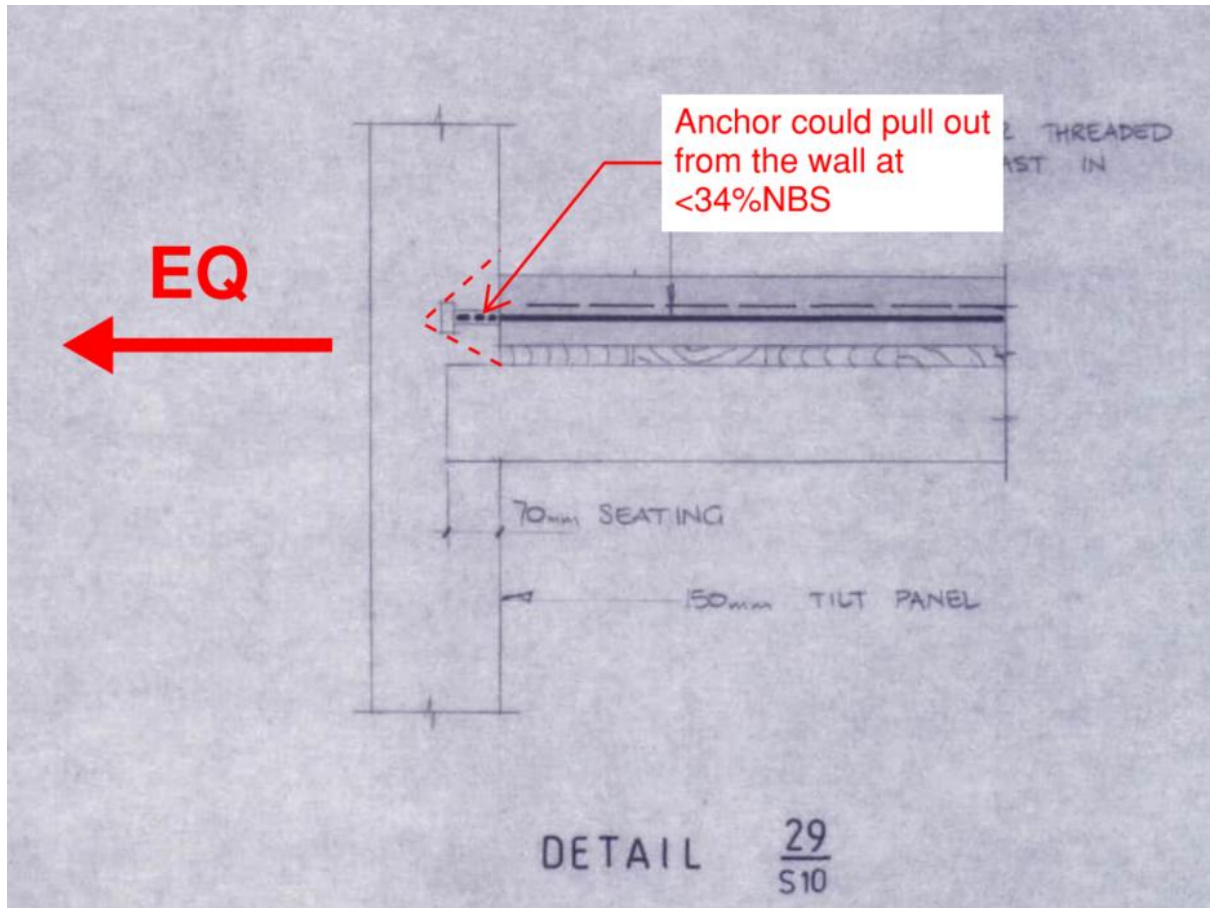


Figure 4. Plan view of the floor and detail to TCM12 threaded rod connection to floor diaphragm.

Occupancy of ground floor D1

The Council has asked us to comment on the potential occupancy of part of the ground floor of Building D1, shaded in Figure 5 and currently used as a café.

This ground-floor area comprises reinforced concrete wall panels supported on a concrete piled foundation system. The wall panels support a reinforced concrete floor above. No elements within this area have been assessed as having an NBS rating below 34%. However, elements of the steel structure on the floor above the concrete café structure are rated below 34% NBS. This indicates a potential high risk for these elements to fail in a significant earthquake, which may result in local or global collapse of the roof above.

Notwithstanding this, it is considered unlikely that collapse of the lightweight roof above the concrete café structure would create a life-safety risk for occupants of the ground-floor area described above. This is an assessment-based opinion and does not guarantee that risk is absent.

If the occupancy decision is based on a 34% NBS target, continued use of this ground-floor area may be possible, provided suitable mitigation measures (e.g. propping or shoring) are implemented to reduce the effects of a potential roof collapse. Specific engineering advice should be obtained to confirm these measures.

Other matters, including access and egress, accessibility and building services, may also be affected. These should be assessed before any occupancy decision is made.

