A novel approach to rehabilitation of a church in Fendalton, Christchurch

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ABSTRACT: This case study outlines the repair and retrofit of a riverside church in Fendalton, Christchurch, that underwent significant damage in the Canterbury earthquakes. The building experienced significant separation and differential settlement due to liquefaction and lateral spread of the underlying soil. The church was originally constructed in 1959 in double height lightly reinforced concrete block masonry. Strengthening undertaken in the 1990's successfully tied the building together, otherwise partial collapse would likely have occurred. The main earthquake damage to the building included: significant wall and floor slab cracking, sloping floors, and movement of the foundations. The retrofit design looked to restore the church to close to the condition it was in before the earthquakes; raise the seismic performance level and mitigate against the risk of further damage in future serviceability level seismic events. To achieve these objectives, seismic mass was removed by replacing the masonry walls with timber shear walls, whilst retaining the large clear-span roof during demolition and subsequent reconstruction. Mitigation of future liquefaction damage was achieved by replacing the heavily damaged areas of slab with a waffle raft slab and carefully located "settlement moderating" piles. The project successfully achieved an appropriate balance between an efficient use of church funds and the need to moderate future seismic risk.

1 INTRODUCTION

The church building in Fendalton, Christchurch, is a Stake Centre owned by the Church of Jesus Christ of the Latter-Day Saints. The original building was constructed around 1959 by church volunteers and was largely designed by the church central drawing office in the USA.

The purpose of this paper is to provide an insight into the building's poor structural condition after the Canterbury earthquakes, and present the innovative retrofit solution undertaken which essentially saved the building from demolition.

Figure 1. The front view of the church in Fendalton.

1.1 Background

The original structure comprised of lightly reinforced masonry walls founded on shallow concrete foundations, with timber roof framing. The site is completely surrounded by the Avon River (see Figure 2). The building is predominately single storey, with some mezzanine plant space. It is irregular both in plan and elevation. There are two large open plan chapel and hall areas, which have double height walls. Extended off the chapel is a single height wing of classroom and office spaces, which has a relatively large number of dividing walls (see Figures 1 and 2). In the 1990's the building was strengthened by a Christchurch engineering firm.

The 2010 and 2011 earthquakes and subsequent aftershocks caused significant damage to the building structure, largely as a result of liquefaction and lateral spreading. As a consequence the building was unable to be occupied until repairs had been carried out. Options for a full or partial demolition and replacement were considered, but were ultimately discounted in favour of a more ambitious option to rebuild the damaged areas with the large roof being retained. This course of action meant the church members could reoccupy their building sooner and the project could be readily funded using the church risk reserves set aside for natural disasters such as this.

The project objectives discussed and agreed with the Church were to return the building generally to its pre-earthquake condition and satisfy the requirements of the New Zealand Building Act. Another key issue that required mitigation and discussion with the client was the potential for damage to occur during a future Serviceability Level seismic event.

