NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING
2019 Pacific Conference on Earthquake Engineering
TURNING HAZARD AWARENESS INTO RISK MITIGATION
4 – 6 April | SkyCity, Auckland | New Zealand

The University of Auckland Clock Tower East Wing: turning a heritage building into a modern teaching space

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ABSTRACT

The Clock Tower East Wing is an iconic building for the University of Auckland of historical significance. It was originally constructed between 1923 and 1926 and is listed by Heritage New Zealand as a Historic Place Category 1.

The building was deemed to be potentially earthquake-prone and required seismic strengthening to meet the requirements of the Building Act 2004. An upgrade project was undertaken in 2015-18 to improve the seismic performance of the building to achieve not less than 67%NBS(IL3).

The University's overriding objective was to preserve the building heritage while improving its seismic performance, to reinstate historical features and to reconfigure the spaces to suit a modern university.

The design and construction processes employed were developed through close collaboration between the project and heritage architects, structural engineer, and contractors. The improvement works were successfully completed, utilising many improvement techniques, including post-tensioned walls and an externally bonded FRP system to improve the capacity of the first-floor diaphragm.

This case study addresses several challenges for the design and construction team to overcome without compromising on the heritage aspects of the building and the owner's operational requirements.

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1 INTRODUCTION

1.1 Brief description

The former Arts building (Fig. 1) at the University of Auckland City Campus consists of the Clock Tower main block and the connecting East Wing building to the rear that used to house the Student's Club for Auckland University College Council. These two buildings are interconnected via an open cloister.

Figure 1: Google Earth aerial view and overall site plan

The East Wing building (Fig. 2) is two storeys, has an approximate footprint of 535m^2 and longitudinal and transverse building lengths measuring 35.0m and 19.5m, respectively.

Figure 2: View of the East Wing building from South

The structure comprises a timber-framed/trussed roof and a reinforced concrete first floor with hollow masonry units to form ribs in the profile of the slab that spans between the main concrete beams. The perimeter walls are reinforced concrete, clad in heavy limestone masonry, and are the primary lateral load resisting system for translational and torsional stability. The building is supported on shallow strip footings and pad thickenings under the main walls.

1.2 Key project objectives

As part of the University of Auckland property upgrade plan, a seismic risk review identified that the East Wing building would likely be earthquake-prone and that several areas needed to be upgraded (Marteddu, Rogers, Almeida, Hartley, & Buller, 2018) to comply with the requirements of the Building Act 2004.

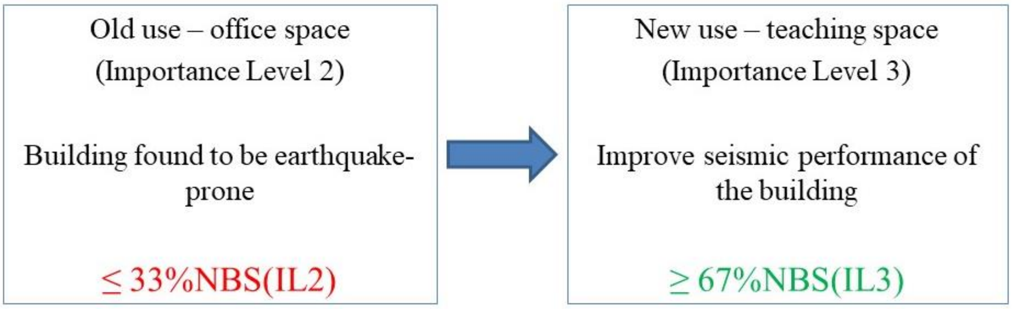


Figure 3: Summary of key project objectives

The University's key objectives (Fig. 3) were to preserve the building's heritage, to reinstate historical features that had been damaged or lost and to reconfigure the space to suit a modern University. From a seismic performance standpoint, the key drivers were to upgrade the building "as nearly as is reasonably practicable" (Building Act 2004, pt 2, s 115) with the Building Code as if it were an equivalent new building.

1.3 Heritage status

The Old Arts building is listed by Heritage New Zealand as a Historic Place Category 1, which is defined as "places of special or outstanding historical or cultural heritage significance or value" (Heritage New Zealand Pouhere Taonga Act 2014, pt 4, s 65).

