Stabilisation of the Cathedral of the Blessed Sacrament following the Canterbury earthquakes

by

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Abstract

As a result of the 4 Sept 2010 Darfield earthquake and the more damaging 22 Feb 2011 Christchurch earthquake, considerable damage occurred to a significant number of buildings in Christchurch. The damage that occurred to the Christchurch Roman Catholic Cathedral of the Blessed Sacrament (commonly known as the Christchurch Basilica) as a result of the Canterbury earthquakes is reported, and the observed failure modes are identified. A previous strengthening intervention is outlined and the estimated capacity of the building is discussed. This strengthening was completed in 2004, and addressed the worst aspects of the building's seismic vulnerability. Urgent work was undertaken post-earthquake to secure parts of the building in order to limit damage and prevent collapse of unstable parts of the building. The approach taken for this securing is outlined, and the performance of the building and the previously installed earthquake strengthening intervention is evaluated.

A key consideration throughout the project was the interaction between the structural securing requirements that were driven by the requirement to limit damage and mitigate hazards, and the heritage considerations. Lessons learnt from the strengthening that was carried out, the securing work undertaken, and the approach taken in making the building "safe" are discussed. Some conclusions are drawn with respect to the effectiveness of strengthening similar building types, and the approach taken to secure the building under active seismic conditions.

Keywords
Damage assessment, Failure modes, Risk assessment, Safety, Structural failures

1 INTRODUCTION

The Christchurch Roman Catholic Cathedral of the Blessed Sacrament (commonly known as the Christchurch Basilica) is located at 136 Barbadoes Street, Christchurch, New Zealand, and is listed in the Christchurch City Plan as a Group 1 Protected Building and in the New Zealand Historic Places Trust Register as a Category I Historic Place, Register Number 47 (NZHPT, 2012). The Basilica's first stone was laid in 1901 and construction was completed in 1905 (see Figure 1). The building footprint is approximately 62 m long by 27 m wide, and the top of the first floor walls is 15 m above ground level.

The neo-classical Basilica was designed by renowned church architect F.W. Petre for the bishop at the time, John Joseph Grimes. The Cathedral of the Blessed Sacrament is a renaissance style cathedral and is the most impressive of all the buildings that Petre designed (see Figure 2). The plan is thought to be modelled on the church of Saint Vincent de Paul in Paris, and Petre used the Classical orders for the architectural detail. The building is widely recognised as one of the finest examples of this type of church architecture in Australasia. One of the distinguishing features of the building is the location of the dome. Traditional designs located the dome above the crossing of the nave and transept, but Petre chose to instead locate the dome over the sanctuary (Hamilton, 1986; Wynn-Williams, 1982). The building was intended to be the spiritual home of the Catholic community in the wider Canterbury region at a time when the Catholic Church contributed significantly to society.

The first event in the Canterbury earthquake sequence was the 4 September 2010 M7.1 Darfield earthquake (Gledhill et al. 2011), that caused minor to moderate damage to the Basilica, but the lesser magnitude 22 February 2011 event (Bradley and Cubrinovski, 2011) caused significantly more damage due to the proximity to the site, the shallow depth and the weak underlying soils. The four most significant events in the Canterbury earthquake sequence are illustrated in Figure 3, and were the most damaging of the 10,000-plus aftershocks to occur in the 20 months since the first event on 4 Sept 2010. Following the 4 Sept 2010 earthquake, Opus International Consultants were commissioned to undertake an evaluation of the damage to the building, and since this time have been providing structural engineering and heritage consulting services for the Catholic Diocese.

2 STRUCTURAL FORM

2.1 *Construction*

The Basilica was constructed over four years commencing in 1901, using locally sourced stone. Construction was progressed at a rapid rate, partly due to the use of steam cranes for lifting materials, which reduced the need for scaffolding (see Figures 4 - 6). The structural form is primarily stone used as internal and external masonry leaves that are keyed into concrete that was poured between the leaves. The concrete is of no-fines construction, and has been poured in lifts that coincide with the facing stones (see Figures 7 and 8). This no-fines concrete was used to prevent moisture from being transmitted through to the internal face of the wall. The mortar used between the stones is a cement mortar, and the stones are keyed together using grout-filled cavities called "joggles" which are intended to provide shear keys between blocks. Other relevant features are:

- The roof is lined with Italian-made terracotta roof tiles supported on steel trusses. The domed roofs have copper cladding on native New Zealand Kauri timber framing.

- The walls are generally 500 mm thick with 125 mm thick limestone cladding internally and externally, and no-fines concrete in between. Adjacent courses of limestone are of varying thickness to provide a positive key between the facing and the infill concrete. Some clay brick lining is used in places internally, that is not visible from the main internal spaces.
- The main dome drum is supported on four large arches that spring from the first floor level. These arches have no-fines concrete internally, with some areas of intentional honeycomb construction to save weight. Some steel straps tie the structure together horizontally. The arches positioned below the dome are supported by large piers with an internal spiral staircase.

During construction some settlement occurred beneath the main columns supporting the dome. However, this settlement stabilised and during construction the settlement was accommodated within the structure between the first floor and the roof.

2.2 *Gravity Load Resisting System*

Gravity loads are supported by the walls everywhere except for the internal support of the mezzanine floor and the roof, where Ionic stone columns support the vertical load. The main dome is supported by four substantial arches that spring from the first floor level, which transfer vertical loads into the large columns located between the ground level and first floor.

There is little record of the foundation dimensions. The foundations are thought to be substantial concrete strip footings under the walls, possibly 1.5-2.5 m deep. These footings are understood to step out to over twice the wall thickness at the base.

2.3 *Seismic Load Resisting System*

Global lateral load resistance is generally primarily provided by the many walls of the building (see Figure 9). For lateral loading that is oriented transverse to the principal axis of the building:

- The only mechanism available to resist overturning of the bell towers was the restoring action provided by the mass of the structure above, such that the bell towers are vulnerable to rocking and overturning in a large earthquake.
- Lateral load is transferred by diaphragm action across the nave through the first floor and roof to the walls of the transepts and the walls at the front of the building.
- The main dome resists lateral load through frame action of the reinforced concrete (RC) frame provided by the strengthening at the top window level. Below the windows, the circular drum transfers lateral load through the supporting arches, and the first floor diaphragm distributes the lateral load to the transept walls and to a number of other buttressing walls below first floor level.

For lateral loading that is oriented longitudinally:

- The main dome structure resists lateral load through the arches, and the first floor diaphragm distributes the lateral load to a number of wall elements below the first floor level.

- The nave walls are effectively split into a series of piers due to the presence of windows, but damage patterns indicate that they contribute to the lateral resistance**.** Refer to damage identified in section 6.3.
- The entry wall resists out-of-plane loads by transferring load through the first floor and roof diaphragms to the tower walls.

3 PREVIOUS STRUCTURAL EVALUATION & STRENGTHENING

3.1 *2002 Strengthening Scheme*

A structural strengthening scheme was designed in 2002 to address the key areas of the building that were determined to be most vulnerable to potential earthquake damage. Following installation of the strengthening in 2004 the building was evaluated to have approximately 45% of the lateral strength of an equivalent new building designed in compliance with the seismic design standard at the time (NZS, 1992). However, this evaluation was based on preliminary calculations and engineering judgement only. The strengthening that was installed included (see Figures 10 and 11):

- Strengthening of the top section of the main dome using reinforced concrete ring-beams that were 200 mm x 600 mm deep, fixed to the existing structure above and below the windows, with every second pier strengthened to connect the beams in a similar fashion (see Figure 12(a)).
- Bracing of the top section of the two front bell towers using 32 mm dia. grade 500 MPa steel bracing and reinforced concrete ring-beams that were 200 mm x 600 mm deep above and below the window openings and fixed into the existing structure (see Figure 12(b)).
- A new 100 mm thick reinforced concrete topping at first floor and roof level, tied into the existing floors to provide diaphragm action (see Figure 13). The roof level diaphragm stopped at the end of the nave and did not extend around the main dome or into the transepts.
- Alternating internal columns along the colonnade were post-tensioned and grouted at first floor level, as were the external parapets and ornaments.
- The gable ends of the roof and the transepts and the ornaments and up-stand above the main entry were secured using steel framing.

3.2 *Strengthened Capacity*

The decision was made in 2002 to not carry out detailed calculations for the whole building, as this exercise would have involved significant analysis at some cost. This decision was made in recognition of the squat proportions of the building, and because the building was considered to be capable of easily resisting the minimum legal lateral load requirement of 0.05g (DBH, 1991) for *existing buildings* at the time, and the more conservative 0.1g or 20% of the code which was required by the local authority at that time as the threshold value for definition of an 'earthquake prone building'. Hence, because the building was deemed to not be earthquake prone, there was no legislative necessity to provide further strengthening to the building.

4 EARTHQUAKE SPECTRA COMPARISON

4.1 *Site Response*

A seismograph forming part of the GNS (Institute of Geological and Nuclear Science) network is located at the Cathedral College site immediately adjacent to the Basilica. In Figure 14 the main spectra for the 4 Sept 2010 and 22 Feb 2011 events (GNS 2010, 2011) have been graphed to compare the accelerations from the two events. Also plotted is the line representing 100% of current code requirements (referred to in New Zealand as 'percentage New Building Standard' and written as %NBS) and the estimated capacity of the building following the 2002 strengthening, thought to be 45% of the code requirement at the time of the original report, but which now equates to 33%NBS because following the 22 Feb 2011 event the designated seismicity of the Canterbury region was elevated by a factor of 1.36 to recognise the increased seismic hazard associated with the continuing aftershocks.

4.2 *Building performance*

The earthquake of 22 Feb 2011 was the most damaging of all the earthquakes at this site to date. This magnitude 6.3 event was centred 7 km from the site, at a depth of 5 km, and produced severe shaking across Christchurch city with the maximum recorded horizontal acceleration of 1.41g at the Heathcote Valley site being 1 km from the epicentre (Bradley and Cubrinovski, 2011). As shown in Figure 14, the ground shaking experienced at the site was severe, even though the high acceleration was only for a short period of time. The magnitude of accelerations that the building experienced was significantly greater than the previously evaluated building capacity for a low period structure, and similar to the design level for an Importance Level 3 building (NZS, 2002) designed to current standards. While the period of the main structure is understood to be short at approximately 0.5 seconds, parts of the building such as the towers and the main dome structure had a different response due to vertical irregularity. This irregularity caused these elements to respond independently from the main structure. Damage also caused the period of the building to increase as more deformation occurred and hence stiffness degraded. Observed displacements during deconstruction indicate the main dome had a period in excess of 1.5 seconds in its damaged state.

