
Seismic strengthening of the three-storey URM CApSc building

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ABSTRACT

The three-storey, bluestone and Oamaru Stone Consumer and Applied Science (CApSc) building (formerly the Home Science Block) at the University of Otago, originally constructed in 1918, was identified as earthquake-prone. The structure is a Category 1 listed heritage building, forming part of the University's Gothic complex. In 2019, the University committed to a comprehensive seismic strengthening and refurbishment programme designed to be sympathetic to the original heritage fabric. This paper describes the identified seismic vulnerabilities of the unreinforced masonry structure and details the strengthening solutions considered and ultimately adopted. Key lessons learned throughout the assessment and construction process, balancing modern engineering requirements with heritage preservation principles, are presented. The project ensures the building's continued functionality and long-term enjoyment.

1 BACKGROUND

In 2019 the University of Otago launched a redevelopment programme to enhance the functionality of its facilities. Egis NZ Ltd (Egis), formerly Calibre Consulting, were appointed as the structural engineering consultants for the proposed Food Science redevelopment, including the Consumer and Applied Science (CApSc) building.

2 BUILDING DESCRIPTION

The CApSc Building, located at 89 Union Place, is a three-storey unreinforced masonry (URM) structure constructed circa 1918, comprising a ground floor, first floor and second floor (attic). The building has a rectangular plan with an approximate footprint of 30 m by 16 m and is listed on the New Zealand Historic Places Trust Register at No. 325 as a Category I Building.

The building is founded on no-fines concrete strip foundations with timber suspended floors and a timber-framed roof. Perimeter walls are of double-leaf masonry construction with a rubble-filled cavity. The external leaf comprises bluestone (vesicular basalt) in ashlar form (squared blocks and even pointing) with



Figure 1: North elevation of the CapSc building (Credit: Cook Brothers Construction, University of Otago)

dressed Oamaru stone (limestone) used for architectural elements, while the internal leaf is primarily clay brick masonry with lime mortar. Wall construction varies with height, transitioning at the attic floor level to limestone masonry internally with a reduced cavity, while the external stone veneer remains continuous.

A small basement in the middle of the floorplate houses the boiler room and is constructed with a concrete slab-on-grade and concrete retaining walls supporting portions of the ground floor. Internal loadbearing walls consist of two or three brick wythes, resulting in overall wall thicknesses of approximately 215 mm to 330 mm.

The ground and first floors are timber-framed, supported by masonry walls, timber bearers, shallow concrete piles, and localised steel beams. The roof comprises slate tiles supported on timber sarking and rafters. Deep timber trusses support both the attic floor and roof, with transverse trusses bearing directly on the perimeter masonry walls and longitudinal trusses supported by transverse members. Steel ties and brackets provide localised truss connections.

Gable walls comprise limestone masonry supported on concrete lintels over window openings, with brick piers providing support.

3 SITE INVESTIGATIONS

Before proceeding with the design, we recommended that the University undertake detailed site investigations and carry out a new seismic assessment to confirm the buildings %NBS rating and identify the element capacities. The investigations included opening up the suspended timber floors to confirm their construction, span and connection details to the walls. Wall plaster finishes were stripped off to determine the brick bond arrangement, and core samples taken to verify the wall make up and for material testing.