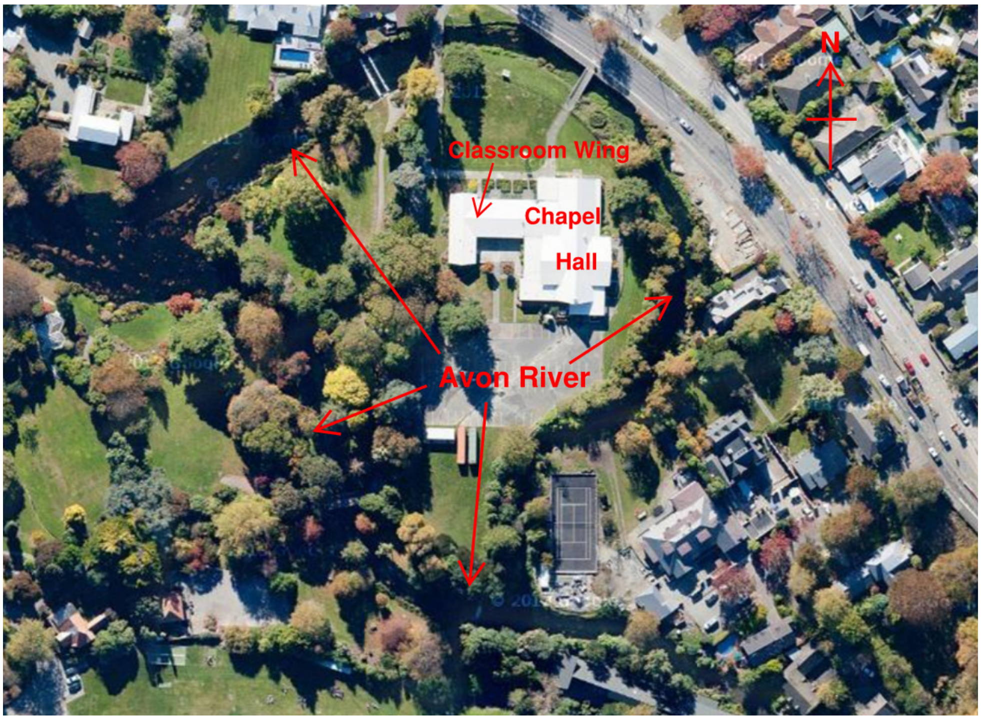

Figure 2. The site of the church in Fendalton.

1.2 The Site and Earthquake Hazard

The unique site in Fendalton, Christchurch is completely surrounded by the Avon River (as shown in Figure 2). The shallow soil in the area largely comprises alluvial deposits from the river that have built up over time. The relatively soft sandy/gravelly soil extends to a depth of at least 15m. Due to the close proximity of the river and Christchurch being of largely flat terrain, the water table is only approximately 1.5m below ground at the location of the building. Saturated loosely compacted sandy soils of this type are of course susceptible to liquefaction during an earthquake.

On 4th September 2010, an earthquake of magnitude 7.1 struck Christchurch and the surrounding Canterbury region. This was followed by three major aftershocks as well as on-going smaller tremors.

Due to the nature of these earthquakes in magnitude, depth and proximity, the building in Fendalton experienced ground accelerations sufficient to induce liquefaction at least three to four times. The high ground accelerations also induced significant lateral forces in the structure and if it were not for the building's previous strengthening, it is probable that the building would have partially collapsed.

2 PAST SEISMIC STRENGTHENING

In the 1990's the structure was strengthened to satisfy the 1992 New Zealand design loadings standard (NZS 4203:1992). The following section briefly describes some of the key strengthening works undertaken. We consider that these works successfully held the building together, and without which the Eastern hall gable would have likely detached from the large roof beams and collapsed.

Figure 3. Hall strengthening. Eastern side during construction.

Two new reinforced concrete shear walls were constructed to replace the two short masonry walls towards the Eastern end of the hall (as shown in Figure 3). These walls were anchored to the adjacent structure and roof bracing was added between the new walls. Steel cross bracing was added to some of the masonry walls adjacent to the stage. Above the stage, plywood diaphragms were installed, which also served to restrain the high Southern gable wall. Independent supports were added beneath critical load bearing elements, which comprised mostly of steel posts beneath existing roof beams.

The chapel received similar treatment to the hall. The masonry spire was removed from the entrance canopy and rebuilt in steel on the front lawn. Diagonal strap roof bracing was installed over the single height office/classroom masonry walls, and a number of timber framed bracing walls were also added across the classroom wing.

3 EARTHQUAKE DAMAGE

The principal cause of the damage was differential ground movement and the inability of the relatively brittle walls and foundations to articulate. The damage described below only covers the primary structural damage. The building and its site also suffered other damage to non-structural features.

3.1 Ground Movement

The ground beneath the Fendalton building settled due to two phenomena, direct vertical settlement due to sand ejection from liquefaction, and settlement due to liquefaction induced lateral spread. When the soil liquefied it flowed towards the surrounding river, which is the lowest point of the site. The river encroaches to within approximately 20m of the hall on the Eastern side of the building (see Figure 4). Due to the proximity of the river, vertical settlement was particularly prominent on the Eastern side of the building. Some lateral displacement of the building was also evident on the Eastern side (see Figure 5).