5 OBSERVED DAMAGE

5.1 *4 September 2010 earthquake*

The first of the Canterbury earthquakes was the M7.1 event that occurred early on 4 Sept 2010 (Gledhill et al. 2011). This event caused some damage to the Basilica, which was generally limited to minor damage to the facing stone. Damage that was noted consisted of minor to moderate cracking to the underside of the first floor diaphragm all around the nave and the main dome, minor cracking and displacement of stone in the west wall of the sacristy, and evidence of out-of-plane movement of the middle column of the north bell tower eastern elevation. Access to the building was restricted following this event until engineers could assess the damage and evaluate the remaining seismic capacity. The main damage can be described as:

- Some evidence of mortar bed joint sliding at the top floor level of both bell towers. The middle column of the north bell tower on the eastern elevation showed evidence of out-of-plane movement.

- The main dome exhibited some mortar bed joint sliding through the window piers.
- Some cracking radiating up from the window arches on each side of the nave, at ground floor.
- Cracking was observed on the underside of the first floor diaphragm all around the nave, and around the main dome. Cracking was also observed on the underside of the first floor external balcony on either side of the nave.
- A link beam between the north transept and the north-west column supporting the main dome had some plaster spall off.
- Minor stone cracking was observed in the west wall of the sacristy.
- Minor cracking of the mosaic floor was observed between columns supporting the main dome.
- Moderate cracking of non-structural masonry walls occurred at the first floor level above the main front entry.

5.2 *26 December 2010 (Boxing Day) earthquake*

The M5.8 event that occurred on 26 December 2010 was centred just to the west of the city and caused little additional damage to the main part of the building, although the two bell towers were damaged. A diagonal failure plane opened up in the south bell tower and to a lesser extent in the north bell tower also. This crack had a width of 15 mm (see Figures 15) and revealed that the towers were vulnerable to collapse in another significant aftershock. As a result, shipping containers were placed to mitigate the hazard that the towers posed to the public, and a design for securing work was carried out. Unfortunately, this securing had not been fully installed at the time of the 22 Feb 2011 event.

5.3 *22 February 2011 earthquake*

The M6.3 event that occurred on 22 Feb 2011 was centred under Lyttelton and caused significant damage to buildings located throughout the city, and also caused significant liquefaction in some areas (Bradley and Cubrinovski, 2011). The Basilica was extensively damaged in this event (see Figure 16). The main damage can be listed as follows:

- Complete collapse of the top section of the north bell tower above the main roof level, and an unstable wedge of the western wall above the first floor level left leaning outward, but still attached. Failure was in a north-west direction.
- Complete collapse of the top section of the south bell tower above the main roof level, and including the entire west wall down to ground floor level. Failure was in a south-west direction. Debris was stopped from reaching the road by the shipping containers that had intentionally been placed in case of this eventuality.
- Minor increase in damage to the main dome at high level. The zinc roof cladding showed signs of twisting distortion. Some timber elements were observed to have dropped out of the dome roof framework.
- Significant damage to the main dome drum, including the entire keystone section of the supporting north arch having dropping out (see Figure 17(a)), and significant cracking to the south arch. The main drum had a residual lateral displacement of approximately 75 mm and there was a noticeable vertical settlement of 100-200 mm.

- Spalling of stone due to rocking compression of the main dome support columns at first floor level. This damage was on the outside of the columns.
- Collapse of a large section of the first floor roof around the main dome on the north side (see Figure 17(b)).
- Rotation of the first floor walls on the east side of the main dome due to movement of the dome structure. The top of the walls displaced outward approximately 150 mm.
- Moderate spalling of the ground floor piers at the top and bottom of the windows on the north and south elevations of the nave. This spalling was thought to be due to in-plane rocking.
- Cracking to the outward facing wall of both transepts at the ground level window arches.
- Moderate shear sliding at the base of the ground floor windows on the north and south elevations at the rear of the building.
- Two ornamental stone crosses toppled, but were kept from falling by the central tie rod that was installed as part of the 2002 seismic strengthening.

Damage was inspection and mapped following this earthquake as shown in Figure 18 and the building was given a red placard. No one entered the building following this event until the "making safe" work had been completed on 6 Oct 2011.

5.4 *13 June 2011 earthquake*

The observed damage from the 13 June 2011 event was limited to a significant block failure plane in the south transept that developed due to movement of the main dome, and delamination of the wall associated with the south nave piers due to significant rocking movement.

6 FAILURE MECHANISMS

6.1 *Collapse Mechanisms*

The failure mechanisms observed for the two bell towers and the transepts were block-type failure mechanisms due to lateral load, characterised by overturning. This failure pattern was previously designated by D'Ayala & Speranza (2002) as mode B2, and other similar failure modes are also known to occur in unreinforced masonry buildings subject to large earthquakes dependent upon the proportions and material properties of the structure. This failure type can be seen in the south tower collapse as shown in the annotated photograph in Figure 19(a) and the post-earthquake condition shown in Figure 19(b).

6.2 *Sliding Failures*

The predominant failure mode throughout the Basilica was that of sliding failure. The propensity for this failure mode was strongly influenced by the geometry of the structure, but also the construction methods used. The weak, no-fines concrete had a compressive strength of only 7 MPa as tested by the extraction of core samples, and the cold joint between pours provided little bond between the aggregate. Due to the relatively squat nature of the building, the sliding failures did not result in collapse, but did cause extensive damage throughout the building due to the resulting movement (see Figure 20). This sliding mechanism will have dissipated energy and lengthened the period of the structure slightly, thus reducing the seismic demand on the building.

6.3 *Nave Piers Failure Mechanisms*

The nave piers suffered significant in-plane damage at both the ground floor level, but also at the first floor level, but no out-of-plane damage was evident from visual observations. The in-plane damage was attributed to 3 mechanisms:

- Rocking mechanism (ground floor piers, see Figure 21(a))
- Sliding mechanism (mainly the first floor piers at sill level)
- Tensile splitting failure (ground floor piers, see Figure 21(b))

The piers that were subjected to a rocking response showed evidence of crushing at the compression toe, and in one case had a corresponding gap opening of 15 mm (see Figure 21(a)). All of the first floor nave piers had suffered sliding failures at the window sill level, and had a 15 mm residual horizontal displacement.

7 SECURING WORKS

7.1 *Heritage considerations*

The objective of the securing work was to mitigate the falling hazards that were presented by the damaged parts of the building, without placing human lives at risk. This work had to consider the heritage status of the building, which impacted the measures taken and the methods used. The primary heritage objective throughout the duration of the "make safe" works was to undertake recording and deconstruction in a manner that would protect and preserve the historic fabric of the building for possible later reconstruction. The structural engineers and heritage consultants had to work together to try and meet both objectives in the safest manner possible.

Emergency make safe works required stabilisation and the removal of falling hazards that included the northern bell tower and the main dome. These works required approval from the NZ Historic Places Trust, Christchurch City Council, Civil Defence and later the Canterbury Earthquake Recovery Authority (CERA) approval and were to become part of a retrospective resource consent application.

7.2 *Urgent Securing*

The approach taken to securing immediately after the 22 Feb 2011 earthquake was complicated by the fact that parts of the building were still extremely unstable in a very active seismic environment. For this reason it was necessary to undertake the initial securing from a distance in order to ensure the safety of the contractors. The solution decided upon was to fix together shipping containers that were craned into position against the building, with sand used for ballast (see Figure 22(a)). Pea-straw hay-bales were used to provide an interface that would not damage the decorative facing stones of the building, which provided an immediate solution that was able to prevent collapse and was stiff enough to control drift. The failure mechanism of the adopted securing system was sliding of the container stack. This method of securing satisfied safety considerations and heritage concerns while achieving a robust structural solution for securing against collapse.

7.3 *Internal Inspections*

As the building was too dangerous to enter following the 22 Feb 2011 event, little was known about the condition of the internal structure. In order to make decisions on how to approach the required securing and deconstruction and the appropriate methods to use, more information was required.

Of particular concern was the main dome, as the main supporting arches had suffered obvious damage. Initially an iPad controlled flying drone (Parrot AR Drone) was used to enter the interior via a broken window and capture video footage of the inside of the building. Using this procedure it was established that some damage had occurred to the main dome support structure that was not apparent from the outside (see Figure 23(a)). The investigation showed that the east and west arches supporting the main dome were largely intact, and that the nave columns were intact and appeared to have suffered little damage.

Following use of the flying drone, a remote-controlled robot was borrowed from the New Zealand Army and used to gain more complete footage of a large part of the inside of the building. The i-Robot PackBot was remotely controlled and it recorded live video footage. The zoom camera had 32x optical zoom which enabled a comprehensive inspection of the internal structure (see Figure 23(b)). This footage was taken just after the 13 June 2011 M6.3 aftershock. The footage showed that the previously intact east and west arches supporting the main dome had suffered moderate damage in the 13 June 2011 event, providing evidence that the structure was degrading with the continuing aftershocks. These observations corresponded to new damage to the transepts in the same event. It was therefore concluded that the main dome was moving significantly in large aftershocks and that the support structure was degrading, thereby causing significant damage to the remainder of the building. Consequently it was concluded that the dome posed a significant hazard which needed to be addressed as soon as possible, given that the frequency and felt intensity of aftershocks was not abating. The dome also posed a fall hazard to an adjacent, undamaged building.

7.4 *Deconstruction*

Because the dome was becoming unstable and causing damage to the remainder of the building in the continuing aftershocks, the decision was made to bring it down. It was determined that the only safe method for deconstruction of the dome was to remove individual sections down to the lower strengthening ringbeam, and then “nibble” down through the drum and arches to the first floor level using a long-reach excavator. Following this work, the remainder of the building was considered to be sufficiently stable to leave until a decision was made on the building’s future. As part of the heritage requirements, a selection of key features were salvaged during this process as representative samples.