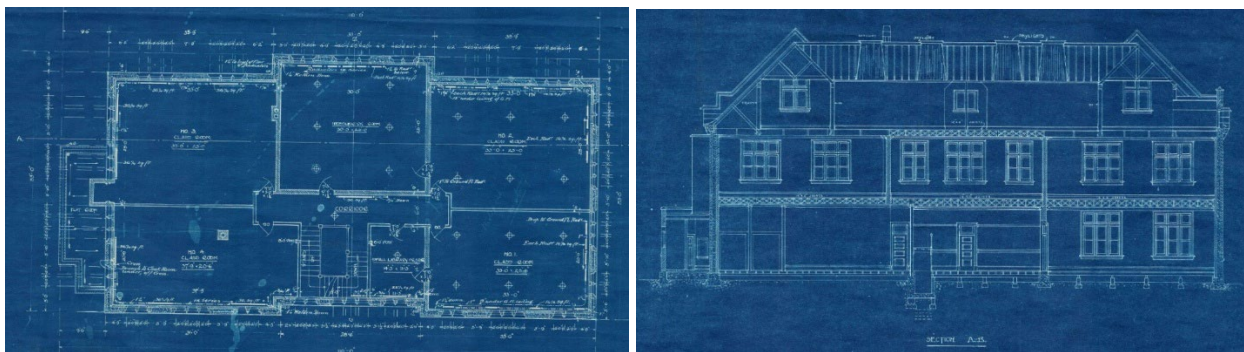


Figure 2: Sample of original Anscombe Architectural drawings

3.1 Investigation findings

Visual inspections and measurements were used to confirm the general layout of the building based on the original existing drawings. Room layouts and general dimensions aligned with drawings and wall locations were confirmed. Internal wall thicknesses were measured. Minor changes to masonry walls were observed where new door openings had been made or old windows filled in.

The findings of the opening up works confirmed in much greater detail than could be assumed, the structure's materials, elements and load paths briefly described in Section 2 above. Unsurprisingly, the floor joists were not tied to the walls and were typically simply pocketed into the masonry walls with 50mm of bearing. The flooring was thicker than expected, generally consisting of two layers of tongue and groove; diagonal 145 x 25mm boards with straight 90 x 20mm boards over.

3.2 Material testing

Dizhur Consulting were engaged to undertake material testing of the samples and completed a Laboratory Material Testing Report in November 2019.

The clay bricks were found to have an average compressive strength around 26MPa.

The mortar was determined to have a strength of 1.2MPa.

The compressive strength of stone is normally very variable, depending on where the stone has been mined. In general, limestone from Oamaru has an average compressive strength of 3.5Mpa. The limestone tested from the CAPSc Building has very similar properties.

The Bluestone testing showed that the stone has a very high compressive strength (>100MPa), which is as expected as it is a very durable material.

Salt testing was also carried out and that indicated that, *while salt is present in the walls, it is below 0.5% by weight which is generally considered the level at which it may start to cause damage to stonework*, (Lloyd RA (2021), "CAPS Building – Salt Testing", The Stone Company).

4 DETAILED SEISMIC ASSESSMENT

The new assessment provided a detailed picture of the building, confirming the structural deficiencies and painting a clear picture for strengthening. The assessment confirmed the building to be earthquake-prone, with a capacity <15%NBS. A key vulnerability was the makeup of the exterior walls, constructed as distinct layers with limited ability to act compositely.



Figure 3: Sample photos of intrusive investigations

4.1 Wall material failure mechanism

The CApSc building was constructed using clay bricks as the internal skin and bluestone as an external veneer, with highly variable rubble/no-fines concrete filling the void in-between. No transverse ties connected the three wythes together. Due to the construction method, weakness in the mortar joints and lack of connectivity between the wythes, vibrations and shaking induced by earthquake motions would be likely to cause bulging and consequently spalling of the external bluestone veneer, severely affecting the stability and capacity of the wall structure.

4.2 Identified weaknesses

The other structural weaknesses that limited the building were identified as:

- The out-of-plane capacity of the gable end walls
- The out-of-plane capacity of the perimeter walls (including a lack of interlock between the three wythes making up the exterior walls)
- The in-plane shear capacity of the perimeter walls
- Lack of an effective diaphragm and connections to distribute in-plane seismic loads
- Lack of adequate floor-to-wall connection to restrain perimeter walls under out-of-plane actions

5 STRENGTHENING OPTIONS

This section summarises potential in-plane strengthening options, outlining their advantages and limitations in terms of design, structural performance, durability, constructability, heritage impact, and cost. All options rely on consolidation of the perimeter walls and tying them to new plywood-sheathed floor diaphragms to improve seismic performance. Each approach varies in complexity, degree of intervention, and impact on original building fabric, requiring careful consideration of trade-offs between structural benefit and heritage preservation. Whilst generally described in relation to the external walls, similar options were investigated for the internal, load-bearing URM walls. A comparison and summary is provided in Figure 4 below.

