Seismic Strengthening of Te Whare o Rehua Sarjeant Gallery in Whanganui

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ABSTRACT
Te Whare o Rehua Sarjeant Gallery is one of New Zealand's most important heritage listed buildings. The Category 1 Heritage Building has stood as an iconic symbol at Pukenamu Queens Park in the city of Whanganui since 1919. Originally built as an art gallery, the unreinforced masonry building constructed in Oamaru stone and clay bricks with concrete foundation, was closed in 2014 due to its earthquake prone status.

This paper covers the challenges faced in the now completed seismic strengthening of the 105-year-old building, detailing the structural investigations, liquefaction risk assessment, the strengthening solution and construction issues. The solution adopted to strengthen the building required a balance of engineering and historic preservation, with concealed structural interventions to maintain the building's historic fabric and integrity.

Central to the adopted strengthening scheme was the use of prestressing to provide both out-of-plane and in-plane strength to the existing masonry walls. The walls were tied together in-plane and supported out-of-plane by a new reinforced concrete diaphragm slab at the gallery floor level and capping beams at the roof level. Over 300 stainless steel prestressing bars, up to 12 meters long were installed within vertical holes drilled in the walls, ensuring the structure's strength without compromising its appearance. At the central dome area and the central core, a combination of carbon fibre, reinforced concrete coupling beams and prestressing were used.

This innovative design combined with the building's unique multi-level cruciform plan layout, resulted in an effective, yet visually unobtrusive strengthening solution to this historic building.

1 INTRODUCTION

1.1 Background

The original Sarjeant Gallery constructed in 1919 is recognised as one of New Zealand's most important and iconic heritage buildings. The Sarjeant Gallery art collection has become one of national importance and numbers over 8500 works of New Zealand and international art, spanning 400 years.

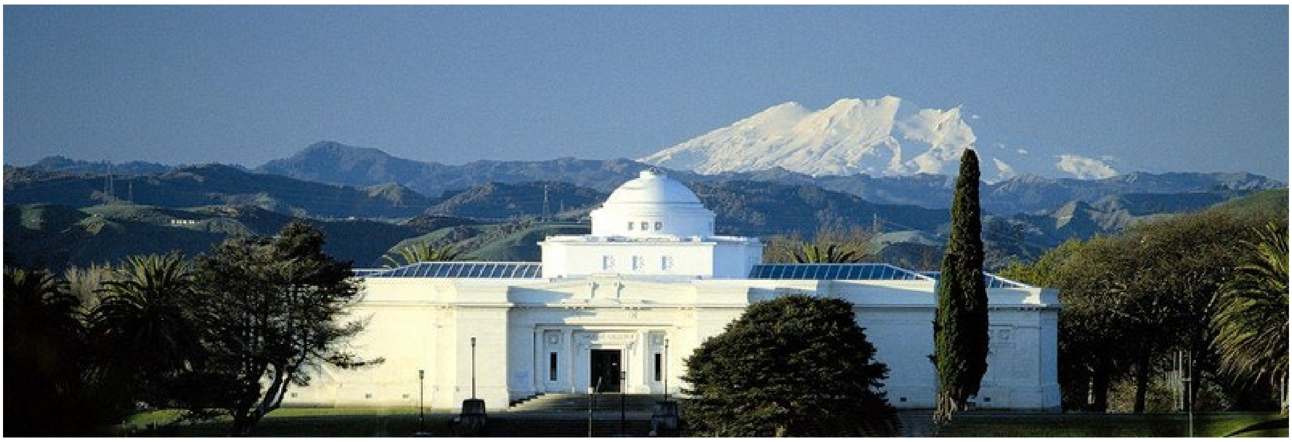

Figure 1: Photo of the original Sarjeant Gallery looking North towards Mt Ruapehu

The Sarjeant Gallery building was closed in 2014 when it was identified as earthquake prone and posed as a severe earthquake risk building. Following on from this from 2014 to 2018, was a period of design and planning for the redevelopment of the site. The Sarjeant Gallery Redevelopment Project led by Warren & Mahoney Architects includes a new northern Extension building connected to the original Sarjeant Gallery by a link bridge but seismically separated.

The construction of the redevelopment project started in 2020 and was completed in 2024. The Sarjeant strengthening design was undertaken to 85%NBS of Importance Level 2 (IL2) or about 67% Importance Level 3 (IL3) seismic loadings to NZS1170.5.

This paper focuses on the seismic strengthening of the original Sarjeant Gallery strengthening.

Figure 2: New Main Entrance to the Redeveloped Sarjeant Gallery with the Strengthened Original Sarjeant Gallery and the New Extension.

1.2 Description of original structure

Figure 3 below shows the main gallery floor and the roof plans. The building measures approximately 39m x 44m in plan and the dome extends to a height of approximately 15m above ground level. The original main entrance is to the South. There is a central core and 4 wings and the building is approximately symmetric about the vertical axis in the plans shown in Figure 3. The foundations of the building step down in several steps across the building from the South to the North, with a total change in elevation of approximately 1.2m.