The decision to deconstruct the dangerous components of the building, rather than immediately institute securing works analogous to the procedures deployed following the L’Aquila earthquake (see Modena et al., 2011) was based on several factors. Most importantly for this specific case, but also for the majority of damaged heritage building in the Christchurch CBD (see Cairns, 2012 for a review of demolition statistics), was the fact that the building was heavily insured. Obviously the Basilica is the property of the Roman Catholic Diocese of Christchurch and the owner’s immediate concern was to expeditiously make the building safe without endangering human life, and to achieve this goal at reasonable cost.

7.4.1 *Main Dome Removal—Phase I, Stage 1*

Following Opus’ recommendation to CERA, and the subsequent Section 38 (Demolition) notice issued for the building in June 2011, it was proposed that the roof of the main dome structure be removed as the first part of the dome deconstruction. The proposed method (see Figure 24) involved:

1. Removing the top level of stone around the base of the copper dome roof.
2. Fixing steel brackets to each of the twenty-four timber trusses forming the dome roof, and installing temporary fixing of the roof to the structure below.
3. Fixing steel ringbeams to the installed steel brackets.
4. Release the base of the timber trusses and lifting the roof off in one piece using a 400 T mobile crane at a reach of 40 m.

This method was adjusted, however due to safety concerns following the 13 June 2011 aftershock, as it was during this event that the crane was not able to swing the crane basket away from the building as fast as anticipated. This delayed response was partly due to the fact that the crane itself was responding to the ground motions. As a result, the deconstruction method was changed to stripping of the copper cladding, and removal of the roof structure in a piecemeal fashion. The last 8 roof trusses were removed in one section (see Figure 25).

7.4.2 *Main Dome Removal—Phase I, Stage 2*

Following the removal of the main dome roof, the high-level stone ring and the piers between the dome windows were removed. This procedure involved:

1. Cutting the strengthened stone, and removing it in sections (see Figure 26(a))
2. Strapping the piers between the windows and releasing them at their base, then removing them in one piece. The strengthened piers were simple to remove, but the un-strengthened piers had to be strapped, and then lifted from their base to ensure that they held together.
3. Following this work, the decorative internal dome was also able to be successfully removed undamaged (see Figure 26(b)). This element was valuable as a heritage item because of its decorative detailing. The removal of this element was relatively straightforward, as the element was only held in place by the floor joists at the top.

7.4.3 *Main Dome Removal—Phase II*

This stage of the deconstruction consisted of removing both the unstable structure at the rear of the building and the dome support structure, down to first floor level. Securing work carried out prior to starting included buttressing both transepts using stacks of containers to provide restraint up to first floor roof level. A timber truss was installed across the east end of the nave to ensure that the nave roof would be supported upon removal of the main dome. Refer to Figure 27 for a plan showing the extent of work.

The building material was removed using an excavator to “nibble” the structure down to the first floor level. A plan was prepared comprising 20 stages of work to take the remainder of the dome down safely, without compromising its stability. Two arches were left as a standing sample of the decorative stonework. Some stabilisation of these arches will be required should the decision be made to keep them as part of a re-build option. The precarious west wall of the north bell tower was also removed as part of this stage of work. The wall was taken down behind the container stack that had been placed as urgent securing after the 22 Feb 2011 event.

The south nave wall received additional securing in the form of steel beams needled through the wall and counterbalanced to take the weight of the upper level and roof in the event of the “failed” piers at ground level losing their load-carrying capacity in an aftershock. The ground floor wall below was also propped from the outside using heavy-duty shoring props to stop the wall from falling

outwards. The detailed methodology (see Figure 28) for the deconstruction of the rear part of the building, including the dome support structure, is outlined below.

7.4.4 *Initial Securing Work of Main Building*

The damaged north and south transepts were buttressed using containers and pea straw hay bales in a similar fashion to the earlier bell tower securing (see Figure 22(b)). This solution ensured that the end of the nave was secure in any aftershocks or vibration arising during deconstruction. Sand was again used as ballast in the bottom containers. The propping to the ground floor of the south wall was installed to ensure that there was no susceptibility to failure during the proposed work.

A timber truss system was installed at the end of the nave roof to replace the support previously provided by the stone arch supporting the main dome. The truss consisted of a parallel chord truss pushed through from one side of the arch to the other, a triangular truss pushed through at the same level then raised to the underside of the roof rafters, and a horizontal truss to provide lateral restraint. This horizontal truss received a steel universal beam fixed against the side of the main arch bordering the nave to prevent the damaged wall falling through the nave ceiling during deconstruction. Plywood was also placed up against the main arch to prevent any smaller debris from falling inward. Sand was dropped down the centre of the dome drum to protect the mosaic floor from any falling debris, to a thickness of approximately 300 mm.

7.4.5 *Deconstruction of Main Building*

Deconstruction of the main building was undertaken using an excavator with up to 60 m reach, to achieve a safe working distance whilst deconstructing the top of the dome. Deconstruction commenced on the southeast corner of the building and progressed to removal of the eastern wall at first floor level, followed by the removal of the easternmost columns at the rear of the alter. This wall was extremely unstable and presented a hazard to classrooms in the adjacent school building. The diaphragm at first floor level remained intact to provide a lateral load transfer mechanism, and to tie the various elements together at this level. Following this stage, the reinforced concrete ring on the dome was cut on the north side of the building, and the section of the dome drum over the arch keystone was removed.

The wall above the main columns was retained to assist in stabilising the columns under lateral loads that may occur in an aftershock. The wall was then removed as demolition of the dome level progressed further. Demolition of the dome drum continued along the east, then west, and finally north faces (see Figure 28). Once the dome drum was removed the remaining walls along the north and east face were deconstructed.

The first floor walls along the south and north sides of the building around the dome were partially retained as a representative sample of the building, and as much of the ground floor walls were retained as possible. As this work progressed, the building was monitored and re-inspected to assess whether the proposed methodology required amending.

8 PERFORMANCE

It is noted that the short duration of the earthquake assisted in minimising damage, as brittle URM structures such as the Christchurch Basilica suffer substantial degradation that can lead to collapse when they are subjected to longer duration shaking.

8.1 *Strengthened Elements*

In the 22 Feb 2011 event the two bell towers experienced high accelerations, which caused failure of both towers that initiated just below the strengthening at the third level. The strengthening of the towers finished at the 3rd floor level, where the loads were transferred to the existing structure. The failure of the towers occurred where the strengthening stopped, as the accelerations exceeded the shear sliding capacity of the walls, and the existing structure had no tensile capacity to resist overturning forces. The presence of window openings in the wall that resisted the shear sliding also contributed to this failure.

The strengthened floor and roof diaphragms performed well, even though some parts of the diaphragm are showing signs of stress (see Figure 29). The diaphragm was critical to restraining the large columns and arches that supported the main dome, and was effective in spreading the loads into the lateral load resisting walls, especially across the nave. A level survey of the walls has shown the extent of miss-alignment to be little more than expected construction tolerance.

The reinforced concrete strengthening to the top of the main dome performed well. As with the front towers, the strengthening addressed the weakest part of the structure which was around the window openings near the top of the structure. The introduction of this strengthening had the effect of moving the failure point lower down the supporting structure to the main arches. The response of the supporting arches was to rock back and forth, which caused damage but also dissipated energy effectively.

The post-tensioned nave columns showed no signs of damage, apart from some moderate spalling damage to the decorative column capitals due to rotation of the columns. The strengthening of these elements has been effective, but this finding is not unexpected as the columns are much more slender than the wall elements, and therefore have a greater ability to sustain rotation.

It is important to note that the 2004 strengthening was carried out to a budget, and that the intention of the strengthening was to address features of the building that were identified to be most vulnerable in a potential earthquake. This strengthening enabled the building to perform to its expected capacity in the Sept 2010 earthquake, and most of the building performed better than expected in the Feb 2011 event.

8.2 *Unstrengthened Elements*

The behaviour of unstrengthened parts of the building was characterised by sliding shear failures of the walls, rocking of some elements, and block-type failure mechanisms of the transepts and towers. Due to the typical construction having no-fines concrete poured with joints in the same plane as the stone joints, the walls had weak shear resistance. There was little cementitious bond (cohesion) due to the lack of fine particles, and poor aggregate interlock because of the cold joints between concrete pours. Because of this construction method, the walls did not resist shear forces as well as would otherwise have been expected, although this shear sliding mechanism did provide some energy dissipation due to friction, which assisted in preventing the building from suffering global collapse.

The block-type failures were heavily influenced by a combination of factors, including the construction methods, geometry of the structure, and interaction between various parts of the

structure. The existing floors in the towers and the transepts provided little restraint to the substantial wall elements.

8.3 *Securing Methods*

The use of shipping containers as propping to the front tower and both transepts proved effective in the 13 June and 23 December 2011 events of magnitude 6.4 and 6.0 respectively. No movement of the containers was observed following these events, and only minor movement of the south transept was observed following the 23 December event. This movement was due to some compression of the pea straw hay bales.

9 CONCLUSIONS

The earthquake strengthening previously performed on the Christchurch Basilica managed to protect life safety with only a moderate level of investment. However, this strengthening was not sufficient to protect an important heritage building from extensive damage in a significant earthquake or prevent progressive damage from aftershocks, and this performance illustrated three main lessons for strengthening of large buildings of this type:

1. Strengthening that ties the building together, such as a well-connected floor diaphragm, is the most effective solution to improve the global performance of the structure, assuming that the loads can be adequately transferred from the diaphragm to the perimeter walls.
2. Addressing the weakest element, such as the top section of a tower, may elevate the failure strength to a limited extent, but continuity of strengthening is required in order to prevent global collapse from occurring. Preferably this strengthening needs to be achieved using a ductile mechanism.
3. Stiffness compatibility of strengthening solutions is critical to ensure that the strengthened structure responds as intended under the design loads.

Securing a significant heritage building to protect against damage progression in anticipation of a significant number of damaging aftershocks is challenging, as heritage requirements must be balanced against the risk to human life. Sound engineering judgement and innovative solutions implemented pro-actively are required to achieve a successful outcome and this challenge is best approached in stages, with risks re-assessed at each stage to incorporate any change in the environment or increased understanding of the building. A collaborative peer review process is advisable to achieve this outcome, and a staged approach to any securing and deconstruction is recommended as the understanding of the building in its damaged state will change as more detail is uncovered. The final securing carried out is likely to be far different to the original plan due to such an uncertain seismic environment.

Less conventional securing solutions such as shipping containers can be used as a means of securing URM buildings without placing human lives at risk, and can be implemented in a very short time-frame. This is an effective solution, especially where large lateral loads are required to be resisted at a significant height.

