Seismic strengthening of a category 1 Heritage Building – St Mary of the Angels Wellington

A.G. Taylor & A.G. Park

Clendon Burns & Park Ltd, Wellington.

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ABSTRACT: Strengthening of this Category 1 Heritage Church building was undertaken following the closure of the building to the public after the series of 2013 Seddon Earthquakes.

Strengthening to as close as practical to 100% IL = 3 was sought. The strengthening utilises a combination of traditional and modern technology techniques to give a robust structure that provides some damage limiting aspects where appropriate for the existing structure.

Strengthening includes concrete insitu shear walls, sprayed concrete shear walls, steel x-braced diaphragms, BRBs (buckling restrained braces), post-tension strengthening, rocking structures with additional energy dissipating devices, and replacement reinforced concrete gravity elements.

1 INTRODUCTION

1.1 Background

This unique purpose built church building was constructed over three years and completed in 1922. The building is believed to be the world's first Gothic design church using reinforced concrete(1). The building has a Category 1 Heritage listing, and is one of many iconic buildings in Wellington(2). The building has two main feature areas; the first is the main church, which consists of the main nave, two side chapels and side vestry rooms. The other is the two towers which are situated at the front of the building, to the Boulcott street frontage, and includes a basement crypt space. See figure 1.

Following the series of 2013 Seddon earthquakes the building was closed to the public so on-going strengthening plans could be fast-tracked and construction undertaken as soon as possible. Due to limited information of the main structure extensive site investigations were undertaken. These included full Geotechnical site investigations, extensive concrete scanning and opening up, point cloud scanning of the interior and exterior faces of the building to give a record of the original building.

Figure 1. Original Street front and internal nave views

1.2 **Previous strengthening**

Previous strengthening schemes were undertaken to the building which included steel braces and concrete columns to the towers, steel nave roof diaphragm trusses, and major window support. Due to the historic listing very little exposed structure could be installed. This made full scale strengthening difficult and costly. However following the 2011 Christchurch earthquakes strengthening to preserve the buildings was deemed more important and more exposed structural options could be explored.

1.3 **Current strengthening strategy**

Strengthening was proposed to bring the building seismic capacity up to as close as practical to 100% IL = 3, and incorporate damage limiting elements where practical(3). The historical category listing required strengthening strategies to have minimal external architectural impact. Various strengthening options were investigated with architectural and engineering collaboration initiated earlier on. See figure 2 showing the main features of the building.