5.1 Wall consolidation

Wall consolidation aims to prevent brittle failure and promote cohesive, monolithic behaviour of the masonry walls. Two methodologies were considered:

- Consolidation of the full original wall fabric (three wythes) using lime-based grout injection and wall ties.
- Removal of the inner skin and infill material, consolidating only the external veneer, with distributed anchors connecting the veneer to supplementary strengthening structures.

Consolidating the entire wall thickness is the most sympathetic option from a heritage perspective, preserving authenticity and original fabric. While removal of the inner skin is feasible, it would require extensive temporary works to stabilise floors, roof, and veneer during construction, increasing risk and complexity. This approach would likely compromise internal heritage features during removal and reconstruction.

5.1.1 Site testing

Site testing confirmed the suitability of full-wall consolidation techniques. Lime-based grout (Mapei Antique) was trialled to fill internal voids and improve wall cohesion. Early trials demonstrated feasibility, informed application risks and techniques, and enabled grout volume estimates. Epoxy anchor pull-out testing also confirmed adequate bond strength for tying the three wall layers together using regularly spaced drilled and epoxied anchors.

5.2 Post-tensioning system

This option considered post-tensioning cables installed either internally or externally. Post-tensioning enhances seismic capacity without introducing new lateral load-resisting systems, allowing masonry walls to remain the primary structural elements while improving performance through clamping forces.

The system is expected to provide excellent structural performance, promoting a unified response under earthquake loading and avoiding stiffness or displacement incompatibility. Stainless steel cables, anchorages, and plates minimise corrosion risks and reduce adverse effects on masonry. Installation requires minimal dismantling, supporting efficient and low-impact construction.

Post-earthquake repairability is favourable, as cables can be re-tensioned or replaced with limited intervention. From a heritage perspective, the approach is largely reversible, visually discreet, and represents an honest expression of seismic strengthening.

Although initial material costs may be relatively high, overall cost-effectiveness is achieved by avoiding new foundations, additional structural systems, and extensive propping, scaffolding, or demolition works.

5.3 Steel frames

This option considered both internal and external steel concentrically braced frames (CBFs), such as chevron or K-braced systems. To achieve sufficient stiffness and reduce displacement compatibility issues, frames would likely be designed for nominally ductile or elastic response, requiring relatively large steel sections.

Implementation would involve significant associated works, including strengthening existing floor diaphragms, constructing concrete ring beams to connect frames to the building, and installing new foundations to resist uplift. Foundation works may include reinforced ground beams, anchor piles, and integration with existing foundations.

CBFs are expected to provide good seismic performance; however, some stiffness incompatibility may remain, and limited masonry damage could occur before frames fully engage. Durability concerns are

minimal, but constructability is more complex than other options due to restricted access and the need to install subdivided steel components, potentially extending the construction programme.

Post-earthquake repair is feasible, with new steel elements likely sustaining minor damage while some original fabric may be affected. The solution is reversible from a heritage perspective, though internal frames would have noticeable visual impact. Costs are primarily driven by foundation and installation requirements.

5.4 Concrete or reinforced masonry core

The reinforced concrete (RC) core option introduces a new central seismic-resisting system, fundamentally changing the building's structural behaviour. Lateral loads would be transferred from perimeter walls to the core through rigid floor diaphragms, likely requiring significant floor modifications or replacement with reinforced concrete slabs.

Substantial foundation works would be required to resist overturning forces, potentially including tension anchors and deep foundation systems. Structurally, the RC core is capable of achieving target seismic performance with minimal durability concerns.

Constructability is highly complex, involving extensive temporary works to prop masonry walls, floors, and roof structures while internal elements are dismantled and rebuilt. This would significantly extend construction duration and increase risk.

Post-earthquake repairability is expected to be reasonable; however, heritage impacts are considerable. The intervention is largely irreversible and would result in significant loss of internal fabric. While external visual impacts are limited due to the core's central location, internal heritage values would be substantially affected. Costs are expected to be high due to demolition, reconstruction, new ring beams, and foundation works.