Remote internal inspections are a useful way of gathering information on the condition of the internal structure. Even if a complex robot such as the Army i-Robot PackBot device used are not available, innovative, cheaper solutions can be found, such as the iPad controlled Parrot AR drone used on the Christchurch Basilica.

In order to ensure that significant buildings such as the Christchurch Basilica are retained for future generations, extensive earthquake strengthening is required. This requirement is difficult to achieve without affecting the building appearance, unless the building can be base-isolated.

10 ACKNOWLEDGEMENTS

The authors wish to thank the Roman Catholic Diocese of Christchurch for allowing publication of the information reported herein, and for allowing use of the images presented in Figures 1, 2, 4, 5 and 6. Grant Wilkinson, Director of Ruamoko Solutions, is thanked for providing valuable peer review assistance throughout this project and the important contributions of the heritage consultants Carole-Lynne Kerrigan (Opus International Consultants Ltd) and David Young, an independent heritage consultant from Victoria, Australia, are gratefully acknowledged.

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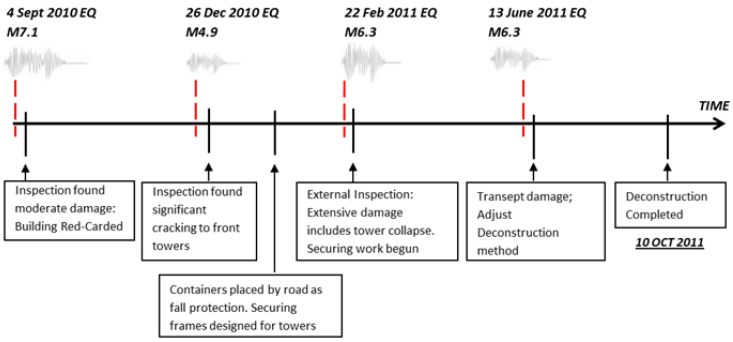
Figure 2. South elevation of the Christchurch Basilica prior to the Canterbury earthquakes (photo courtesy of the Roman Catholic Diocese of Christchurch)



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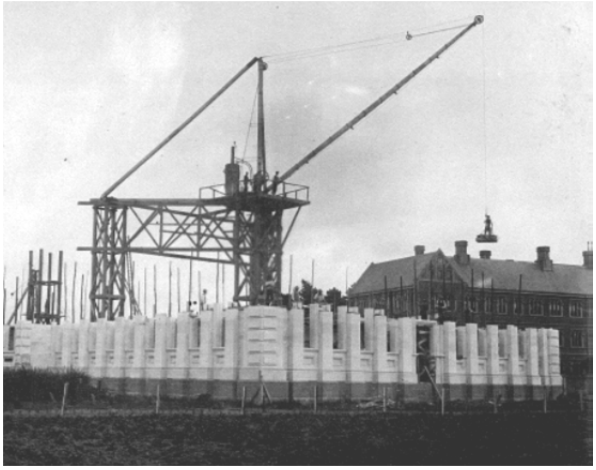

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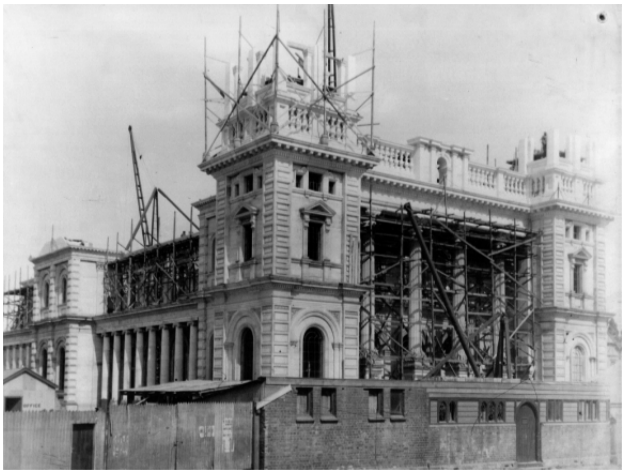

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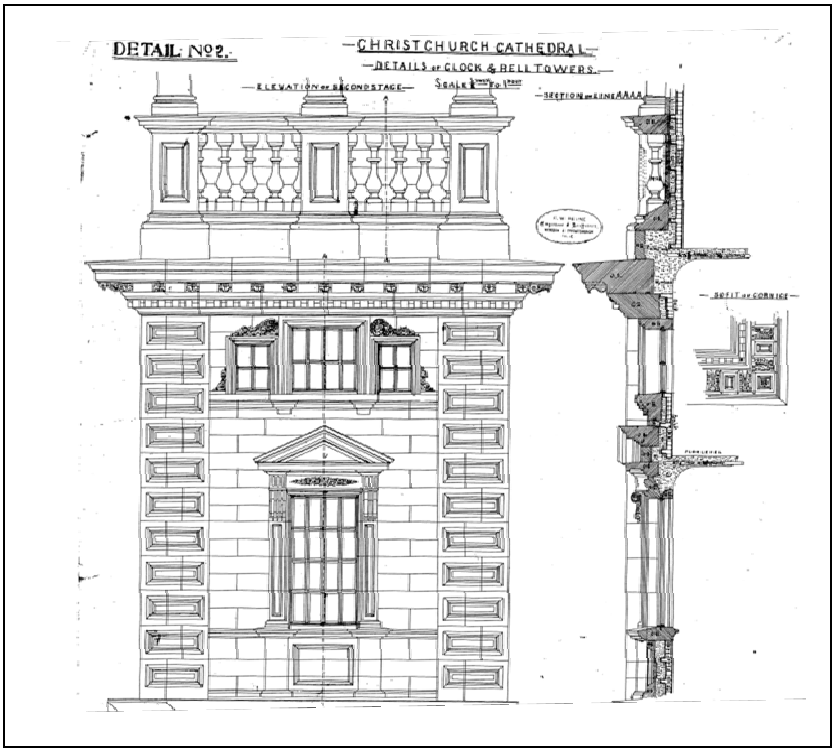

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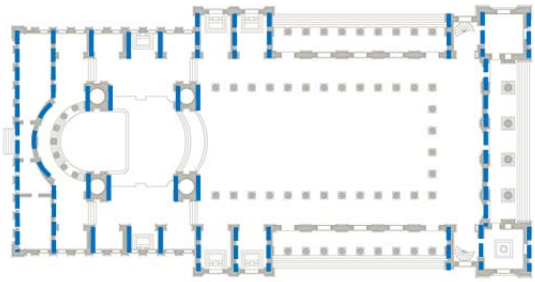
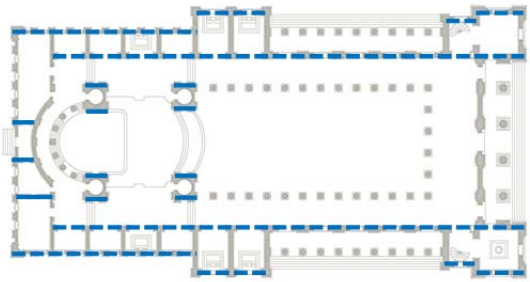
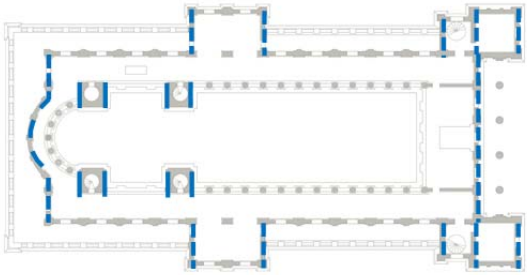


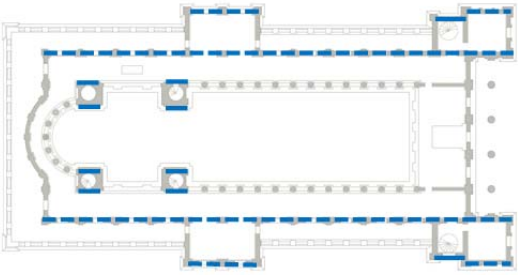
(a) Location of walls in the transverse direction (ground floor)



(b) Location of walls in the longitudinal direction (ground floor)



(c) Location of walls in the transverse direction (first floor)



(d) Location of walls in the longitudinal direction (first floor)

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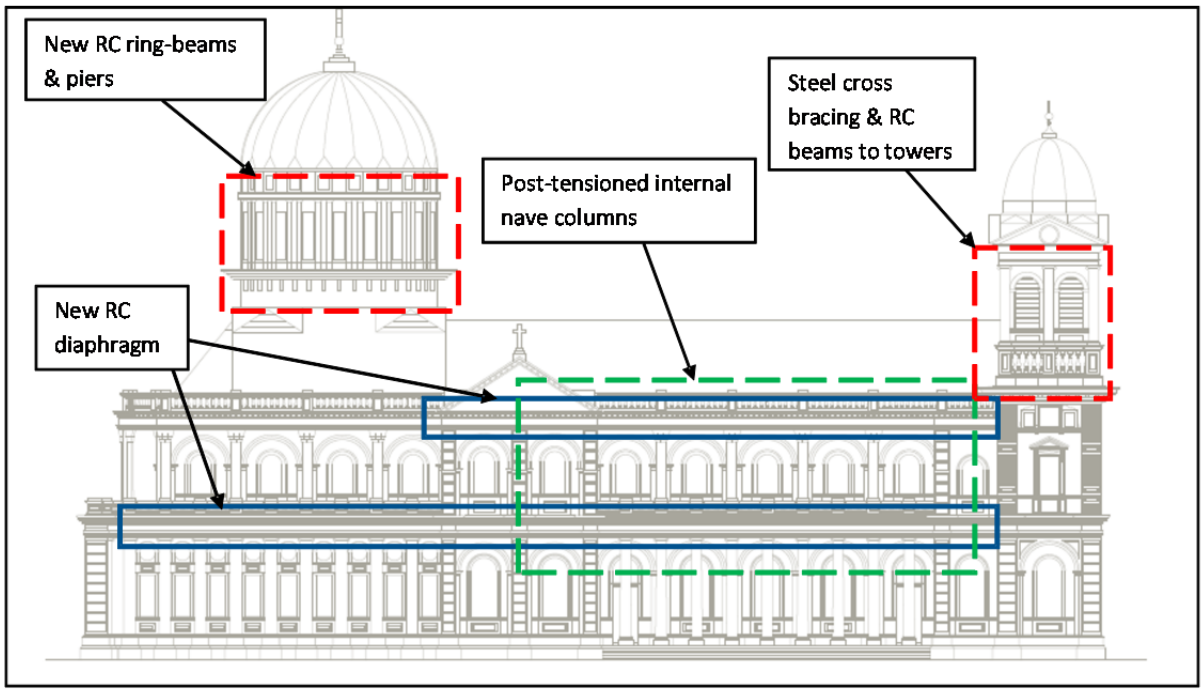