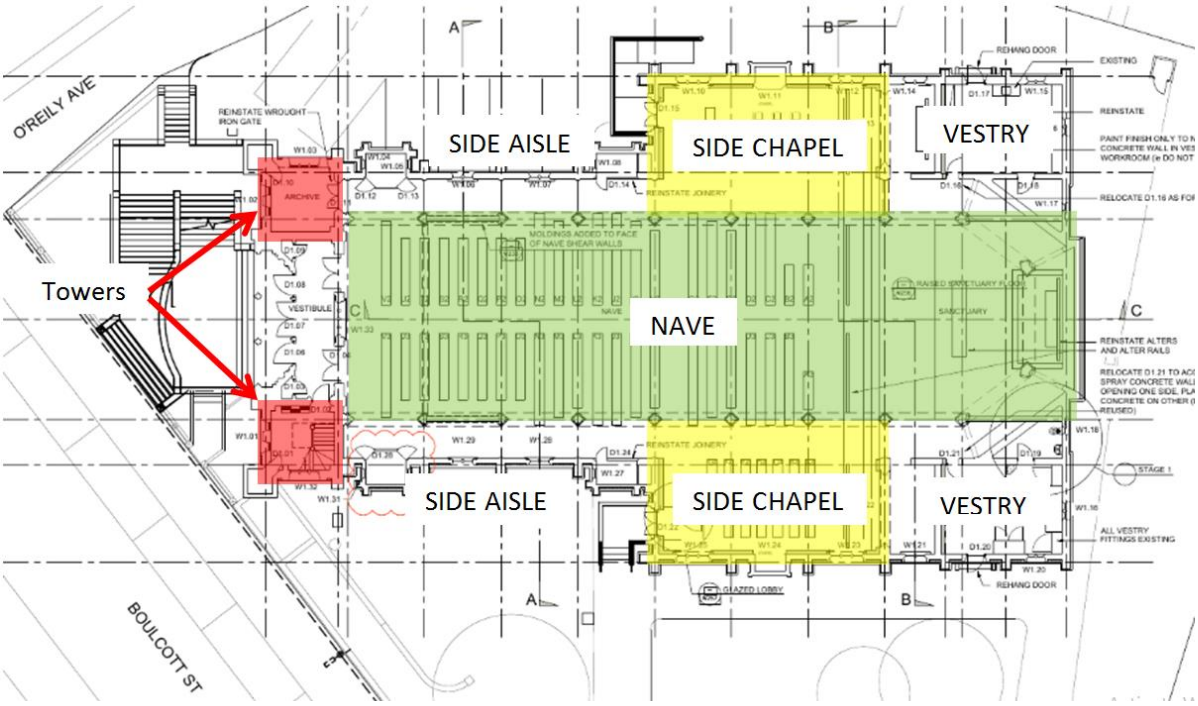

Figure 2. General arrangement

2 FOUNDATIONS

2.1 **Geology of the site**

It was found early on that the founding soil to the building was going to be as unique as the building sitting on top of it. A full geotechnical investigation was undertaken and found that the rear of the church was partial founded on rock, while the rest was founded on overlying alluvium soils(4). The rock naturally slopes towards Willis Street. Two old stream beds, one situated each side of the church were found, along with the dormant Lambton fault line running below the site. Due to the old streams and the presence of ground water liquefaction of the overlaying soils was a potential.

2.2 Strengthening techniques used

The main church building was piled down to the underlying rock using a series of Micro piles, which act in both compression and tension. The micro piles also encountered sheared or shattered rock layers which are present in a fault zone. This type of rock has a lower capacity of bond, to that of more competent rock. Therefore bond lengths were required to be lengthened to give the required capacity. Also a series of large ground beams were installed to help tie the building together over the fault which could experience ‘sympathetic displacements’ during a major Wellington Fault event. Due to the existing crypt level being present, and therefore giving a relatively small distance to the underlying rock, and the rocking nature of the strengthening solution of the towers, traditional mass concrete underpinning of the tower foundations was undertaken. Along with tension only anchors the tower foundations were made to be as rigid as possible, so that a more favourable rocking action of the towers could be obtained.

Lateral resistance of the building was provided by securing to the rock. However due to the middle part of the building being piled with micro piles little lateral resistance could be relied upon, and the end elements of the building were required to resist most of the lateral forces. The front would resist the lateral loads using the tower foundations. The existing foundations had been underpinned down to rock. The rear portion of the building was founded directly into the rock, so lateral resistance could be achieved. Therefore the buildings ground floor slab and the inclusion of new ground beams are required to span the lateral forces out to the resisting end elements.

3 CHURCH

3.1 Church structure

The main church structure consists of reinforced concrete portals to the nave. The portal apex stands approximately 16m internally, from the church floor, with high-level stained glass window clerestory to three sides of the nave, excluding the tower end. Side aisles are present along part of the nave along with openings to two side chapels, on each side, giving more floor area. The external walls are typically 400-600mm thick concrete, reinforced with one layer of ½” bars @ 800 crs, each way. Side chapels have a gable roof forming an internal gutter with the nave. The apex of the side chapel roofs is just above the nave eaves.

3.2 Strengthening techniques used

Strengthening consists of four shear walls, in the longitudinal direction of the nave, with two at the Tower end and two just outside the rear of the nave. Shear walls were chosen to give stiffness compatibility with the existing external walls. The walls extend from new foundations to the nave eave level, so connection into the new roof diaphragm could be achieved. The two front walls pass through the stained glass window clerestory. As such four windows were required to be carefully removed and installed to the inside face of the shear walls. New back lighting was installed to give the appearance of sunlight through the windows. The walls would help reduce displacement of this original flexible clerestory section and provide some protection to the feature windows. The position of the rear walls meant that they penetrate the lower vestry roofs and are exposed to view. As such the form of the wall was to be in keeping with the exterior of the building. A flying buttress arrangement was established which was to help reduce shadowing of sunlight to the stained glass windows to the end of the nave.

In the transverse direction sprayed concrete shear walls were installed to help strengthen the existing walls and give intermediate transverse support to the nave roof diaphragm. However due to the nave and the side chapel walls being connected at a level below the eave the use of prototype tested BRBs were used to transfer loads from the nave roof diaphragm to the side chapel gable end walls. This also gives a ductile behaviour so that overloading of the supporting walls does not take place, under extreme loading. The BRBs are also replaceable if required.