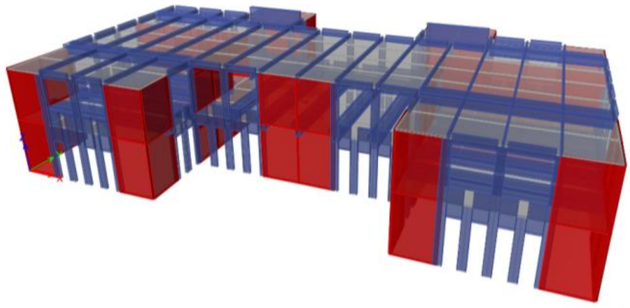
The Arts Building with student wing was originally constructed between 1923 – 1926. The East Wing annexe building is integral to the main Clock Tower building, was constructed in the same style and is of equal historical significance. The building is somewhat unusual for its time in that it has a limestone cladding concealing its poured concrete walls (Heritage New Zealand, n.d.).

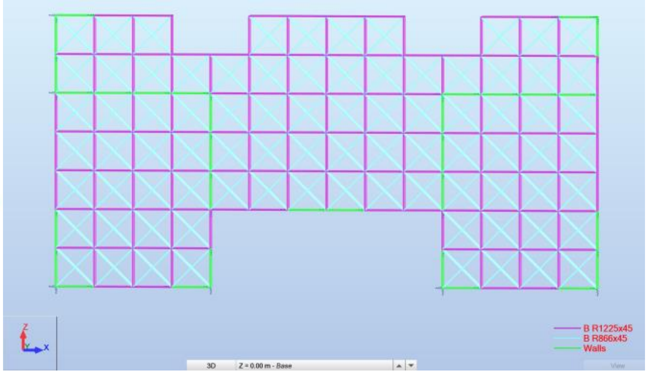
2 ANALYSIS AND DESIGN

2.1 Seismic analysis

The East Wing building structure satisfied the criteria for an equivalent static method of analysis, as defined in NZS 1170.5:2004 (Standards New Zealand, 2016).

A three-dimensional analysis was carried out using ETABS (Fig. 4). This showed that even though new reinforced concrete masonry walls were added to the interior of the building, the exterior concrete piers attracted significant overturning forces which resulted in them failing in bending. If the piers could resist the overturning forces calculated, the building would meet the seismic design criteria. A structural ductility factor of $\mu = 2.0$ was used to determine rocking actions due to earthquakes (Kelly, 2009). Strengthening of the existing shear wall flexural capacity was achieved with bonded post-tensioning tie rods by (1) applying a pre-tension load to resist μ = 2.0 seismic actions and (2) limiting tie rod forces to below yield stress for μ = 1.0 seismic actions. The post-tensioning allowed for pre-compression of the piers so that there was no nett tension under the applied bending moment. It enabled for full use of the compression strength of the existing materials ensuring they made the best possible contribution to the strength of the building. A further benefit was the increase in shear capacity. An analysis with μ = 1.0 has also been used to ensure there were no potential brittle mechanisms such as shear failure in walls.





Figures 4 and 5: Three-dimensional ETABS model of the East Wing building structure (left) and Two-dimensional RSAPRO model of the first-floor diaphragm (right)

Diaphragms were analysed separately using the truss method (Scarry, 2014) to find internal design actions. The floor inertia forces were established using a pseudo-equivalent static analysis method (Standards New Zealand, 2016) and a two-dimensional analysis was carried out using RSAPRO (Fig. 5).

The floor mesh decided upon was 2.450m x 2.450m as this suits the grid. The extent of the floor diaphragm and the positions of shear walls and openings were modelled to the nearest 2.450m. Voids in the slab were modelled as oversized due to where they are located on the grid, which is conservative, and diagonals were defined as compression-only members.

In a slight variation to the truss method, the shear transferred from the diaphragm to each shear wall has been applied as a load, and the inertia forces have been applied uniformly in the opposite direction. The corners of the floor model are supported with some very soft springs to provide nominal stability i.e. to ensure the model is not a mechanism. The springs are soft enough to have very little influence.