5.5 Shotcrete

Shotcrete strengthening was considered either applied to the internal brick layer or after removing the inner brick and applying it to the bluestone veneer. Challenges arise from incompatibility between cement-based materials and masonry, particularly regarding stiffness and moisture transfer.

Achieving adequate stiffness to be effective would require thick shotcrete layers with double reinforcement due to numerous openings and narrow wall piers. Shotcrete would extend continuously across panels, spandrels, and openings, requiring new foundations and removal or concealment of internal finishes.

Although initial stiffness would be improved, long-term performance is uncertain. Direct contact between shotcrete and clay brick may trap moisture, leading to progressive deterioration, brick disaggregation, and potential separation between materials, reversing the intended load path. These risks cannot be fully mitigated, and concealed masonry would be inaccessible for future repair.

Constructability is complex, requiring staged installation, propping, and partial dismantling of floors. Post-earthquake repairability is poor, and the approach is invasive and irreversible from a heritage perspective. While materials are readily available, life-cycle cost, durability concerns, and extensive temporary works make this option difficult to justify.

5.6 Timber framed structure fixed to a bluestone veneer

A timber-framed strengthening system presents challenges due to differences in material behaviour. Timber is ductile and flexible, requiring larger displacements to engage, whereas masonry is stiff and brittle, creating displacement incompatibility that may lead to masonry damage before timber contributes effectively.

	CONCEPT A Post-Tensioning		CONCEPT B Concentrically Braced Steel Frames		CONCEPT C New RC Structure	CONCEPT D Shotcrete		CONCEPT E Timber Structure
EVALUATION CRITERIA	Option A.1	Option A.2	Option B.1	Option B.2	Option C.1	Option D.1	Option D.2	Option E.1
Design Considerations								
Structural Performance								
Durability								
Constructability								
Post-Earthquake Future Repairs								
Heritage								
Costs Considerations								

KEY SCALE:

- Minor Impact / Efficient Solution
- Modest Impact / Adequate Solution
- Moderate Impact / Acceptable Solution
- Major Impact / Not Achieving Key-Objectives

Technically viable
approaches

Unsuitable
approaches

Figure 4: Key considerations summary table

Structural performance is expected to be limited due to short wall lengths and inter-storey heights. Variable thickness of the bluestone veneer further reduces reliability. While timber durability is not a major concern, constructability is complex, requiring extensive temporary works to stabilise veneer, support floors, and safely remove inner wall layers.

Post-earthquake performance is a significant concern, with potential for severe cracking, permanent deformation, and partial veneer collapse, possibly rendering the building beyond repair. The intervention is invasive and irreversible, negatively affecting heritage fabric.

Although initially appearing economical, the approach is not cost-effective when considering limited structural benefit, extensive temporary works, and potential foundation modifications. Overall, the option offers minimal advantage relative to its structural, heritage, and financial impacts.

6 THE ADOPTED SOLUTION

The adopted solution utilised:

- Exterior walls consolidated with injection of Mapei Antique with M12 SS threaded rod wall ties at 600 crs each way.
- Two storey vertical internal (cored) post-tensioned cables (25mm dia. Macalloy bar) to the northern and southern external wall piers, with horizontal, external post-tensioned cables (4-22mm dia. Macalloy spiral strand wire ropes), at level 1 to the perimeter of the building, anchored with SS brackets. The existing eastern and western walls inherently had >67%NBS in-plane and out-of-plane