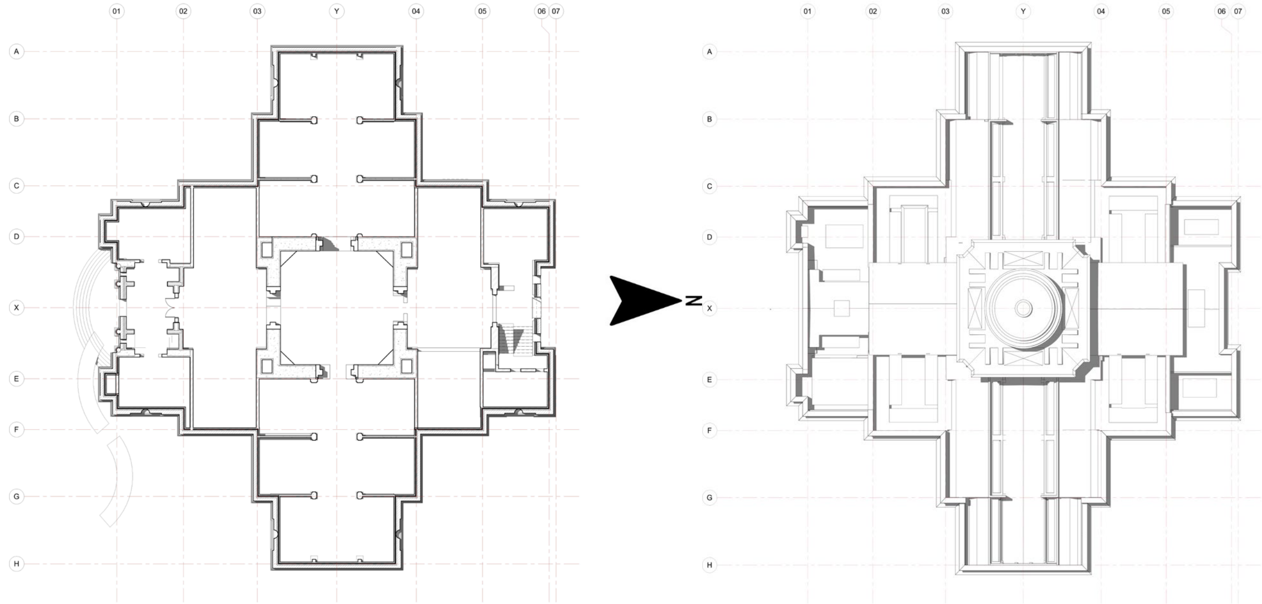

Figure 3: Sarjeant gallery floor plan and roof plan (reproduced from original drawings)

In the North and South wings the internal walls are brick masonry. In the East and West wings, the internal walls are reinforced concrete. The external walls are comprised of an Oamaru Stone exterior, concrete core, brick layer, cavity and then another brick layer on the inside of the cavity. The concrete core at the main gallery level has an internal/external skin of brick which is filled with concrete. The wall layout is generally self-buttressing in cells which provides it with some inherent robustness.

The Sarjeant Gallery roof plan is shown on Figure 3 above. The roof structure has multiple levels and steps and is complicated by the provision of the light diffusion chambers. The dome is located in the centre which can be seen on the East-West section in Figure 4 below. There are a number of skylights which have diffusion chambers below to prevent the light shining directly into the art gallery.

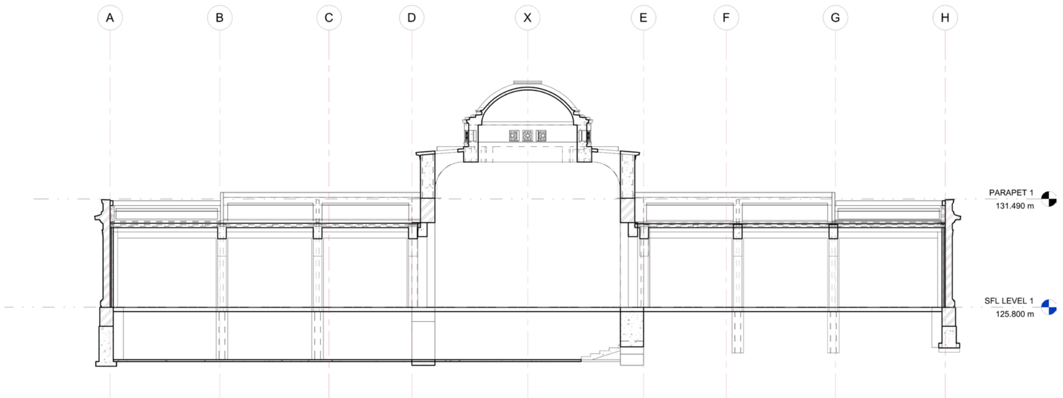

Figure 4: East-West section through building (reproduced from original drawing)

2 FOUNDATIONS

2.1 Site Location and Geology of the site

Alluvial sediments have infilled the Whanganui River valley that was cut into soft sedimentary rocks during the last glacial period. The alluvium was then covered with sand dunes that provide the current topographic relief in the CBD area. The Sarjeant Gallery is located near the crest of one of the oldest dunes in Queens Park, Whanganui (Figure 5). Elevation of the rock surface in the Sarjeant Gallery area is around 30 – 40m below sea level and below the deepest site investigation points (Figure 6).