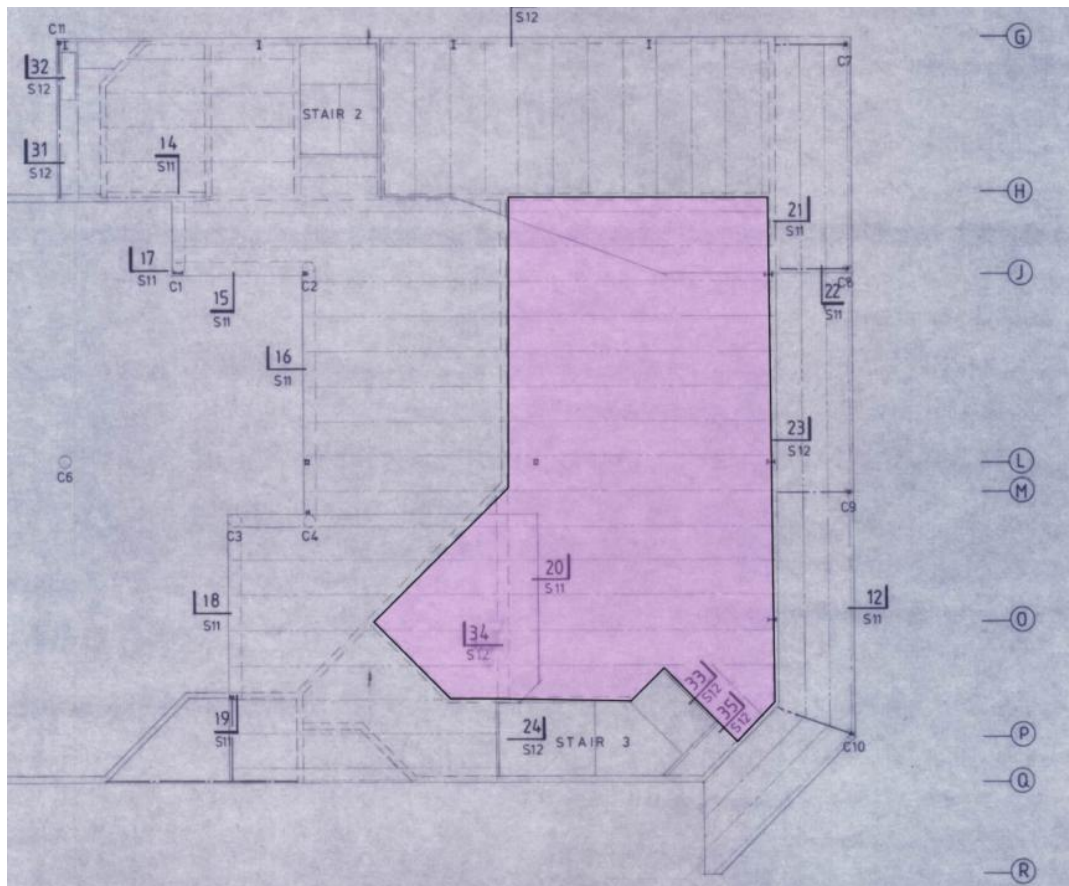


Figure 5. Ground floor of D1 building area (shaded pink)

4. Seismic Grade & Relative Risk

All items discussed in Section 3 scored below 34%NBS. The building's overall seismic rating is governed by its weakest element and is assessed at 25%NBS. This places the building in Grade D, with a risk 10 to 25 times greater than that of a similar new building and classifies it as earthquake-prone under the Act.

The parts of the building not covered here scored 35%NBS or more. These are the first-floor timber walls, the timber floor diaphragm, the precast (PC) shear walls (in-plane performance), the PC spandrels, the D1 diaphragm, and the D3 steel frames and shear walls.

Strengthening the building to 34%NBS would place it in Grade C, which corresponds to a medium risk level and a risk 5 to 10 times greater than that of a similar new building. Damage would still be expected in a significant earthquake.

5. Concept Strengthening Design

The high-level concept strengthening scheme appended to this report would improve the building's rating to 34% NBS. For the elements identified above, the scheme includes:

5.1. Portal Frames

The lateral capacity of the first floor of the D1 & D2 areas, and the ground & first floors of the D3 area, is limited by the lateral stiffness and flexural strength of the existing portal frame structures.

To strengthen & stiffen these areas of the building, the following works area proposed:

- **Transverse Direction:** Enhance the strength & stiffness of the existing portal frames by the installation of additional steel frames placed strategically within and connected to the existing frames. This methodology promotes the future flexibility of the building by largely retaining the existing open interior spaces. In selected locations to the building perimeter, tension only cross-braces are proposed.
- **Longitudinal Direction:** Install new steel braced frames to be constructed within or against existing exterior and interior walls.
- Improve load distribution with new RB20 Reid brace roof bracing.

We note that the exact location and selection of braced frames vs portal frames will need to be reviewed as part of the ongoing design development to ensure coordination and optimisation with any proposed architectural reconfiguration of the internal spaces and any proposed recladding of the building.