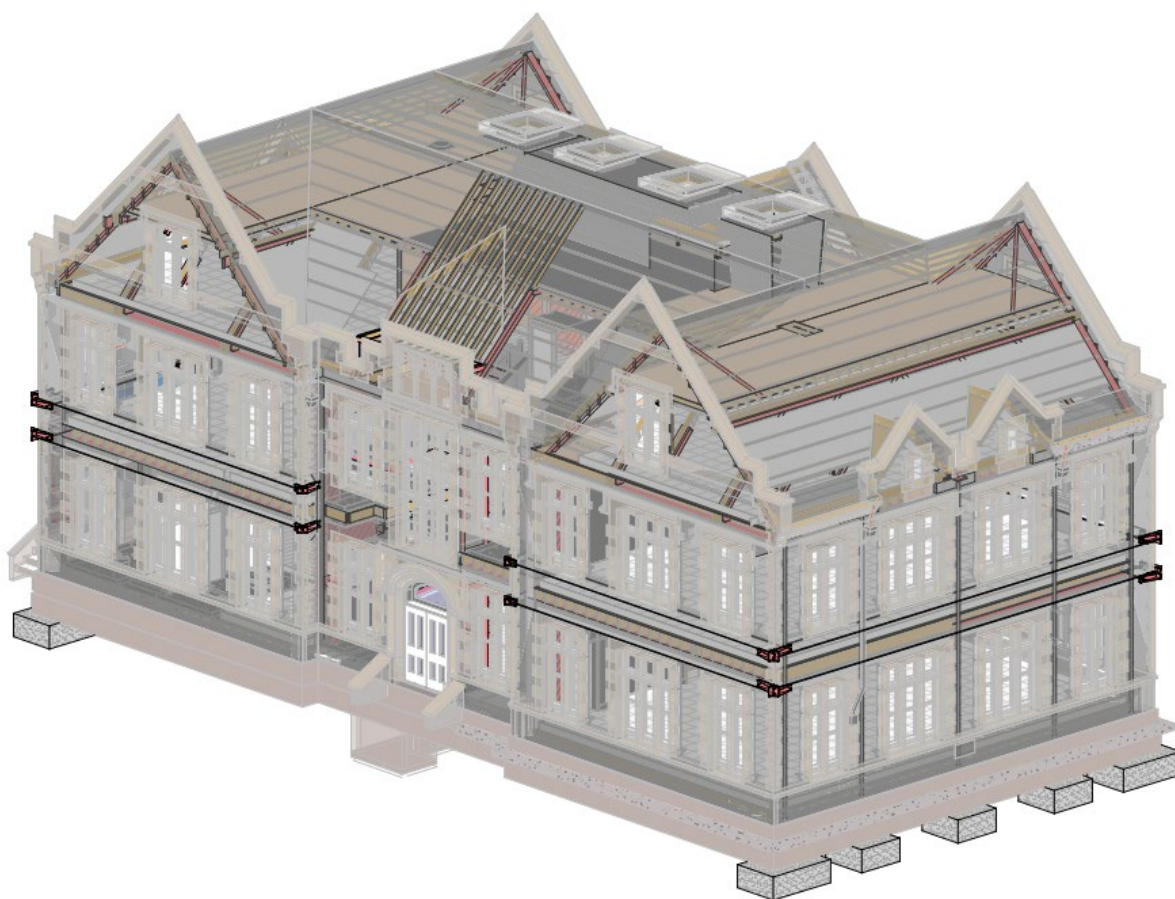


Figure 5: CapSc building northeast view

strength once consolidated and tied into diaphragms at each level with the additional horizontal post tensioning undertaken.

- Replace internal load bearing clay brick walls with reinforced concrete masonry. Strengthening the original internal URM walls with post-tensioning or FRP was investigated but were found to only be able to achieve around 45%NBS. Given the central core location and associated significant tributary area loads and the eventual extent of openings required in the walls for doors and services, the decision to replace these walls was necessary to achieve the desired functionality and performance. (There was some loss of heritage fabric, however these walls were plastered and not noted as of particular heritage significance, when compared to the exterior walls and stairs).
- Plywood diaphragms throughout levels 1, 2 (attic) and attic ceiling with perimeter steel strap drag ties/chords, and interior and exterior wall anchors. The timber floor diaphragms were strengthened using a flexible diaphragm approach considering a sensitivity analysis comparing a defined diaphragm period method with the pESA approach to provide a robust and resilient basis for seismic load distribution. A clear hierarchy of strength was implemented by amplifying diaphragm demands using an overstrength factor of 1.5, resulting in diaphragms designed for 100%NBS elastic actions ($\mu = 1$, $S_p = 1$).
- Steel framing to strengthen the attic trusses and support the stairs and upper-level gables
- Parapets strengthened with vertical rebar drilled and epoxy grouted into the top of the parapets (extending an equal distance below the roof support line (750mm typ.) at 400 crs.