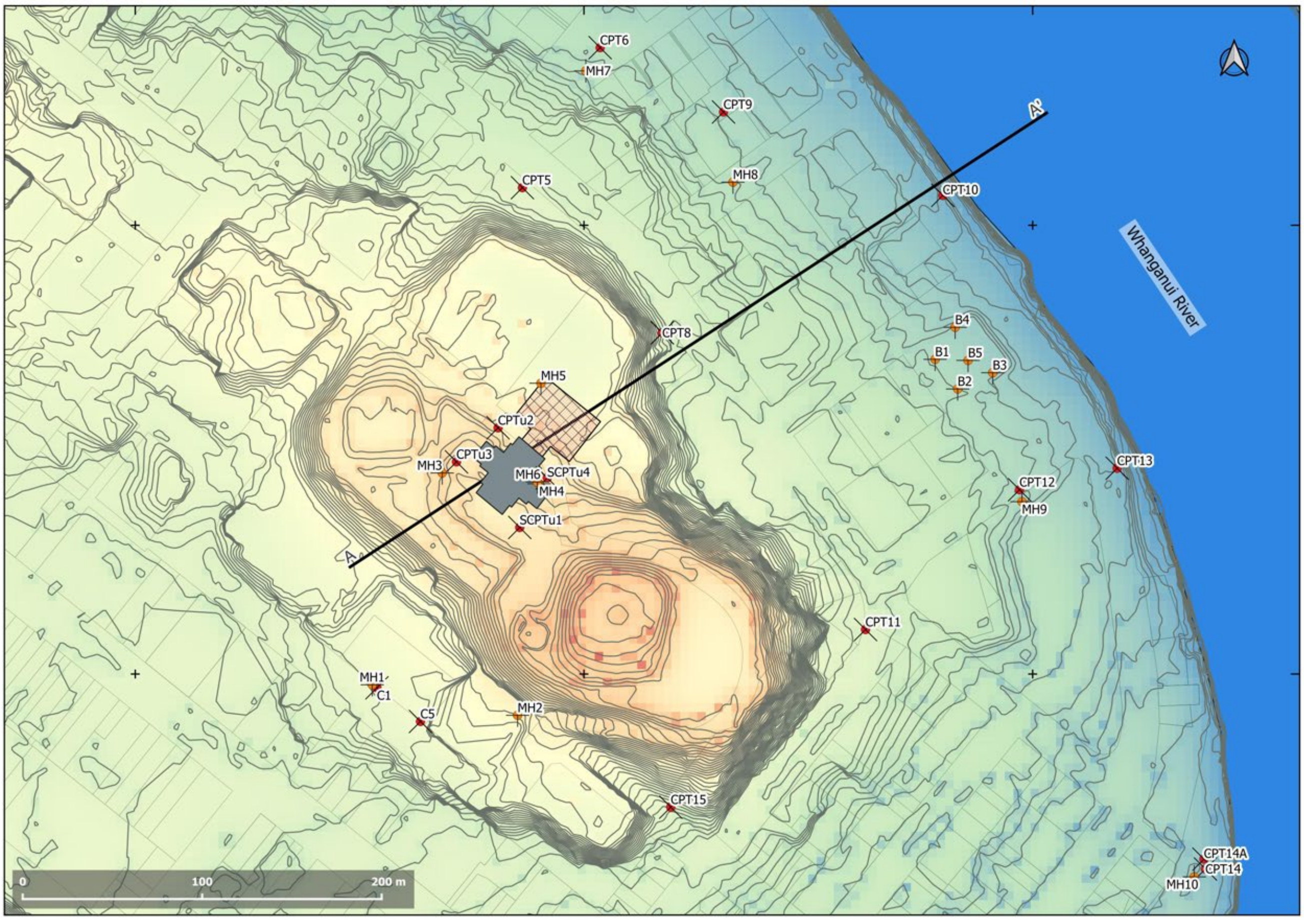

Figure 5: Locations of the site investigation points.

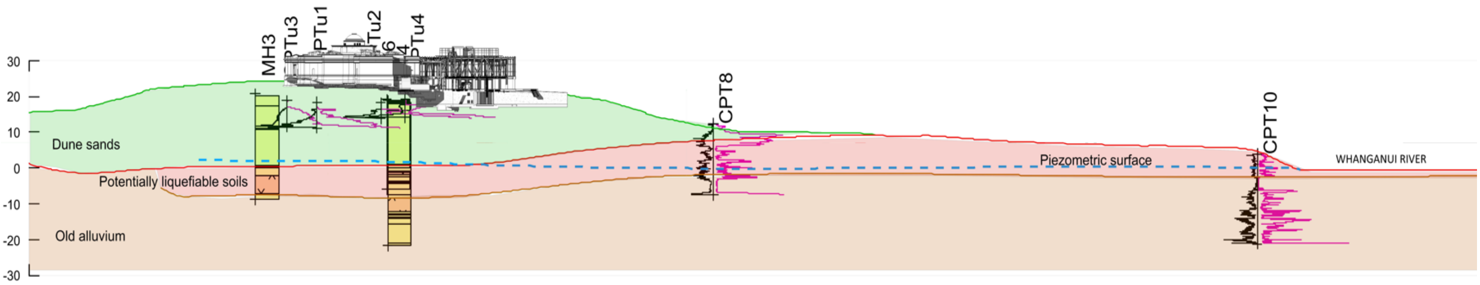

Figure 6: Section A-A'

2.2 Geotechnical Investigations

Geotechnical investigations were carried out by Riley Consultants between 2011 and 2015 and included cored drilled holes and cone penetration tests, refer Figure 5 for the locations of the site investigation points. Riley Consultants identified the site as potentially prone to liquefaction and lateral spread.