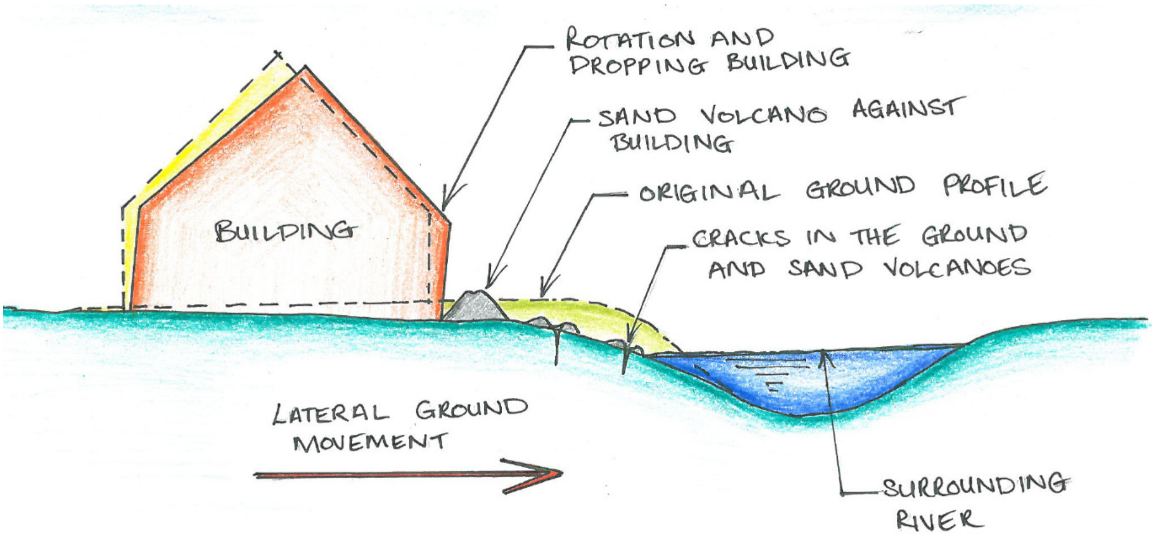

Figure 4. Lateral spread towards the river that occurred when the sub-surface soil liquefied.

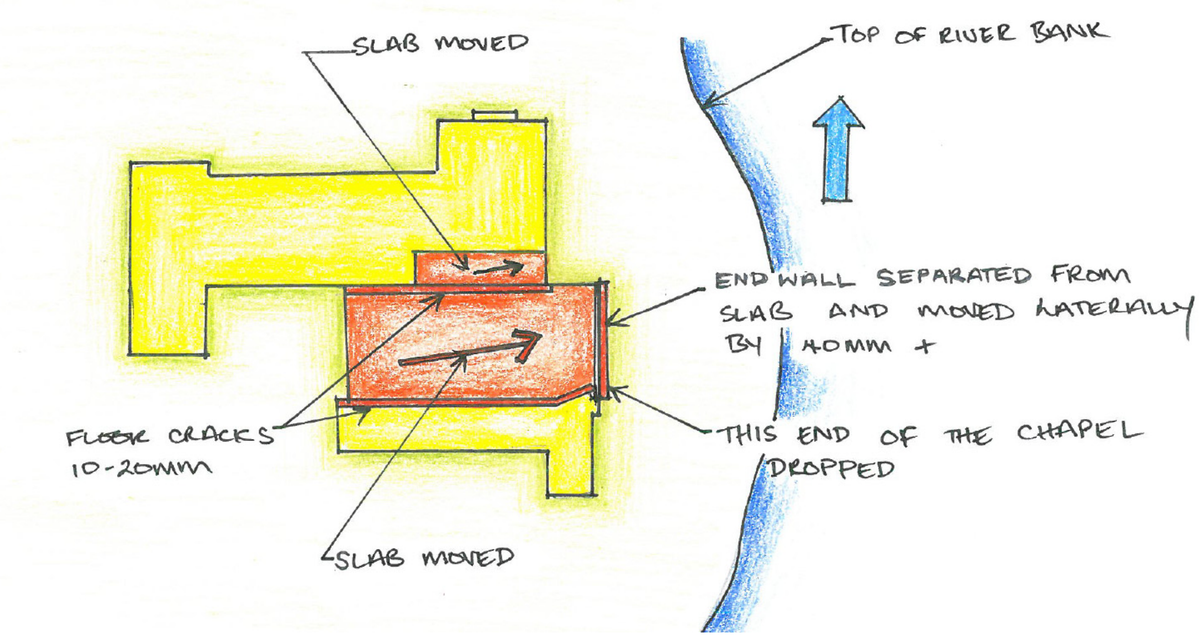

Figure 5. Plan view of lateral spread towards the river that occurred when the soil liquefied.