The analysis showed that the first-floor diaphragm had inadequate strength in tension and an FRP system was applied to strengthen the slab and reinforce its connection to the perimeter walls.

2.2 Investigations

Extensive physical inspections and some intrusive investigations were carried out during the design phase and included the verification of geotechnical conditions, key elements and details and material testing. These investigations were fundamental to identify differences between the available drawings and what had been built. Additionally, they provided an appropriate degree of confidence in the assumptions that were made. (Marteddu et al., 2018).

3 CONSTRUCTION

3.1 Strengthening with post-tensioned bars

The objective of the strengthening with the post-tensioned bars (Fig. 6) was to increase the in-plane capacity of the existing reinforced concrete shear walls and cloister stone piers (Marteddu et al., 2018).

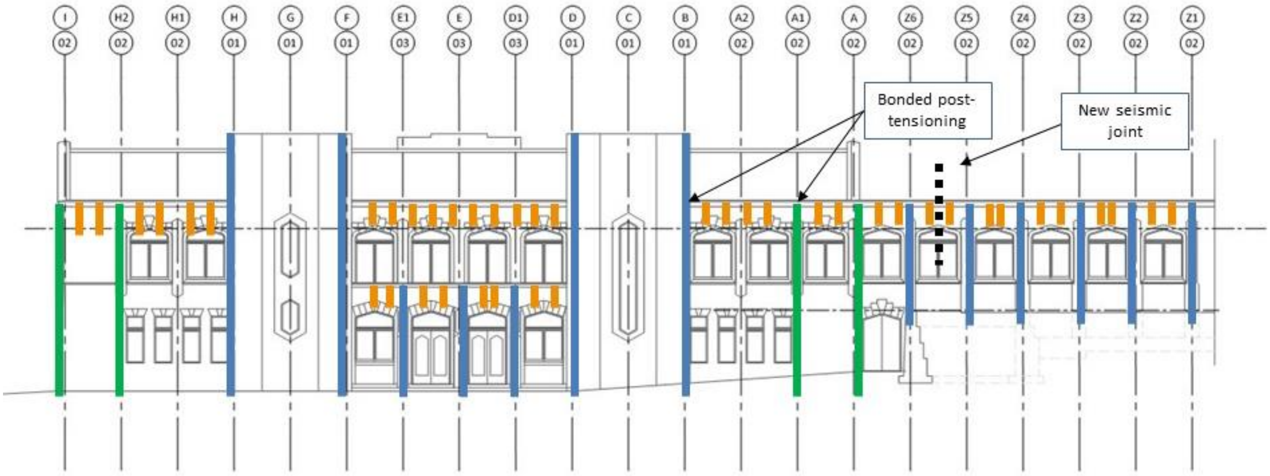
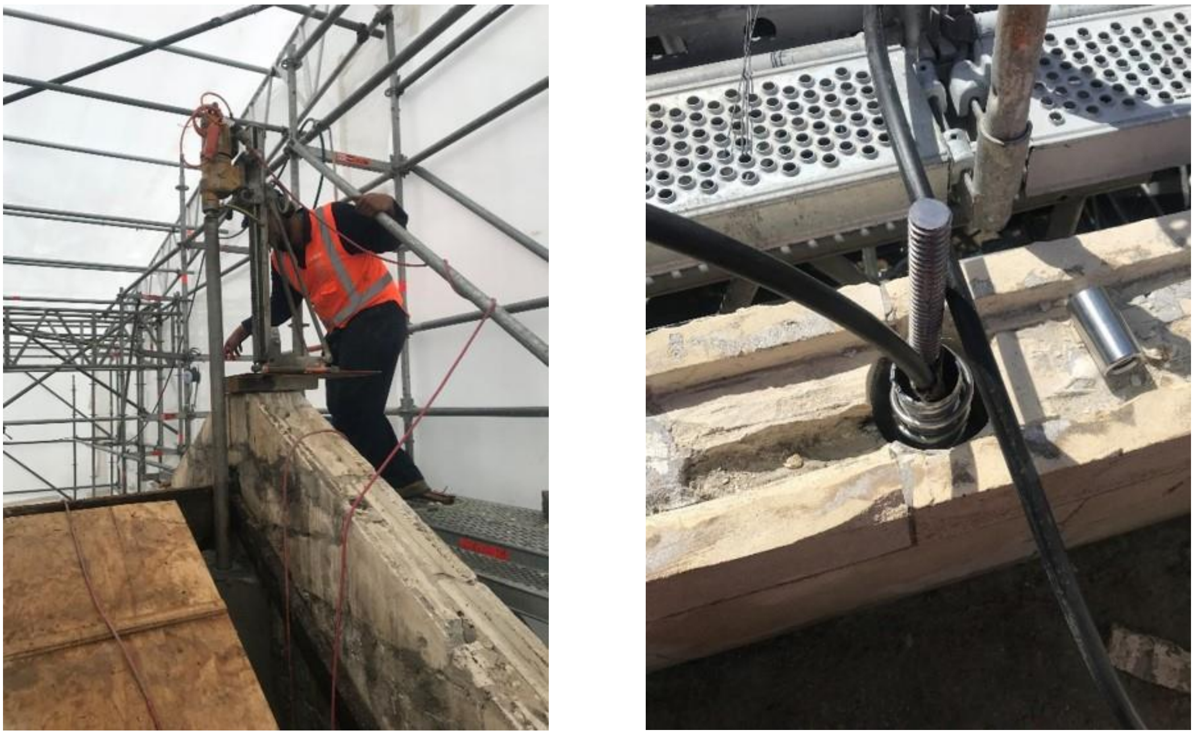