7 CONSTRUCTION

Due to the nature of the proposed seismic strengthening, site environment/climate, and heritage status, the Stage 1 contractor, Amalgamated Builders Ltd, fully scaffolded the building, including the roof and encapsulated it in a plastic wrap. This enabled exterior repairs and all strengthening works to be undertaken without compromising the integrity of the structure and heritage building fabric from environmental exposure, while also removing risks associated with construction programme, site safety, and construction sequencing. The building stayed fully scaffolded through the duration of Stage 1 strengthening, while the roof scaffold and building wrap were removed upon completion of the exterior Stage 1 works.

7.1 Grout consolidation of exterior walls

With initial trial tests having been undertaken as part of the initial strengthening feasibility study, quantities had been confirmed and a construction methodology developed. This involved the successive installation of grout through internal wall face annulus holes at a regular grid on each wall elevation, progressing in lifts due the flowable characteristics of the consolidating material. Instances where grout continued to be injected with no sign of ‘filling’ the zone were encountered, which required continual monitoring from the external elevation and repointing of the external bluestone where voids/discontinuities were identified and the grout was flowing out. The requirement to procure the consolidation product early in the design phase, due to the quantity required and need to import into New Zealand along with the associated construction programme, resulted in the product guarantee being void at the time of installation. This resulted in a need to quantify material properties through testing to confirm chemical and product properties were still within tolerance/acceptable limits with reference to material data sheet properties prior to installation. Upon completion of the grout consolidation and curing, a grid of epoxy grouted M12 stainless steel anchors were installed at 600mm centres to tie the wythes together and act as an equivalent monolithic element, and enabled the post-tensioning strengthening works to be undertaken.

7.2 Post-tensioning

Core drilling of the Northern and Southern Wall elevation piers required meticulous efforts by the subcontractor (Contech) to ensure vertical alignment was maintained along the entire pier height and provide clearance to the vertical tendons. The installation of the post tensioned tendons (vertical) and cables (horizontal) involved initial site setup by the sub-contractor where permanent scaffold was already in place, followed by the installation of tendons/cables, stressing and capping/locking off.

For the vertical cables, the methodology involved a four-step process per pier being an initial slack pull up, full load up, with two final pickups to the prescribed post-tensioned load. For horizontal post-tensioning, it was important to stress the wall spandrels evenly to ensure a balance of load and not induce any significant bending/tension demand in the spandrel. This required each set of four cables on the spandrel to be stressed in sequence to balance the induced compression loads at the extreme fibres of the spandrel itself. Similar to the vertical post tensioning, this involved a sixteen-step process per set involving removing the initial slack, a 50% pull up, 100% pullup, and a final pickup. The physical site set up, installation and stressing were undertaken within one working week for each stressing system (vertical, horizontal), with no significant issues arising during installation. Theoretical elongations of cables/tendons were compared against actual elongations and pressure gauge readings to verify installation capacities, with minimal additional ‘pick ups’ required.



Figure 5: External post-tensioning brackets and cables (Credit: Cook Brothers Construction, University of Otago)

7.3 Internal Walls

Demolition of the existing internal URM walls, temporary propping the existing floors, and installation of new reinforced concrete block masonry walls presented significant challenges with consideration to construction sequencing and site safety.

Prior to demolition of the existing walls, portions of the existing floor sheathing were removed and the floors propped off one another down to ground level. Propping and removal of the existing sheathing was completed at sufficient distance away from the walls to enable safe working room for the demolition and construction of the new wall and foundations. Sequencing involved localised excavation and batter in the vicinity of the basement to enable skin walls to be poured followed by the installation of new, reinforced concrete foundation beams overtop, to form the central 'H' of the building. The new masonry walls were poured in consecutive stages, typically consisting of two lifts per floor, and incorporated dedicated penetrations for doors, and service zones. Only after adequate cure time were the floors re-connected to the new masonry walls and props removed, enabling strengthening of the floor diaphragms to take place.