5.2. Suspended floor & concrete panels

The lateral performance of the ground floor to D1 & D2 relies on the suspended concrete floor diaphragm distributing lateral loads into the in-plane precast concrete shear walls, which then need to transfer these shear loadings down to foundations. Some strengthening of the connections and shear walls will be required as follows:

- Provide new seating angles beneath ribs where seating does not comply (refer to seismic strengthening drawings sheet S04).
- Provide additional shear connections to ensure adequate load transfer from diaphragm into shear-walls (refer to seismic strengthening drawings sheet S04). – *Note that additional investigation of existing ties may eliminate this requirement.*
- Provide new transom beams across atrium between D1 & D2 areas to restrain full-height panels out-of-plane (refer to seismic strengthening drawings S04).

Before design and costing can be finalised, we would need to validate/verify the following:

- Existing D3 concrete walls to be scanned to confirm reinforcement
- Closer inspection to roof structure may be required.
- More clarity is required on the span direction of concrete stairs to D1/D2.
- Some invasive inspection of welds may be required, particularly to panel hold downs.
- Additional investigation to foundation / support along Grid A (Building D3) may be required.

6. Building (Earthquake Prone Buildings) Amendment Act 2026

Based on our understanding, the Building (Earthquake-prone Buildings) Amendment Act 2026 is expected to take effect from 1 July 2027. Under the proposed system, the existing earthquake-prone building (EPB) framework would be replaced, and the definition of an earthquake-prone building would be limited to specified buildings within designated seismic zones.

A building within scope of Building (Earthquake Prone Buildings) Amendment Act 2026 would be an earthquake-prone building if—

- a) the building is located in a medium or high seismic zone; and*
- b) either of the following applies:*
 - i) the building was designed before 1 January 1976 and is at least 3 storeys high and built of concrete or other heavy materials (as explained in the EPB methodology), and is evaluated, using the EPB methodology, as an earthquake-prone building; or*
 - ii) the building is built of unreinforced masonry (as explained in the EPB methodology) and is evaluated, using the EPB methodology, as an earthquake-prone building.*

Based on the above criteria, it is our interpretation that the building at 41 Weld street, Hokitika **would likely not be classified as an Earthquake prone** building under the Building (Earthquake-prone Buildings) Amendment Act 2026.

From 1 July 2027, New Building Standard ratings (%NBS) will no longer be used to identify EPBs. No buildings will be required by the Building Act to have %NBS ratings. %NBS ratings would be voluntary². Notwithstanding the above, we consider it is likely that engineers would still be using %NBS to communicate building deficiencies to the building owners as it has been widely adopted.

The risk of building damage due to earthquakes has not changed. Aotearoa New Zealand is a seismically active country and earthquakes can occur anywhere. The new EPB system focuses on buildings that are expected to pose the greatest risk to life.³

The Act changes how buildings are classified and regulated. It does not change the underlying seismic hazard or a building's physical condition. A building's status under the new EPB system should not, by itself, be taken that its seismic risk has changed. Any conclusion about the building's actual seismic risk should be based on a building-specific engineering assessment.

² MBIE "Changes to the earthquake-prone building system" Building Performance <https://www.building.govt.nz/managing-buildings/managing-earthquake-prone-buildings/changes-to-the-earthquake-prone-building-system>.

³ MBIE, above n 1.

Disclaimer

This report has been prepared by Eliot Sinclair & Partners Limited ("Eliot Sinclair") only for the intended purpose as a Detailed Seismic Assessment Summary Report.

The report is based on:

- Desktop review of available building information and reports.
- On – site investigation of as-built structure.

Where data supplied by other external sources, including previous site investigation reports, have been relied upon, it has been assumed that the information is correct unless otherwise stated. No responsibility is accepted by Eliot Sinclair for incomplete or inaccurate data supplied by other parties.

Whilst every care has been taken during our investigation and interpretation of describe conditions e.g., subsurface conditions available data to ensure that the conclusions drawn, and the opinions and recommendations expressed are correct at the time of reporting, Eliot Sinclair has not performed an assessment of all possible conditions or circumstances that may exist at the site.

Variations in conditions may occur between investigatory locations and there may be conditions such as include examples e.g. subsoil strata and features that were not detected by the scope of the investigation that was carried out or have been covered over or obscured over time. Eliot Sinclair does not provide any warranty, either express or implied, that all conditions will conform exactly to the assessments contained in this report.

The exposure of conditions or materials that vary from those described in this report, or occurrence of additional strong seismicity, or any update to the Building Act, NZBC or MBIE's Guidance may require a review of our recommendations. Eliot Sinclair should be contacted to confirm the validity of this report should any of these occur.

This report has been prepared for the benefit of Westland District Council and the regulatory authority for the purposes as stated above. No liability is accepted by Eliot Sinclair or any of their employees with respect to the use of this report, in whole or in part, for any other purpose or by any other party.