Figure 6: Strengthening with post-tensioned bars on the Southwest elevation

This was achieved through the core drilling of vertical holes down through the existing walls from the top of the parapet to the underside of the existing foundations, up to 12m deep (Fig. 7). The tight tolerance required on the verticality of the holes was achieved by using a longer than usual drilling mast and by securely anchoring this mast to the existing structure prior to commencing drilling. Drilling was carried out using a wet coring process due to restrictions on noise levels during University operating hours.

Typically, 25mm diameter stainless steel Macalloy S1030 bars were used and an initial post-tensioning force of 80kN was applied to the bars. A top anchor plate and an anchor block containing additional spiral reinforcing steel (Fig. 8) were used to controlling the maximum permissible bearing stresses into the unreinforced stone (Ingham, 2011).



Figures 7 and 8: Rig set up for vertical drilling (left) and spiral reinforcing steel (right)

The core cavities were then pressure filled with Mape-Antique I over the core full height. This product was specified because its elastic and mechanical properties are compatible with those of the materials originally used, eliminate the risk of condensation and avoid an alkali-aggregate reaction. Stainless steel bars were used over regular carbon steel for long-term durability since the lime-based products do not provide corrosion protection like cement-based mortars. Geo socks were required as a grout retainer to prevent loss of grout as the existing substrate was extremely porous (Marteddu et al. 2018).

3.2 Strengthening of the first-floor diaphragm using FRP

The existing slab had a very low ability to transfer the required loads due to the very thin concrete topping and a low level of steel reinforcement. Therefore, an externally bonded FRP system was proposed to strengthen the slab and reinforce the connection of the floor to the perimeter walls (Marteddu et al., 2018).

The strength of the existing substrate is an important factor in bond-critical applications of FRP systems (American Concrete Institute, 2008). Careful surface preparation is required to ensure that an adequate adhesive bond between the FRP system and the existing concrete floor is achieved. Prior to construction work, an extensive survey of the first-floor slab was carried out to map existing loose, spalled and cracked concrete, and the substrate was repaired (Fig. 9) to meet the system requirements (American Concrete Institute, 2008). As a result, approximately 8m3 of Sikafloor Level-30, a high performance cementitious, self-levelling and fast drying screed mixed with Sika Pea Metal aggregate was used to reinstate the topping slab to a thickness of 15 to 60mm.