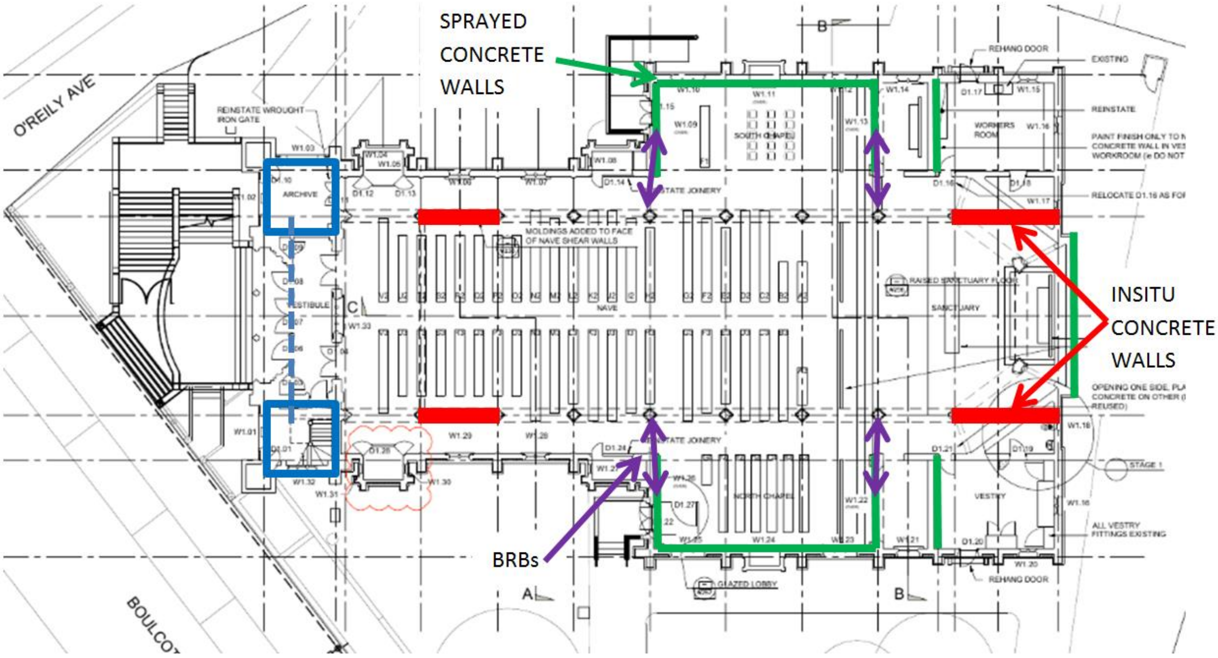

Figure 3. Plan indicating main lateral resisting elements

The existing roof of the nave and side chapels had been replaced with copper roofing supported on a grillage of timber members between the concrete portals. However it was found that the most of the original slate roof was present below copper. This slate was removed to reduce seismic loads to the building and the risk of damage from the tiles. New steel x-braced diaphragms were installed connecting to the buildings eaves, new shear walls and the towers. As the towers were of a much different structural system it was likely that interaction between the two building types would produce damage, and transfer of loads. As such separation of the two was sort as much as practical. However we still required the towers to support the end of the nave roof diaphragm. With this in mind a movement joint was formed with steel UFPs (U-shaped Flexural Plate dissipaters) bridging across the movement joint to help limit load transfer between the buildings.