Ian R Brown Associates Ltd (IRBA) assisted by Professor Nicholas Sitar (University of California, Berkeley), were engaged in 2018 to undertake a peer review of Riley Consultants' geotechnical report. IRBA concluded that there is a very low risk of liquefaction in the soils beneath the Sarjeant Gallery. This significantly reduced the predicted ground deformation due to liquefaction and improved the design bearing capacity.

The geological setting, age of sediments, as determined from radiocarbon dating of wood recovered from drillholes, discontinuous nature of sandy layers, past seismic history of the area, and depth to potentially liquefiable soils were aspects which informed IRBA's assessment.

2.3 Design of Foundation System

The foundation was designed utilising the interconnected existing and new concrete pads and beams in bearing. Vertical compression only springs were included in the ETABS analysis model to account for local uplifts at some locations of the building and to capture the vertical stiffness of the ground. A sensitivity check was undertaken on the vertical spring stiffness.

The building has a stepped foundation system, and this has been analysed in ETABS by using non-linear horizontal springs which allow sliding once a local base shear capacity is exceeded. This enabled the actual expected distribution of lateral load out into the ground to be determined, therefore capturing the correct load in the lateral load resisting elements.

3 STRUCTURAL INVESTIGATIONS

The following investigations and material testing were undertaken to improve our understanding of the building:

- Trial pits on the existing foundations to determine their depth, extent and condition for post tensioning.
- Cores through existing walls to confirm the make-up of typical walls.
- Reinforcement scanning and strength testing.
- Drilling investigation holes and other minimally intrusive investigations.
- Concrete compression strength testing via cylinder samples and indicative Schmidt hammer tests.
- Masonry in-situ bed-joint shear tests.
- Masonry compression strength, Brick strength and mortar strength testing were undertaken.
- Oamaru stone masonry Compressive strength, Modulus of elasticity and Flexural bond strength testing
- Bond strength testing for anchoring the post-tensioned Macalloy bars.
- A detailed point cloud of the building was captured. This was useful for helping to understand the make-up of the building during both the design, 3D analysis and Revit modelling.

4 STRENGTHENING STRUCTURE BACKGROUND

4.1 Level of Earthquake Strengthening Adopted

Determination of the appropriate seismic strengthening solution and % New Building Standard (%NBS) for the Sarjeant Gallery required structural upgrading objectives to be balanced against the impact on the building's unique heritage features. A key outcome for the strengthening of the existing building was to minimise the impact of the strengthening on the heritage elements. While a %NBS well in excess of the target strength would be desirable, the post-tensioned cable strengthening method uses the existing 100-year-old building materials, which have a limited strength and therefore will pose an upper bound to the achievable %NBS, which is just above 85%NBS (IL2) or about 67%NBS(IL3).

4.2 Adopted strengthening strategy

Central to the adopted strengthening scheme is the use of prestressing to provide both out-of-plane and in-plane strength to the existing masonry walls. The walls were tied together in-plane and supported out-of-plane by a new reinforced concrete diaphragm slab at the gallery floor level and capping beams at the roof level. Over 300 stainless steel prestressing bars, up to 12 meters long were installed within vertical holes drilled in the walls, ensuring the structure's strength without compromising its appearance. At the central dome area and the central core, a combination of carbon fibre, reinforced concrete coupling beams and prestressing were used.

4.3 Alternative Strengthening Systems Considered

Initially, lead-rubber bearing base isolation was an attractive system for strengthening the building because it minimised the seismic loads in the unreinforced masonry and reduced the strengthening and subsequent impact on the heritage fabric. After being taken to preliminary/developed design this was ruled out due to potential challenges with getting a contractor in Whanganui with complex base isolation experience and the risk of cost escalation when implementing base isolation.

5 KEY STRUCTURAL FEATURES AND CONCEPTS IN THE DESIGN

The key structural features and concepts of the strengthening are outlined in this section. There were numerous complex design challenges in the design with this paper providing a brief overview.

The University of Auckland (UOA) were engaged to undertake an engineering peer review for the proposed strengthening design. The peer review was undertaken by Dr Jason Ingham (Professor of Structural Engineering) and Dr Charlotte Toma (Senior Lecturer).

5.1 Central Core and Dome Structure

The reinforced concrete dome was analysed using both first principal hand calculations and a Finite Element model in ETABS. The loads in the dome were determined using a 3D modal response spectrum analysis. The reinforced concrete dome has a diameter of 6m and has adequate capacity for seismic load. The dome's lower ring beam had inadequate tension capacity and was strengthened using carbon fibre around its perimeter.

The load then transfers into the 4 portions of circular drum wall beneath the dome which transfer the load down to the Upper Roof Level. The seismic load from the dome and upper roof is transferred out to the main core lines by the Upper Roof Diaphragm. The Upper roof has large original reinforced concrete beams beneath it to support the upper roof, drum walls and dome. A new 200mm thick reinforced concrete slab was constructed on the top of the upper roof to transfer the seismic load from the dome and upper roof loads to the main core walls and also to increase the strength and stiffness of the upper roof. Refer Figure 7 and Figure 8 below showing the overall strengthening elements for the entire building and also for the central core area.