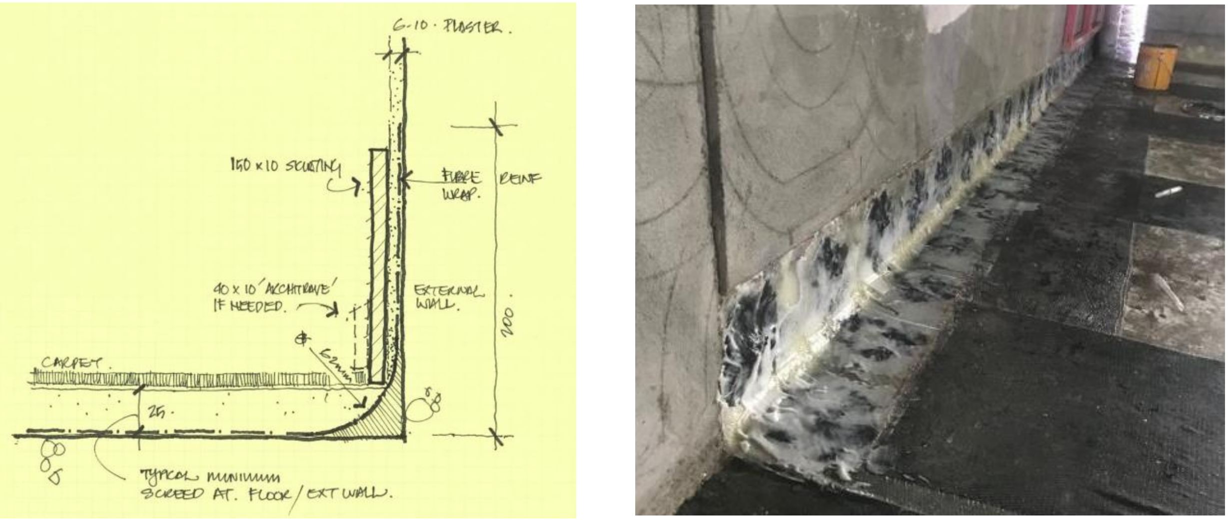
A total of 650m^2 of wet-lay SikaWrap-600 C unidirectional carbon fibre fabric strips was installed on the main slab by BBR Contech (Fig. 10).

Figures 9 and 10: Substrate repair (left) and FRP on-site installation (right)

The connection of the floor to the perimeter walls was also strengthened using FRP strips. This was carried out using FRP curved to a fillet radius of 65-75mm at re-entrant corners around the slab/wall interface (Figures 11 and 12). This prevented stress concentrations and voids between the FRP system and the concrete. Unidirectional glass fibre fabric was installed using 230m^2 of SikaWrap Hex-100G strips and

900No. SikaWrap FX-50 C carbon fibre ropes anchored into the wall to enhance the floor to wall connection and ensure adequate shear transfer.

Close coordination with both project and heritage architects was maintained to integrate the fillet and edge detail into the architectural finishes without any heritage impact (Figures 9 and 10).



Figures 11 and 12: Perimeter skirting detail (left) FRP on-site installation to a fillet radius (right)

A critical aspect of FRP installation is ensuring correct temperature, humidity, and surface moisture at the time of installation. This was achieved by using heat sources to keep the concrete surface temperatures and the ambient air within the required limits.

4 PROJECT LEARNINGS

The building strengthening work is now complete, and we have met our objectives: it complies with Building Act 2004 requirements, the work that has been done is not visible and does not adversely affect the heritage value of the building, and the space that is provided is flexible and meets the current needs of the University.

A true partnership amongst the project team members, involved in the decision-making process and pushing for ideas of reuse and no compromise resulted in a quality building that retains its historical value, yet work for everyone. Because the client's overriding objective was to preserve the building's heritage, we needed to strengthen the building in a manner that concealed our work and didn't leave any sign we had been there. Improvement techniques such as the use of post-tensioning and externally bonded FRP systems enabled us to make full use of the compression strength of the existing materials ensuring that they made the best possible contribution to the seismic strength of the building.

The seismic upgrade work was incredibly complex work that required us to manage the many items of discovery uncovered during the construction process. A key lesson is to never underestimate the importance of carrying out inspections and investigations into the existing structure. These are essential to develop an appropriate degree of confidence in the assumptions that were made or to develop further the design when required.

5 ACKNOWLEDGEMENTS

The authors would like to acknowledge the contributions of Architectus, as project architect; Salmond Reed, as heritage architect; Greenstone Group, as project manager; Argon Construction, as the main contractor; and the wider project team for working collaboratively and finding solutions to ensure the project is a success.

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