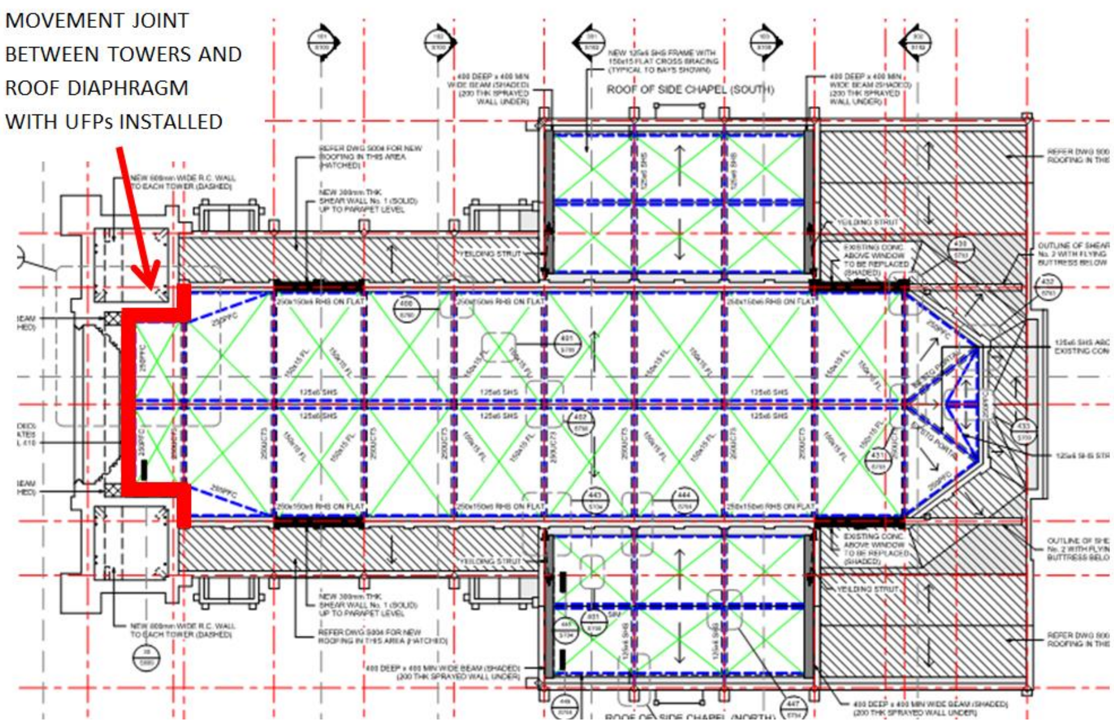

Figure 4. Plan indicating main roof diaphragm elements

As a new lateral resisting systems had been established the existing gravity system also had to be capable of withstanding lateral movement induced actions. A large part of this system was the concrete portals and internal gravity columns. Due to the very nominal amount of reinforcing steel present and the inadequate detailing a risk of damage and collapse was found. As such various strengthening options were investigated, however in the end it was agreed to replace the existing columns and portal knee joints with new insitu elements that had new well detailed reinforcement. The use of the point cloud information gave the contractor a re-usable formwork that took into account the unique column detailing, so that the reinforcing could be maximised, and reduce the amount of finishing works required. The top part of the portals were kept and strengthening with FRP (fibre reinforced polymer) wrap, with connected to the steel roof diaphragm using bolted connections through the depth of the portal. Lateral resistant of the top of the columns and bottom of the portal legs was done using steel x-braced diaphragms to the side aisle and side chapel roofs.