The main central core has 1.2m thick unreinforced concrete walls with large openings on all four sides. The central core walls were strengthened using post-tensioned Macalloy Bars between the basement wall and the upper roof, as seen in Figure 6 below. The wall and spandrels beneath the perimeter of the Upper Roof were strengthened using 350-millimetre-thick reinforced concrete. This strengthened the walls themselves and provides coupling action between the central core walls. These were also provided with anchorage details for the Macalloy post tensioning. At the main roof level, the core walls went from approximately 1.2m thick to 0.55m thick, enabling the Macalloy bars to be installed approximately central to the main 1.2m thick core walls

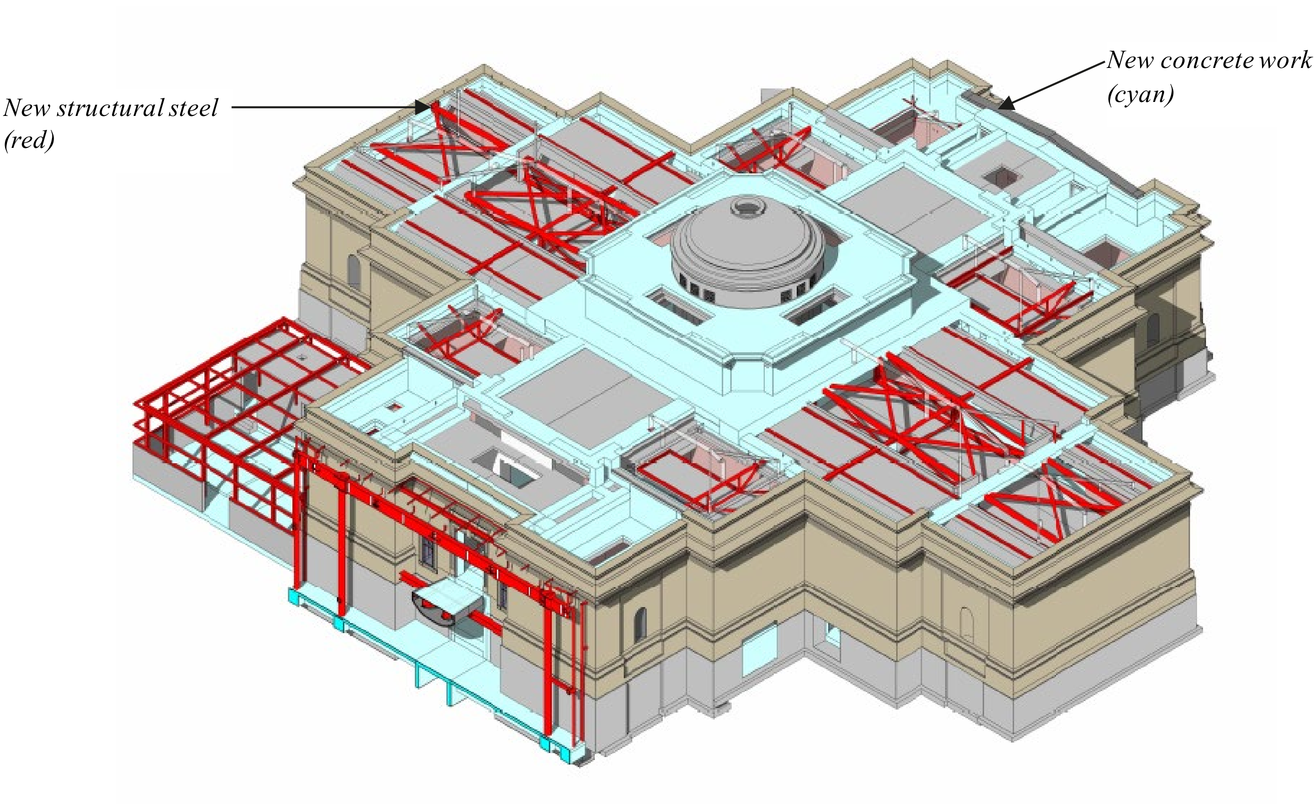

Figure 7: 3D Image of the Overall Strengthening System showing the new structural elements.

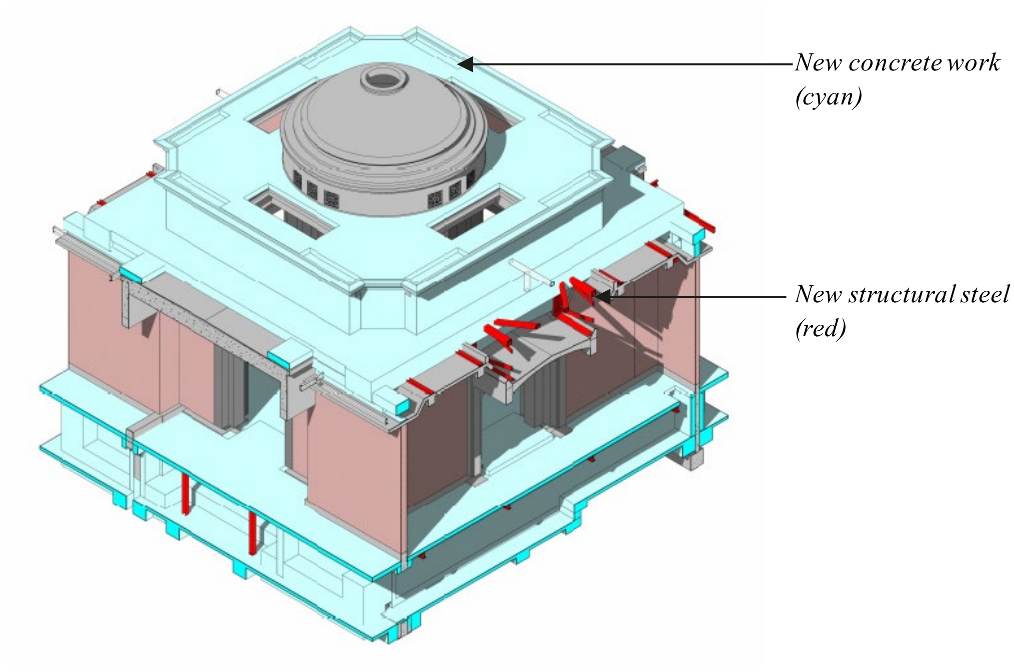

Figure 8: 3D Image of the Overall Strengthening System showing the new structural elements.