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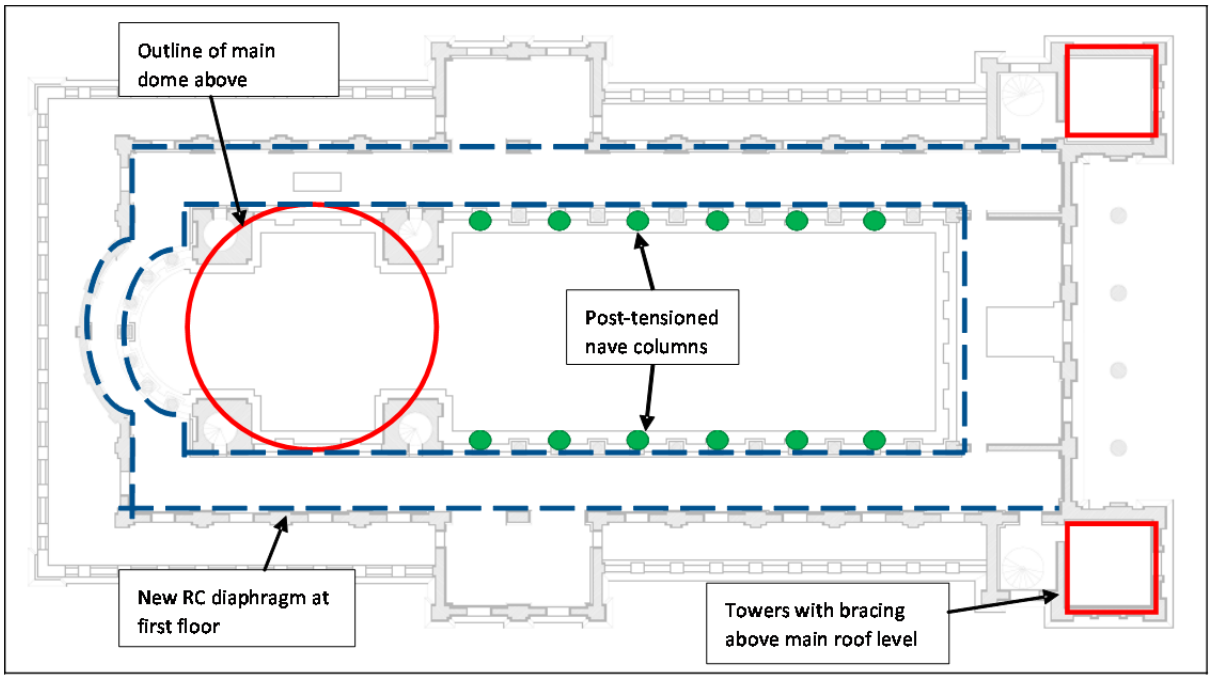

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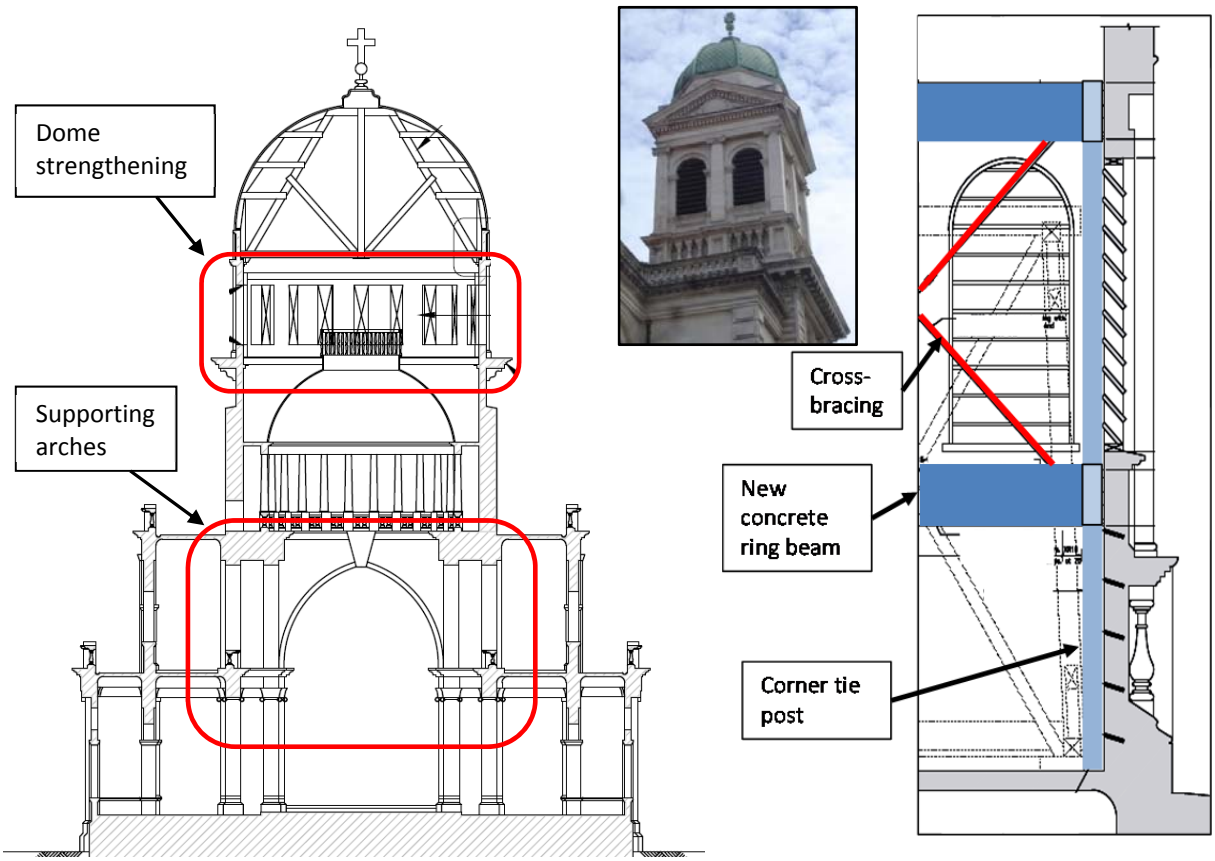

(a) Section through main dome

(b) Strengthening to bell towers

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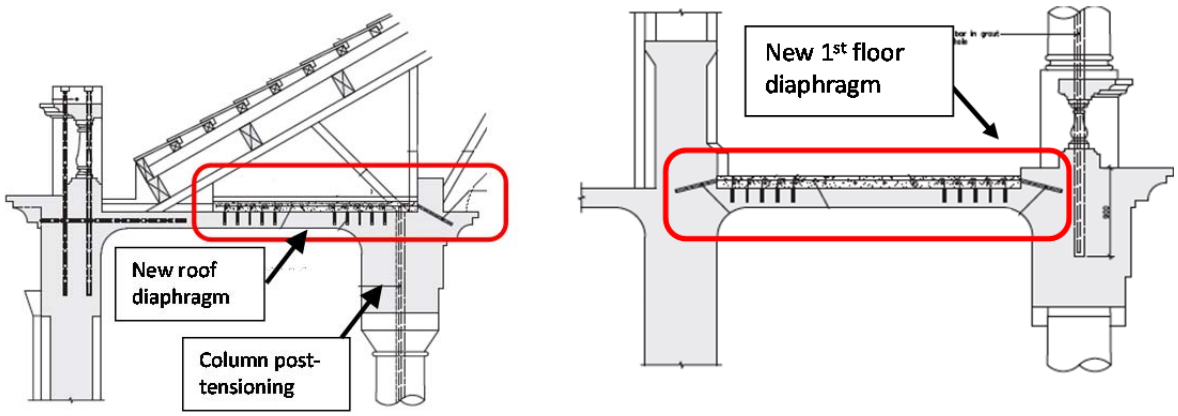