A survey of the floor levels across the building was undertaken. The resulting slopes in the floor due to differential settlement are presented in Figure 6 below. In general, it was observed that areas of the building with a floor slope no worse than 1:150 had suffered relatively little structural damage. These areas were proposed to be repaired and the floor slab to remain as it was, on the basis that the slabs were still structurally intact and the 1:150 slope was considered difficult to detect by the Church.

3.2 The Hall

As shown in Figure 6 there had been a relatively significant level of settlement on the building's Eastern side, which is the side closest to the river. The pattern of settlement over the three most significant earthquakes had been similar in terms of location; each seismic event just increased the severity of the settlement. High differential settlement had also occurred at locations where the double height heavy masonry walls had plunged downwards resulting in a low point in the surrounding slab area. As a result the hall was the most critically damaged area. Two of the double height masonry walls had large diagonal cracks principally as a result of their brittle construction and movement imposed by the differential settlement and lateral spread towards the river (as shown in Figure 7). The

concrete slab underneath the hall's parquet floor also had sustained significant cracking and a noticeable slope.

3.3 Classroom Wing

Most ancillary rooms are made up of closely spaced single height masonry walls and some timber walls. These had mostly only undergone superficial cracking. The majority of the cracking did not require structural repair. There were however nine instances where structural repair was required. Structural repair consisted of local replacement of mortar, and full block replacement in severe cases.

Figure 6. Floor slopes calculated from floor level surveys.

Figure 7. Double height masonry wall in the hall exhibiting large diagonal cracks, and lateral movement of the access ramp on the Eastern side.

4 SEISMIC RETROFIT SOLUTION

The retrofit solution undertaken sought to mitigate the liquefaction risk and restore the building close to the condition it was before the earthquakes. Structural works were predominately undertaken in the hall area, which received the most significant structural damage. The other areas, including the chapel and the low height amenity rooms received largely cosmetic repair. Being an existing damaged structure, some issues were only presented to the design team as they were uncovered during construction.

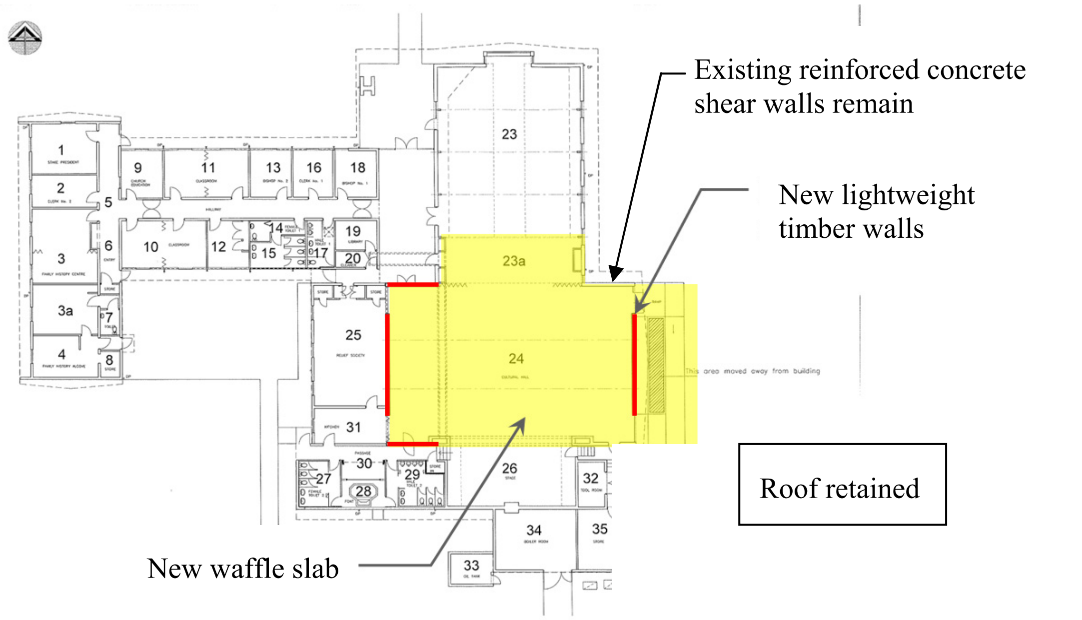

Figure 8. Location of features requiring replacement.