New reinforced concrete ground beams were designed on both sides of the main core walls, with reinforced concrete skin walls extending up to the underside of the main gallery floor. The ground beams were designed to couple the main core walls together at ground level to enable the entire core structure to rock in an earthquake. The reinforced concrete skin walls were extended up to the underside of the main gallery floor level to provide a load path for developing the load from the post-tensioned Macalloy bars into the new ground beams.

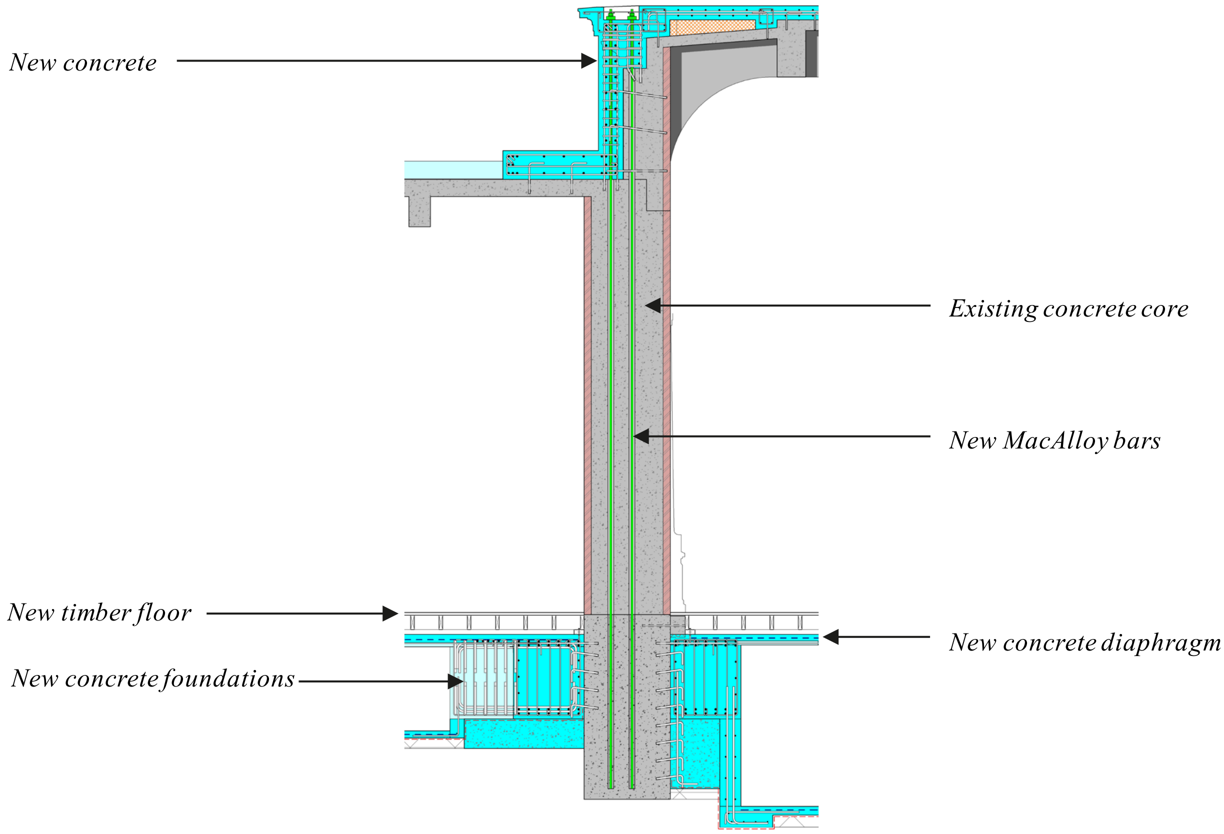

Figure 9: Central Core Strengthening Section

5.2 Main Perimeter Walls

Investigations carried out indicated that the external wall is typically comprised of an external layer and inner layer separated by a cavity. The outer layer is a composite of Oamaru Stone, Grout and Brick. The inner layer comprises of a single wythe thick brick wall, with wires in the bed joints connecting the outer and inner layers of walls.