(a) New roof diaphragm

(b) New 1st floor diaphragm

Figure 13. Details of diaphragm strengthening

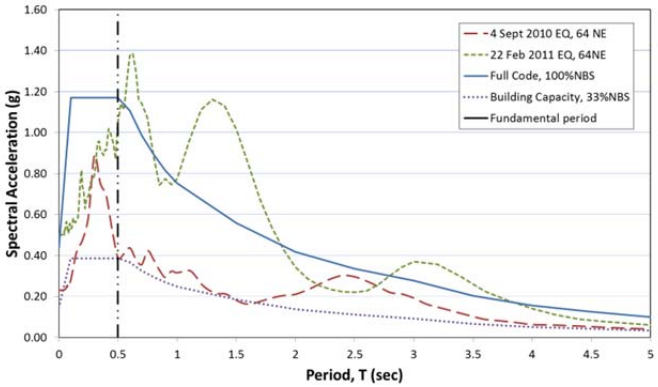

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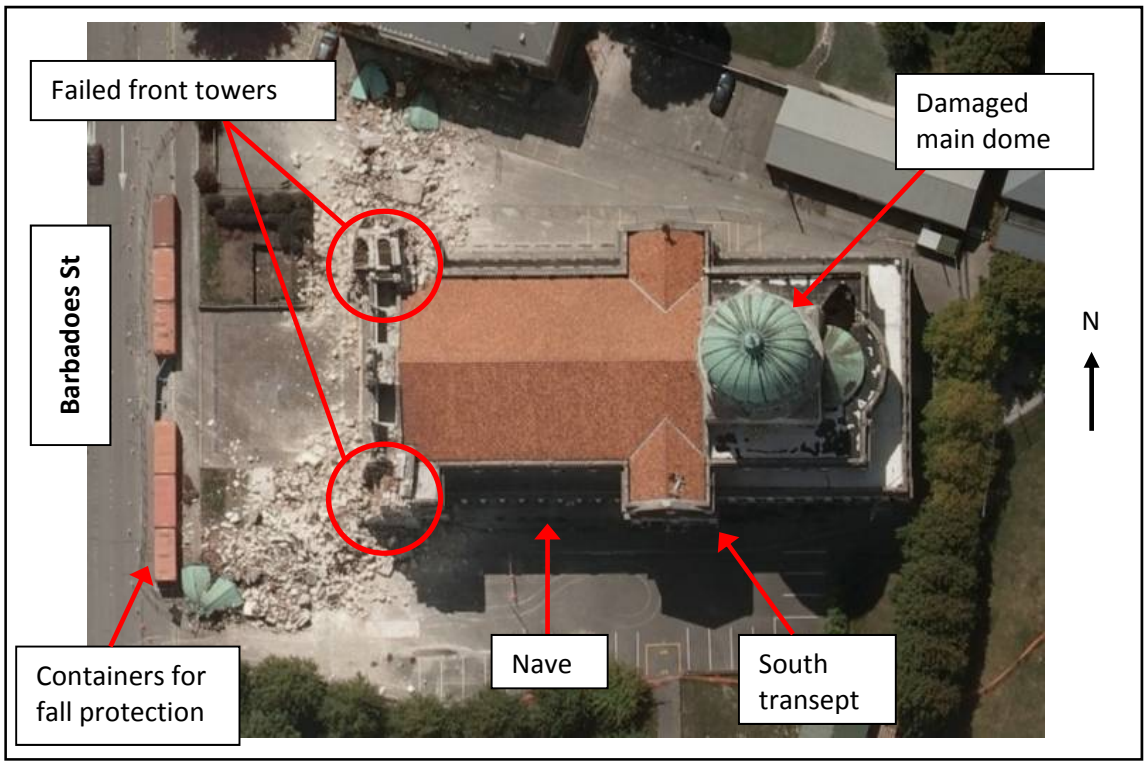
Figure 15. Damage to the south bell tower following the 26 December 2010 earthquake



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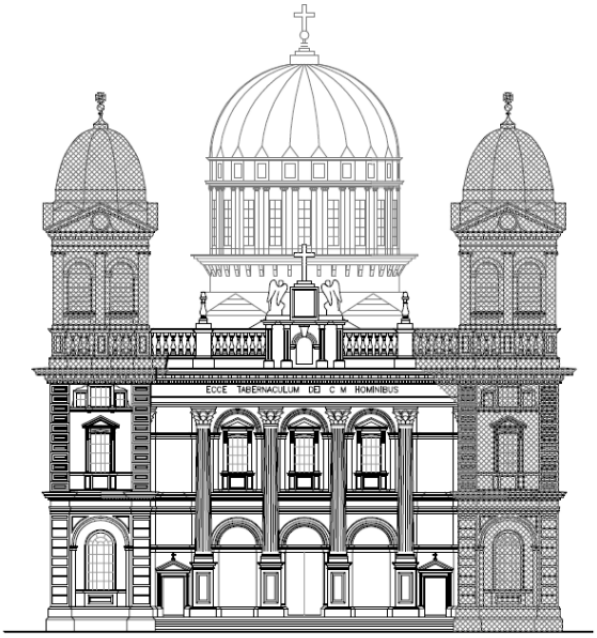
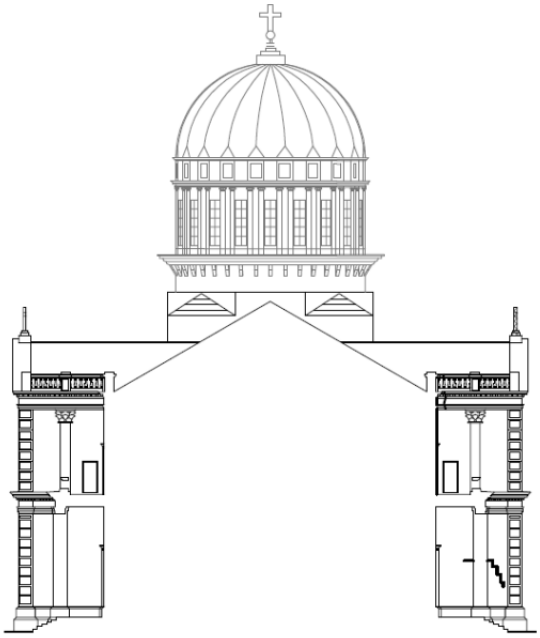
(a) Failed north arch supporting the dome

(b) East end showing damaged walls and collapsed roof

Figure 17. Damage to arches and roof from the 22 February 2011 earthquake



(a) West elevation



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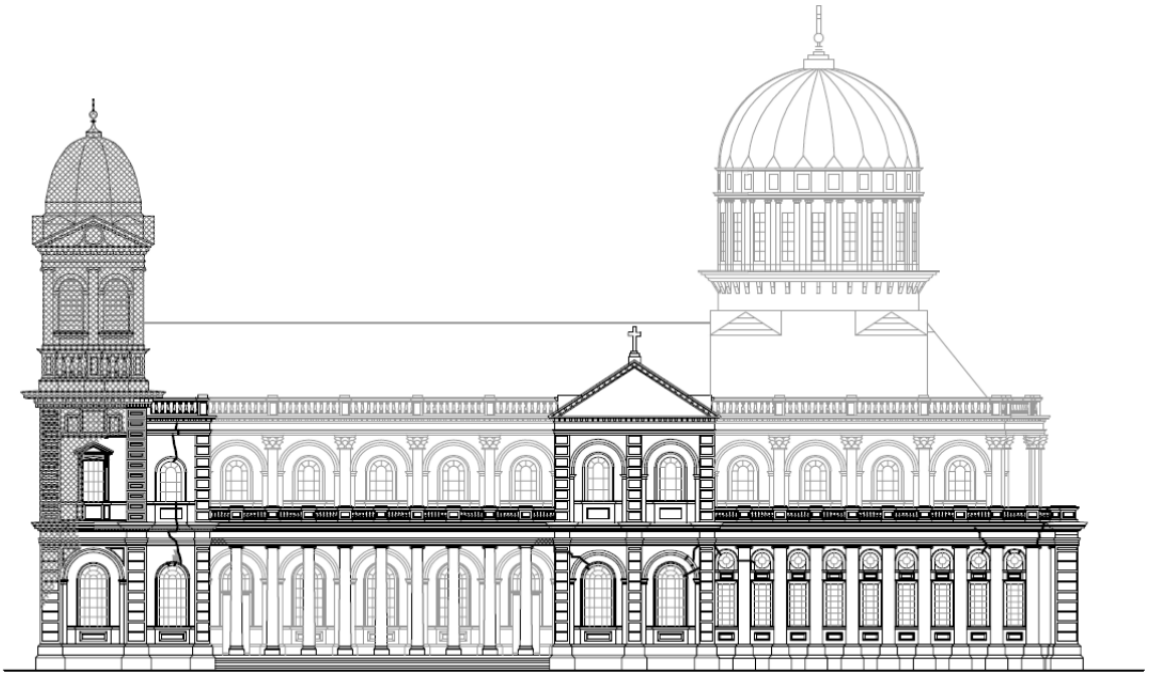

(c) South elevation (outer)