7.4 Diaphragm strengthening

Construction-phase refinements were undertaken in response to COVID-19-related material constraints and site geometry such as substitution of plywood sheathing and steel ties/straps, while maintaining the approved seismic design intent and resilience objectives.

To enable confidence in the as-built structure, proof load tests were undertaken on all post-installed chemical anchors to verify minimum performance, ensure reliability, and verify the associated load paths and

structural systems could be relied upon as per the original design methodology and intent. In most cases, test results demonstrated compliance, with one instance of an anchor failing prematurely associated with poor/inadequate installation.

Challenges included the need for floor levelling and how this interacted with the proposed diaphragm strengthening, blocking, and tie elements and the requirement to not overdrive screws through the steel tie plates to ensure adequate shear transfer between steel/timber elements.

7.5 Staged construction, strengthening vs fitout

The project was delivered under a constrained budget and programme, requiring a staged delivery approach comprising Stage 1 (Seismic Strengthening) and Stage 2 (Fit-out), each procured through separate tender processes and constructed by different contractors. This necessitated careful management of a broad range of stakeholders and the controlled handover of an active construction site, while ensuring continuity of critical technical information between stages. The extended project duration (2019–2025) further required robust knowledge management to retain and transfer key design concepts, analytical methodologies, and performance objectives, particularly in response to personnel changes within the primary design team. This approach ensured that construction outcomes remained compliant with the original seismic design intent and the principles approved by Heritage New Zealand.

Stage 2 construction, completed by Cook Brothers Construction, required the integration of extensive mechanical, electrical, hydraulic, and fire service penetrations through previously strengthened floor diaphragms and walls, together with the relocation and geometric modification of door openings. These interventions necessitated a review of the Stage 1 seismic analysis models to identify feasible penetration locations and opening configurations, while ensuring that the integrity and performance of the Stage 1 strengthening works were not compromised. This approach preserved the intended seismic load paths and resilience objectives of the original strengthening scheme, though would have been more efficient if they had been incorporated into the original strengthening design.

Early into the Stage 2 works with a new contractor, review of proposed services penetrations revealed that some works had already occurred, including heavy penetration of a masonry pier. Investigation confirmed a post-tensioned vertical tendon had been cut, significantly compromising the pier and reducing the bracing line capacity to about 80%, with overall building capacity reduced to approximately 50–55%NBS. While gravity capacity was also reduced (due to the number of holes), no immediate gravity stability risk was identified. The incident highlighted shortcomings in services coordination, staged construction complexities, and reliance on an uncoordinated BIM model.

8 KEY TAKEAWAYS

Through investigation and development of the seismic strengthening scheme, based on the DSA findings, incorporation of the client fitout requirements, and physical construction stages, key takeaways included:

- The detailed seismic assessment confirmed the building was earthquake-prone (<15%NBS), with resilience limited by poor wall composite action and subsequent in-plane and out-of-plane capacity, inadequate diaphragms, and insufficient wall-to-floor connectivity.
- Early intrusive investigations and material testing were fundamental to understanding actual construction, material variability, load paths, and system behaviour, providing a reliable basis for resilient seismic design. Slight changes in design parameters can have significant effects on the results and potentially change the governing mechanisms

- Strengthening options were evaluated not only for strength and stiffness, but for their ability to deliver robust, compatible, and repairable seismic performance while minimising heritage impact.
- Consolidation of the masonry walls combined with post-tensioning and strengthened timber diaphragms was adopted to promote a unified structural response, controlled damage, and post-event repairability.
- Construction methodologies incorporated testing, verification, and careful sequencing to ensure the intended seismic load paths and resilience objectives were achieved in practice.
- The staged delivery highlighted that seismic resilience is highly dependent on effective coordination, knowledge transfer, and protection of completed strengthening works during subsequent construction stages.
- Ensure the project has adequate budget and programme allowance to enable well-coordinated, robust solutions that support resilient seismic performance and avoid compromising long-term outcomes, as elegant solutions take time to develop and integrate with the existing building fabric.

The authors would like to acknowledge and thank the University of Otago for their strong support of the seismic strengthening works, including their proactive engagement, informed decision-making, and commitment to maintaining the project's seismic resilience objectives.