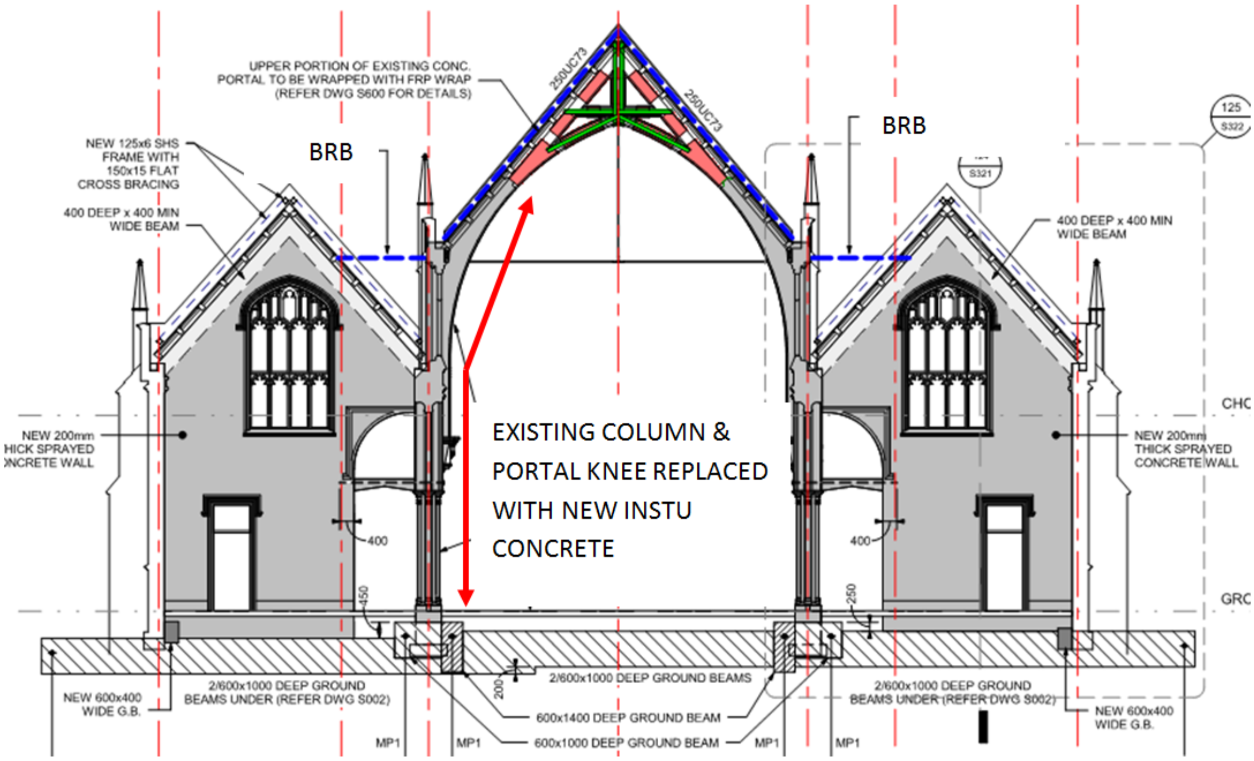

Figure 5. Section through Church at side chapel gable end wall location

4 TOWERS

4.1 Tower structure and existing strengthening

Located to the front of the building the two towers, either side of the main church entry, are in the order of 25-30m high, from the crypt floor, and are only approximately 3.5-4.0m square. The existing foundations were found to be no larger than the tower walls.

4.2 Strengthening techniques used

It was proposed to strengthen the foundations to give a rigid block to which a rocking tower arrangement could be sort. By installing two levels of beams, the bottom to be part of the foundation, the top level to be part of the tower, a cut to the original tower walls gives a rocking plane. To help with energy dissipation plug & play mild steel dissipaters were installed to the centre of the towers, by using a series of cruciform beams. The dissipaters bridge the rocking/cut plane between the two levels of beams, to give energy dissipation that will work in all directions of the tower rocking. The rocking arrangement gives a low-damage structural arrangement for the towers, where the energy dissipation can be replaced if required in the future.

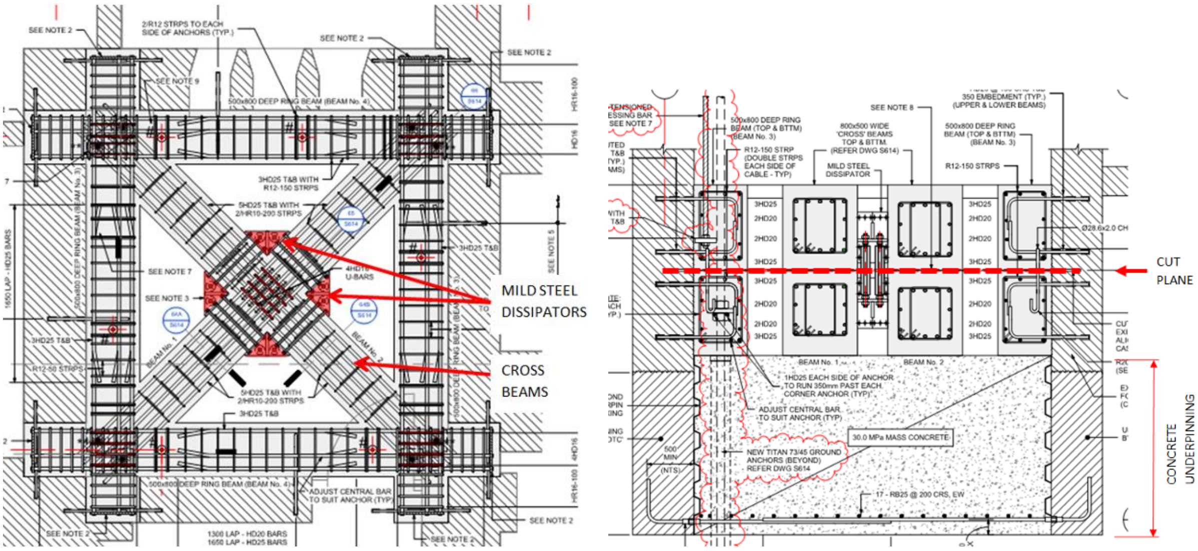

Figure 6. Typical plan and section through tower fouindations

The towers were strengthened by installing post-tensioning stressing bars from the upper foundation beams to the Belfry slab which is situated about 2/3rds up the towers. The additional compression load to the existing walls gives better moment and shear capacity, to overcome the rocking initiation actions. New concrete ring beams have been formed on the inside of the tower walls to transfer loads into the walls from a series of Macalloy stressing bars. The stressing bars span uninterrupted between the ring beams. They are tensioned producing additional internal compression loads to the tower walls.