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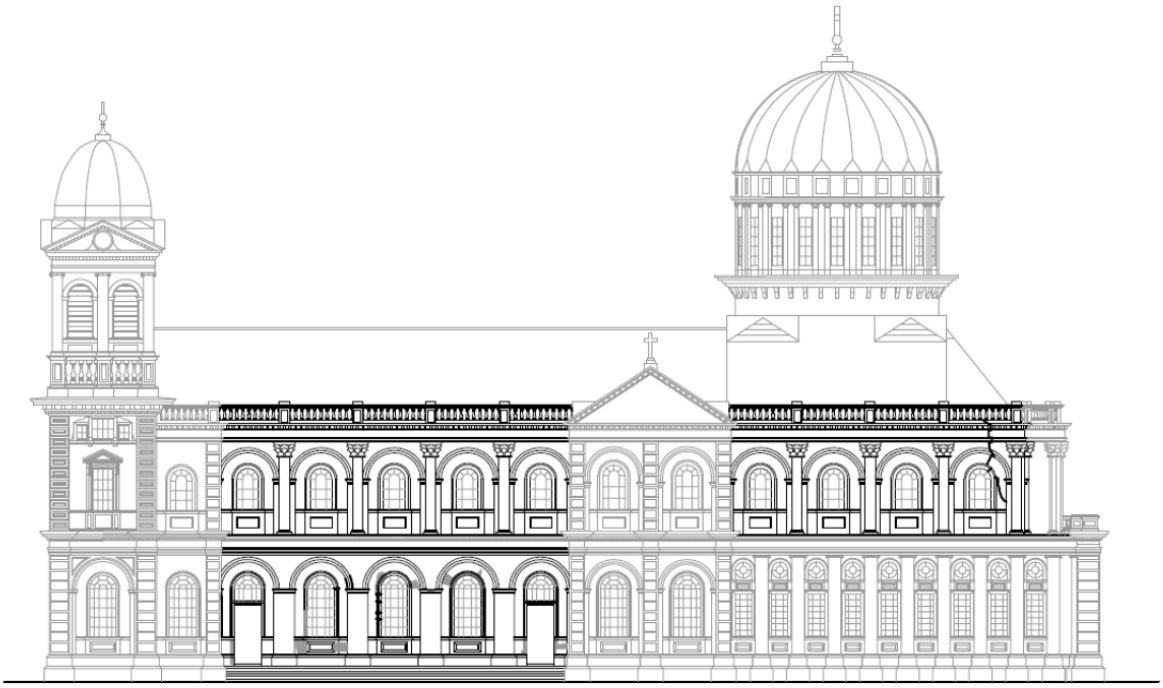
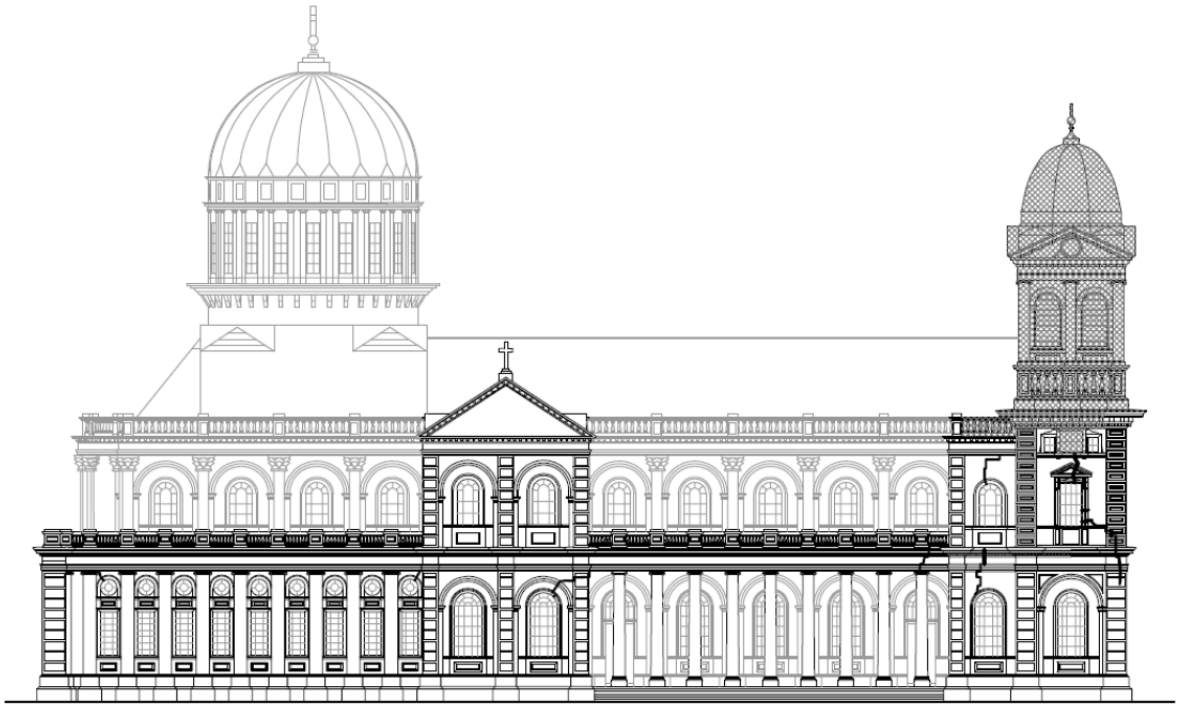
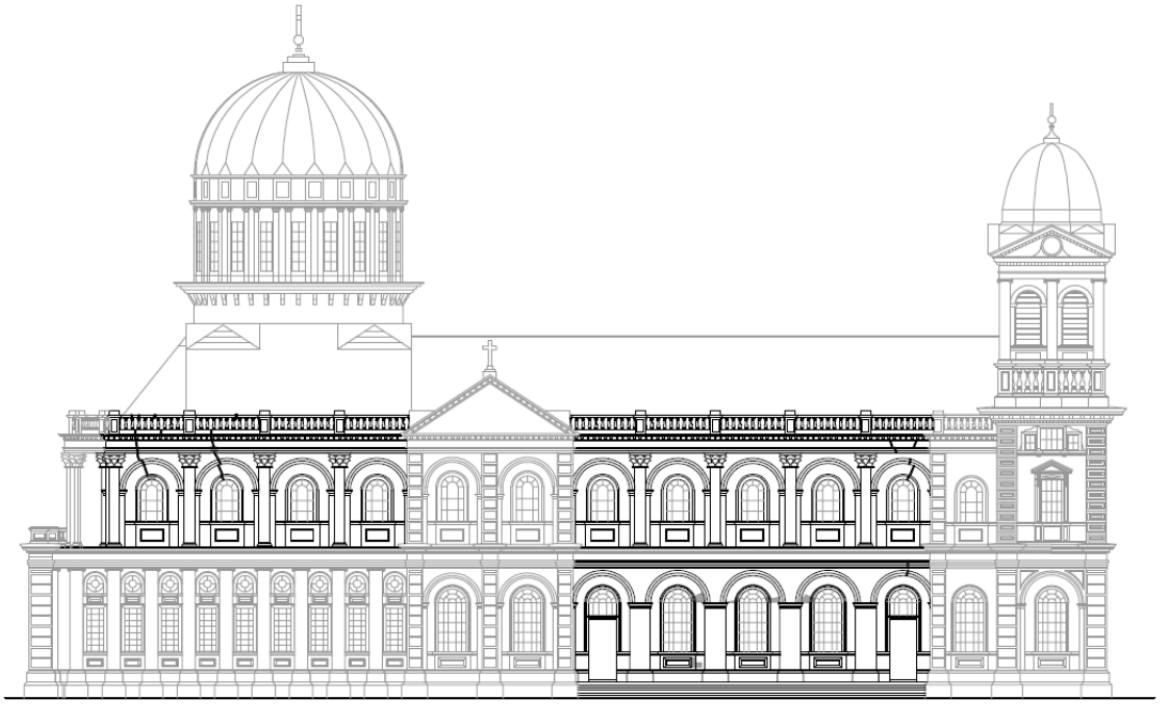


(d) South elevation (inner)

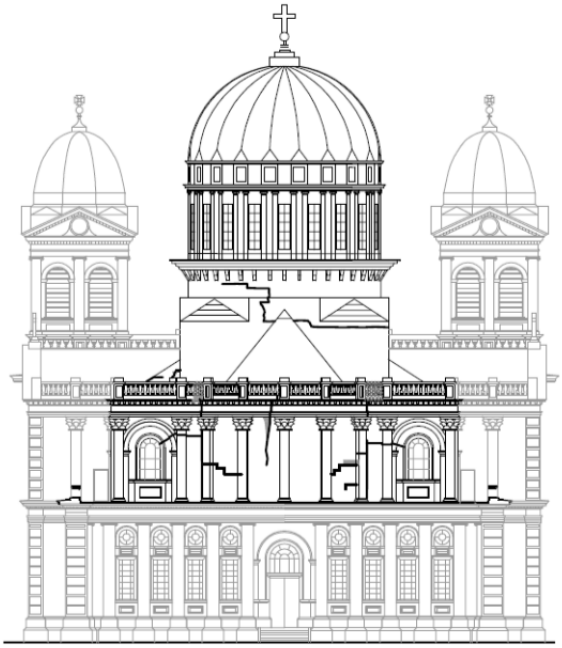


(e) North elevation (outer)

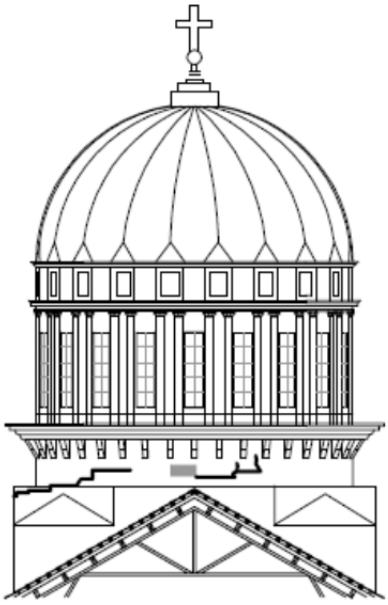
Figure 18. Continued



(f) North elevation (inner)

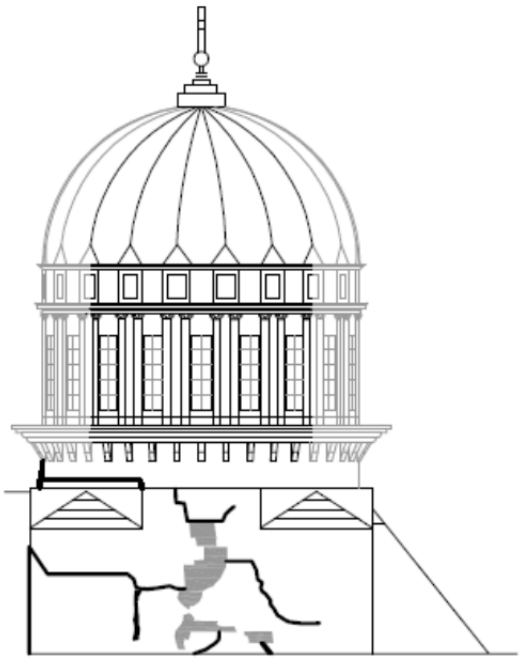


(g) East elevation

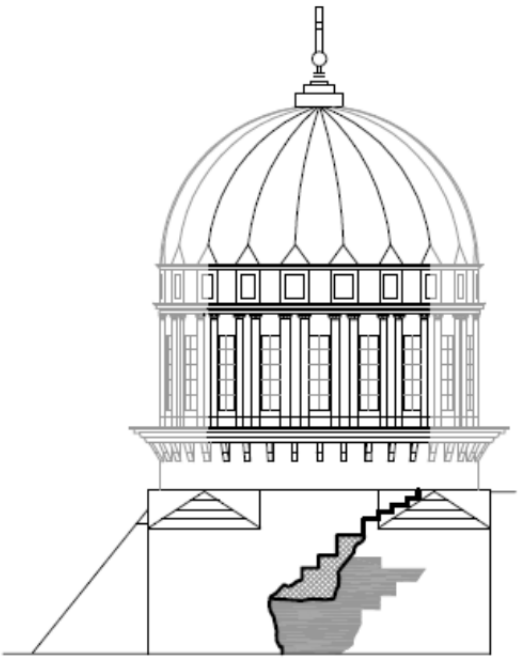


(h) Rotunda, west elevation

Figure 18. Continued

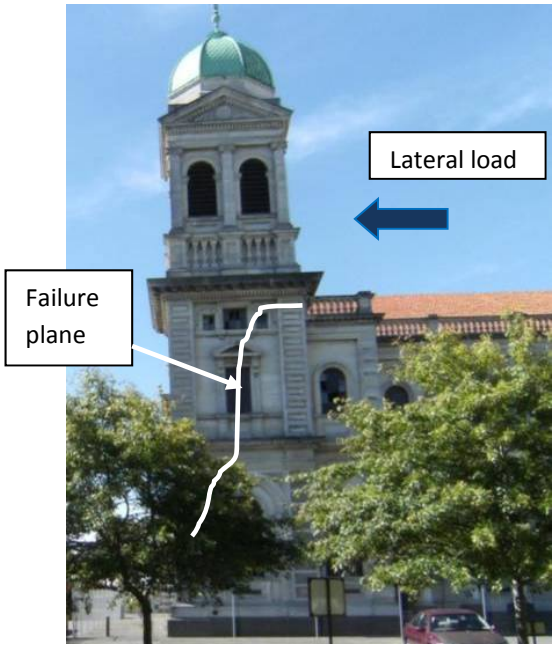


(i) Rotunda, south elevation



(j) Rotunda, north elevation

Figure 18. Continued



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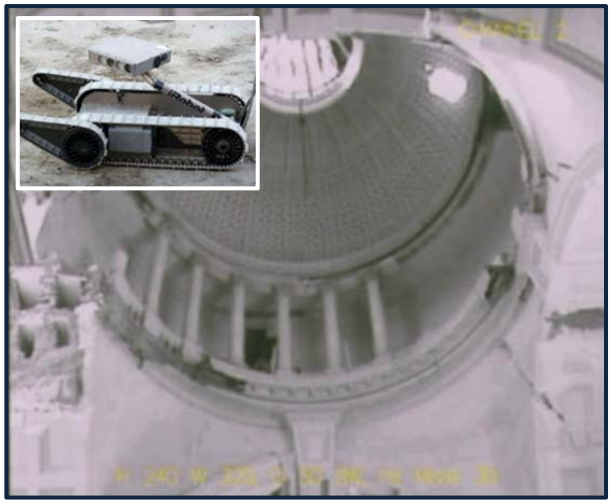