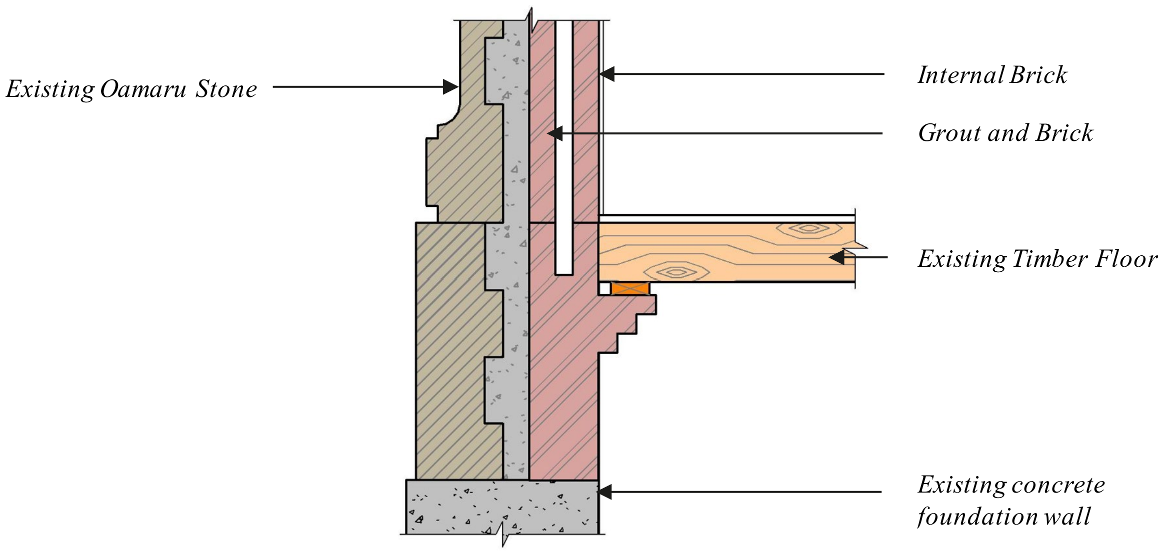

Figure 10: Make-up of External Wall at main floor level

The perimeter walls were strengthened using the following system:

- 32mm diameter Macalloy bars at 1.5m centres post tensioned to 450kN from the roof down to the basement level. These were installed within the main section of the existing perimeter walls.
- Reinforced concrete heading beams at the roof level to anchor the post tensioned elements, support the walls out-of-plane and to act as diaphragm chords.
- On the inside of the perimeter walls at main floor level, stitching was provided between the inner single thickness brick wall and the main 350 millimetres unreinforced masonry wall using approximately 6400 Python C-screws on a 400x400 millimetre grid.

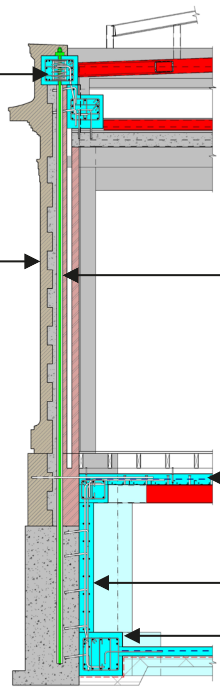

Figure 11: Section through portion of strengthened perimeter wall

5.3 Out-of-plane Design

The out-of-plane design of the post-tensioned masonry walls was undertaken using the following design approaches:

- Using the thesis "Seismic Performance of Unreinforced Masonry walls Retrofitted with Post-Tensioning Tendons" by Daniel Louis Lazzorini.
- Using the MBIE Assessment Guidelines methodology in C8.8.5
- First principles load balancing theory from Post Tensioning.

An analysis on using Python ties for out-of-plane coupling action between in the inner and outer skins was completed. Given the walls were spanning approximately 5.5m out-of-plane vertically it was beyond the realms of testing that had been done at the time. Through first principle approaches it was determined that the python ties may provide 5-10% additional out-of-plane strength. For the strengthening in this case the Python tie coupling action was ignored however they will provide some additional coupling, damping and recentering capability to the walls out-of-plane, in addition to their primary purpose of tying the inner single brick leaf to the main post-tensioned structural wall.

5.4 In-plane Design of Post-Tensioned Walls

The in-plane capacity of walls was calculated using a combination of the MBIE Seismic Assessment Guidelines Section C8 and first principles. The effective weight on the walls was increased due to the post tensioning. Flanges at the ends of walls due to return walls were allowed for at both the compression and tension ends of the walls. At the basement level, the perimeter and internal walls were provided with a reinforced concrete skin. This provided connection between the new reinforced concrete diaphragm and the new ground beams. The new basement walls were also stitched to the existing walls which enabled the in-plane loads in the existing walls to be transferred into the new foundation wall. For in-plane wall loads the new foundation walls, foundation beams and gallery floor beams create a deep beam which are beneficial for coupling walls together and increasing the bending and shear capacity of the original foundation walls.

5.5 Other Factors involved in Post-Tensioning Design

Only stainless-steel Macalloy bars and anchorages were used for durability purposes, negating the need to fully grout the drilled holes. Grouting can be a relatively complex and expensive process with BBR Contech indicating from prior experience that non-stainless grouted bars can end up being as expensive due to issues with grouting.

At the bottom of the walls the bars were anchored into the existing concrete basement wall by grouting them up to the gallery floor level. A testing methodology was developed and implemented to determine how much the masonry and concrete creep under PT loads, enabling PT losses due to creep considered in the design to be verified. The Macalloy bars were tested to 150% of their design loads. The majority of the Macalloy bars used were 32mm diameter and were post tensioned to 450kN. Allowance was made for prestressing losses due to steel relation, and creep and/or shrinkage in the masonry or concrete

5.6 Main Gallery Floor and Diaphragm

The original gallery's timber floor was supported on earthquake prone brick arches. The brick arches were not considered important to the building's heritage so were removed and replaced with a new steel gravity structure. A new concrete floor was introduced on the new steel gravity structure beneath the gallery floor level. This supports the re-instated non-structural timber floor and the gallery's occupancy 4-5 kPa gravity loads, spanning them out to the new steel gravity structure. For heritage reasons the original timber flooring was retained and placed over bearers on the new concrete floor.