The top of the towers had steel braces installed as part of the previous strengthening. This was adequate for elastic actions to the top of the towers. As the two towers are separated by the main entry a tie between the towers across the front façade was installed to ensure that the towers would keep in phase with each other and reduce the risk of collapse to the front façade. A tie was located between the two Belfry slabs, with a Macalloy stressing bar tie to the centre of a large SHS strut. The Macalloy bar is designed to take the tension tie loads and the SHS strut will take the compression forces. The SHS was not connected to the towers but has steel bearing plates so that it can rotate without damage, therefore giving a low-damage arrangement.

The front façade between the towers consists of a large rose window within the gable end of the main nave roof. The gable end has two skins of concrete walls. As such the top of the wall skins were tied together with a new steel truss that will restrain the top of the front façade spanning between the towers, along the sloping roof lines.

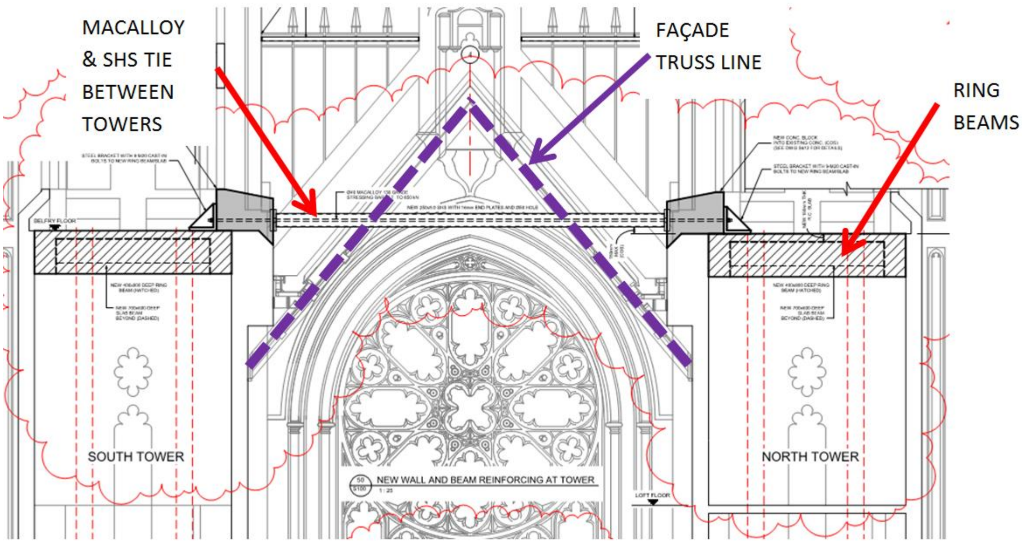

Figure 7. Top of towers and front façade elevation indicating main structural components

5 CONSTRUCTION

With a variety of structural strengthening arrangements and the unique nature of the existing building, site supervision during construction was demanding and time consuming. The close collaboration between senior engineers, the contractor, and peer reviewing/temporary works engineers meant that the project was able to overcome site issues in a timely manner, without compromising the contractor's established construction sequence. The input from the highly experienced and passionate contractor was also invaluable in coming up with solutions to suit the permanent and temporary structural requirements.

5.1 Testing

Testing of the UFPs, BRBs, and tower dissipaters was required to ensure that the performance matched the design. The BRBs were within the limits of the manufacturer's prototype testing criteria so the performance could be predicted very closely. Full scale cyclic testing of the UFPs was undertaken, to ensure that the steel and the bending procedure would not affect the performance of the devices. It was found that test results were within the design limits.

Full scale testing of the tower dissipaters were undertaken with our first batch of units all failing. It was found that the steel procured by the contractor was not actually code compliant. As such a different steel was procured and full scale testing undertaken, with results matching the design parameters.

6 CONCLUSION

Strengthening to this unique iconic Category 1 Historic Wellington Church was undertaken utilising a combination of traditional and modern technology techniques to give a robust structure that provides some damage limiting aspects where appropriate for the existing structure.

7 ACKNOWLEDGEMENTS

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