(b) i-Robot PackBot with screenshot looking up into the damaged dome

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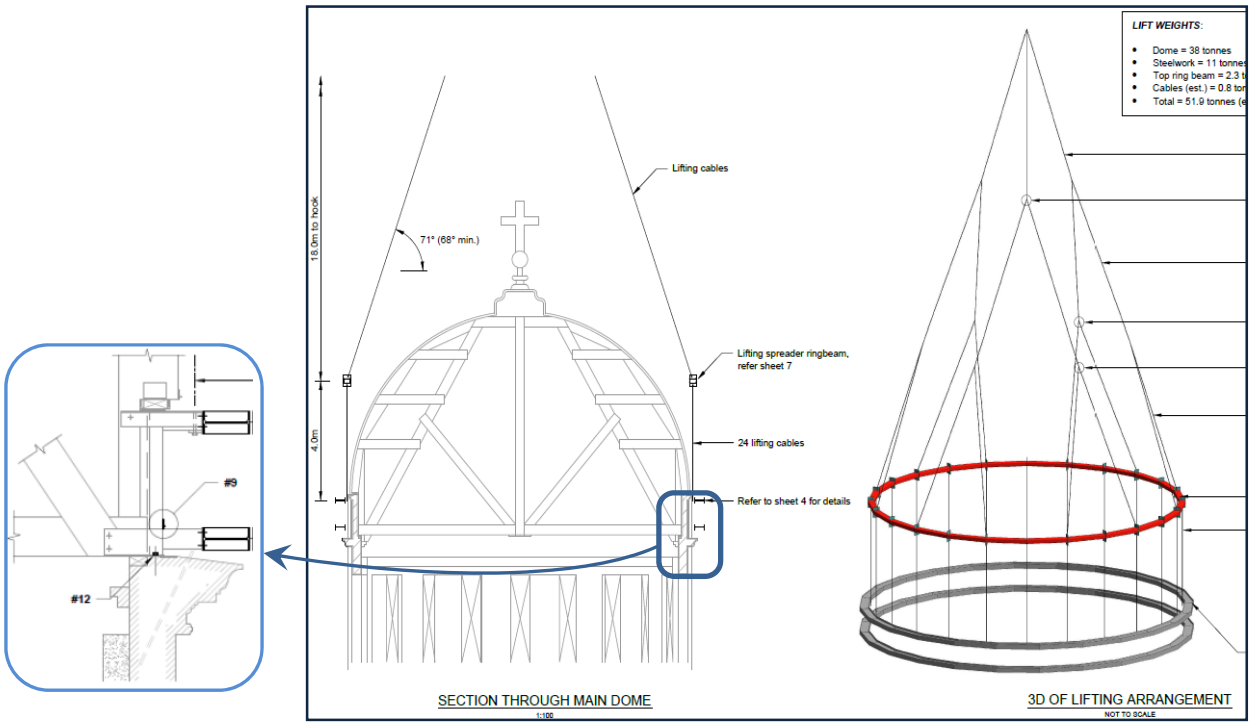

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(b) Dome roof being lowered onto the ground

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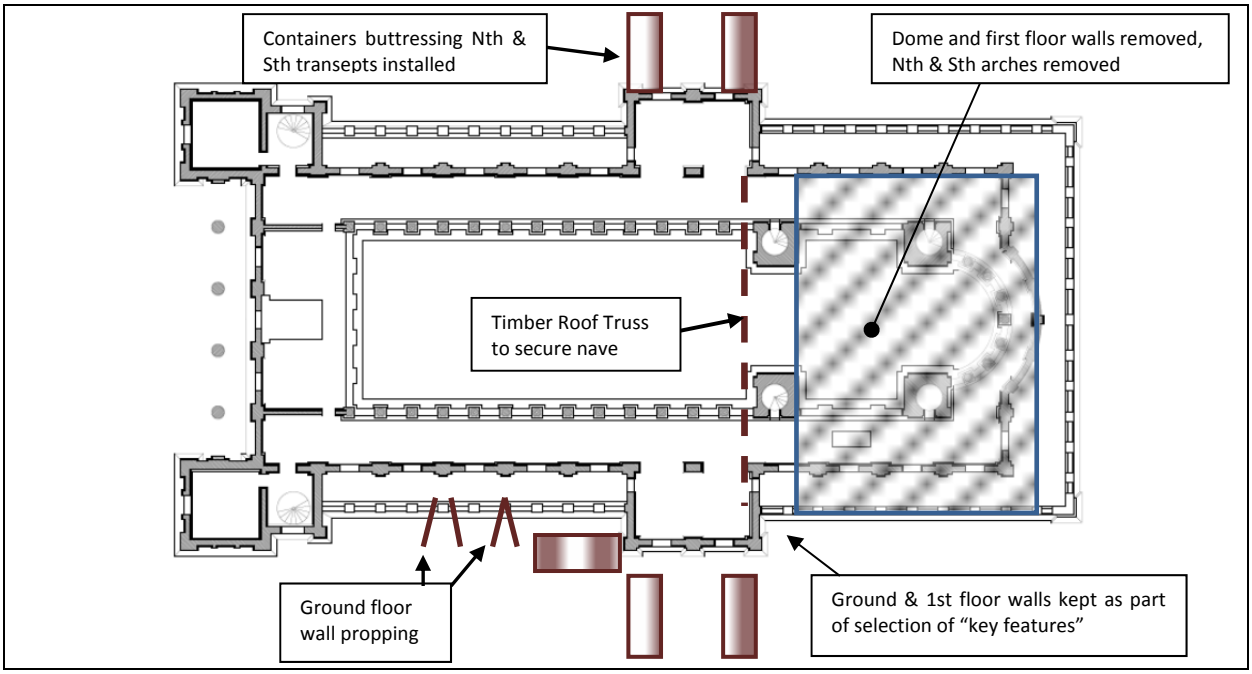

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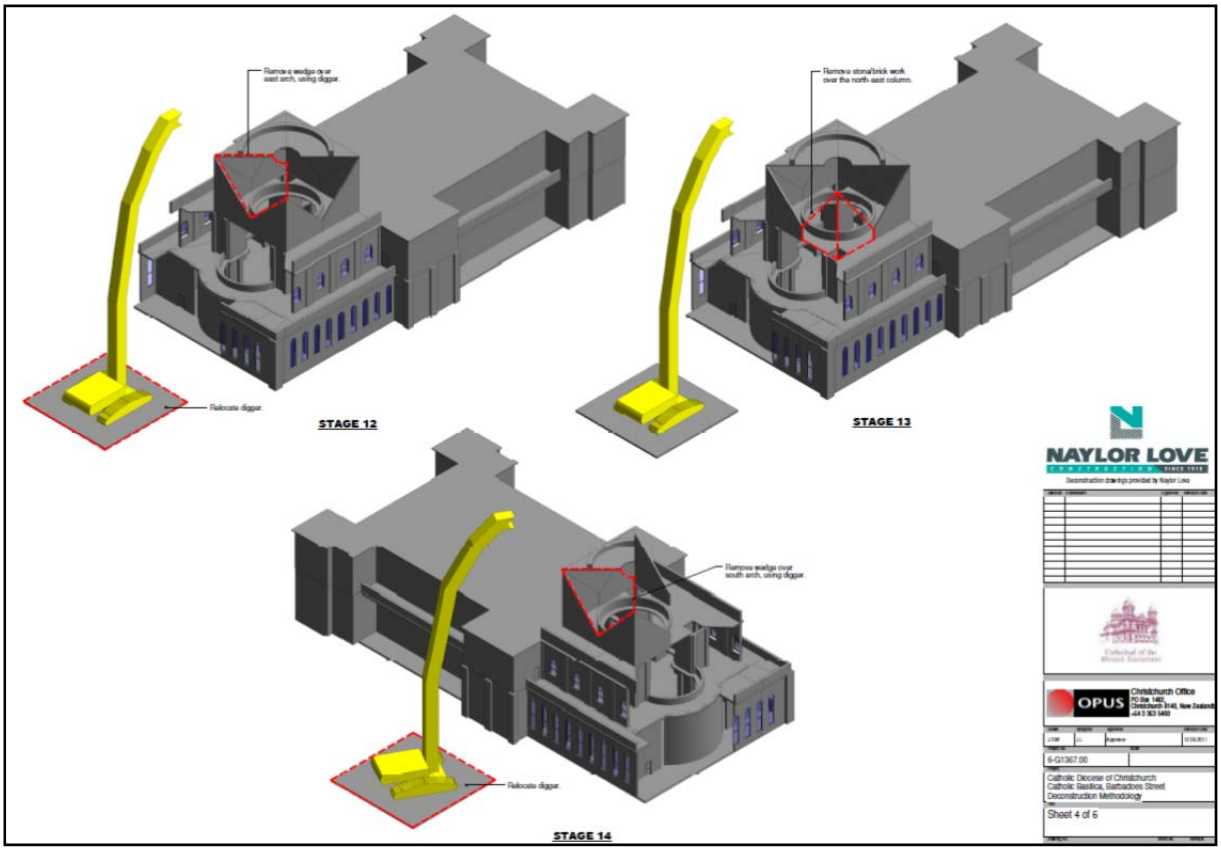

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