The concrete floor also provides a new diaphragm at main gallery floor level, reducing the effective out-of-plane height of the walls and providing robust restraint and load sharing between elements at this level. The diaphragm was designed using the upper bound loads from the Pseudo Equivalent Static Method.

5.7 Main Roof Diaphragm Strengthening

The main roof of the building has a lot of horizontal offsets and vertical steps which required careful consideration during the strengthening. The following elements were introduced and designed for loads obtained from the Pseudo Equivalent Static Method:

- Reinforced concrete beams were constructed at roof level to support the walls out-of-plane and to act as diaphragm chords. A large ring beam was designed around the perimeter of the central core at main roof level to enable some transfer of load between the building's Wings and the Central Core. The new roof diaphragm beams were used for anchoring the post-tensioned Macalloy bars.
- Steel diaphragm ties were installed on the top of some of the roof beams in the East and West wings.
- At the North-East, North-West, South-East and South-West corners of the main roof, new reinforced concrete slab was installed on the top of the existing concrete roof. The inside of the existing parapets had reinforced concrete applied. These elements create a diaphragm where the parapet is taller.

- New steelwork was installed in the light diffusion chambers to provide a lateral load path for the light diffusion chamber slab and to provide some support to the existing roof elements.
- FRP strengthening repairs to the underside and sides of cracked existing roof beams were completed. The new roof diaphragm chords/beams on the existing beam lines had hanger rods installed down into the existing beams. These develop lateral loads out of the existing roof structure into the new diaphragm, act as a secondary gravity support and increase the spandrel shear capacities.

6 TEMPORARY WORKS AND CONSTRUCTION CHALLENGES

During construction it was necessary to develop some innovative temporary works with the contractor. The following temporary works were particularly challenging and interesting:

6.1 Underpinning of walls and staging excavation.

In the Eastern Wing of the Sarjeant, it was necessary to underpin some of the existing walls and columns by approximately 1.3m to deepen the basement. Access was particularly confined, and the underlying material of sand made this challenging.

After several discussions with the contractor, it was decided that a unique form of sheet piling using interlocking 300PFC and a network of waler beams would be used. The rationale for using 300PFC's is that these were able to be carried into the confined space using men. The PFCs were installed into the sand by vibration using modified jack hammers. The interlocking PFC's prevented any sand leakage. The underpinning was staged to minimize the amount of the building which was reliant on the temporary works at the same time.

6.2 Battered excavation at perimeter of the Existing Sarjeant

On the North side of the Sarjeant it was necessary to dig down approximately 4.5m to construct the basement for the new extension building. Due to the relatively high gravity bearing loads (225 kN/m) on the perimeter wall and the strip footings being supported on sand, particular care was necessary to prevent collapse of the building into the excavation and potential excessive settlement causing damage to the existing building.

Through discussions with IRBA, a temporary staged excavation support was developed which involved a reinforced concrete apron, Titan Ischebeck anchors and regular monitoring of the movement. The excavation and supporting apron were undertaken in 3 vertical and 5 horizontal stages to help manage risk.

Figure 12: Temporary anchored wall to support the adjacent basement excavation

6.3 Grout Injection Underpinning

A grout injection underpinning trial was undertaken on site as a possible alternative to 6.1 and 6.2 above. With the grout system used, grout permeation was not achieved in the particular sand tested. Therefore, grout injection underpinning was not adopted.

6.4 Removal of the North Wall Feature and reinstatement at a higher elevation

The Warren Mahoney design has a link that connects the original Sarjeant to the new Northern Extension at the first floor Gallery Level. This link is a steel bridge that spans between the original and the new building, requiring the entrance to be lifted to a new elevation. To allow the existing entrance with the North Wall features to be retained and lifted to the new elevation, required meticulous planning of the temporary works and careful construction sequencing of the permanent works.

6.5 Repair of the Corner Cracks due to Corroded Cast Iron Pipes

At 16 corners of the original building, there were cast iron downpipes within the external wall providing roof drainage which were severely corroded. The corrosion of these cast iron down pipe was the principal cause of vertical cracks at these corners. Steelwork installed providing the temporary horizontal stitching, together with cautious sequencing, allowed the successful removal and replacement of all the cast iron downpipes.

7 CONCLUSIONS

With a variety of structural strengthening arrangements and the unique nature of the original Sarjeant Gallery building, site supervision during construction was demanding and time consuming. The close collaboration between the engineers and the contractor meant that the project was able to be delivered successfully.

This innovative design combined with the building's unique multi-level cruciform plan layout, resulted in an effective, yet visually unobtrusive strengthening solution to the historic and iconic original Sarjeant Gallery building.

8 ACKNOWLEDGEMENTS

We would like to acknowledge; Whanganui District Council who demonstrated great leadership and persistence to make the Sarjeant Gallery Redevelopment project happen and key contributions from the main Contractor McMillan Lockwood. We also thank RCP Project Managers, Warren & Mahoney Architects, the Conservation Architect Chris Cochrane, Nigel Fitch of Riley Consultants with assistance of the liquefaction review, and Jason Ingham & Charlotte Toma from University of Auckland for the structural peer review. The photos in Figure 1 and Figure 2 are courtesy of the Te Whare O Rehua Sarjeant Gallery Web site.

9 REFERENCES

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