4.1 Mass Removal

It was proposed to reduce the seismic weight of the structure by replacing the hall's double height masonry walls with light-weight timber walls (location shown in Figure 8). The masonry walls had been damaged significantly, such that more than a cosmetic repair was required. The new lightweight timber walls reduced the pressure under the existing foundations and they will also be more forgiving than masonry with potential movement from an earthquake (see Figure 9). Another major advantage of the removed mass was significantly reduced demand on the existing reinforced concrete shear walls, compensating for the Z factor increase from 0.22 to 0.3.

The significantly lighter timber walls with stiffer foundations were tied into the new surrounding slabs. To assist in preventing the walls from settling differentially or moving away from the floor in a future seismic event. The demolition of these walls had to be carried out in a staged approach, and significant temporary bracing was required to support the large roof (see Figure 3).

4.2 Settlement "Mitigating" Piles

As noted previously, the site towards the Eastern end of the hall was considered to be subject to an elevated risk of liquefaction settlement as a result of lateral spreading towards the river. On the basis of geotechnical advice received, some settlement was expected in this location in the serviceability limit state. It was therefore considered important for the design to incorporate mitigation measures to reduce the risk of repairs being required to an acceptably low level.

Four steel screw piles were installed along the eastern wall line only. The project team did not favour a more widespread use of piles, as they would tend to create high points when the building platform settles. Piles were however considered to be beneficial on the Eastern wall line to "temper" the magnitude of settlement in this zone which is close to the river. These piles were deliberately founded in a softer material than one might for a new building, as some settlement of the pile was seen to be

beneficial. Essentially the concept sought to tune the foundations to encourage a more uniform settlement profile across the site.

The screw pile shaft was well tied to the foundation beam, so as to be able to plastically deform in the event of lateral soil loading whilst maintaining gravity load support. The screw pile nearest the existing concrete shear wall also served to improve the overturning performance of that wall where it is most susceptible in the lateral spread zone. The screw pile was considered the ideal pile type given these objective and access constraints.

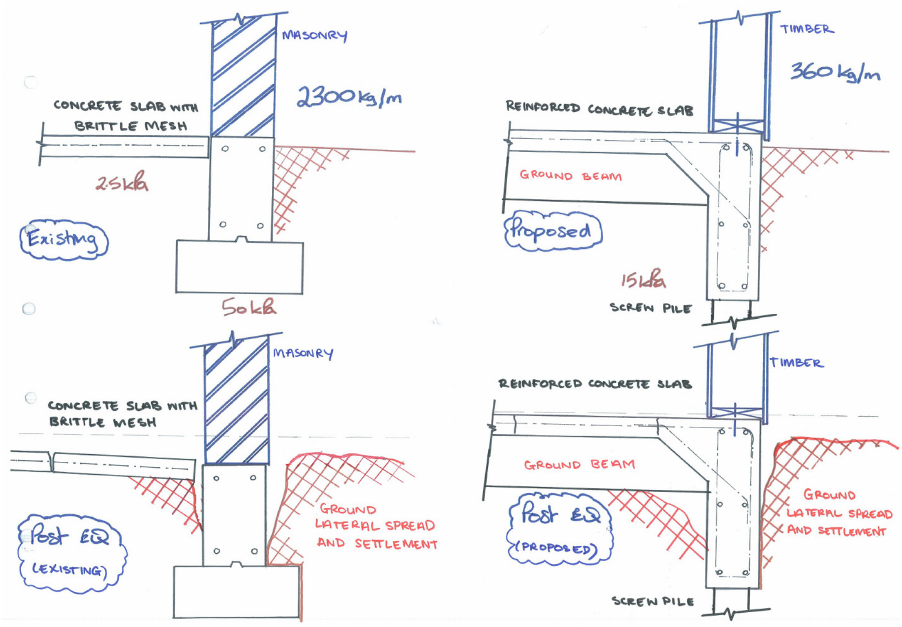

Figure 9. Existing and new foundation system, before and after a seismic event.

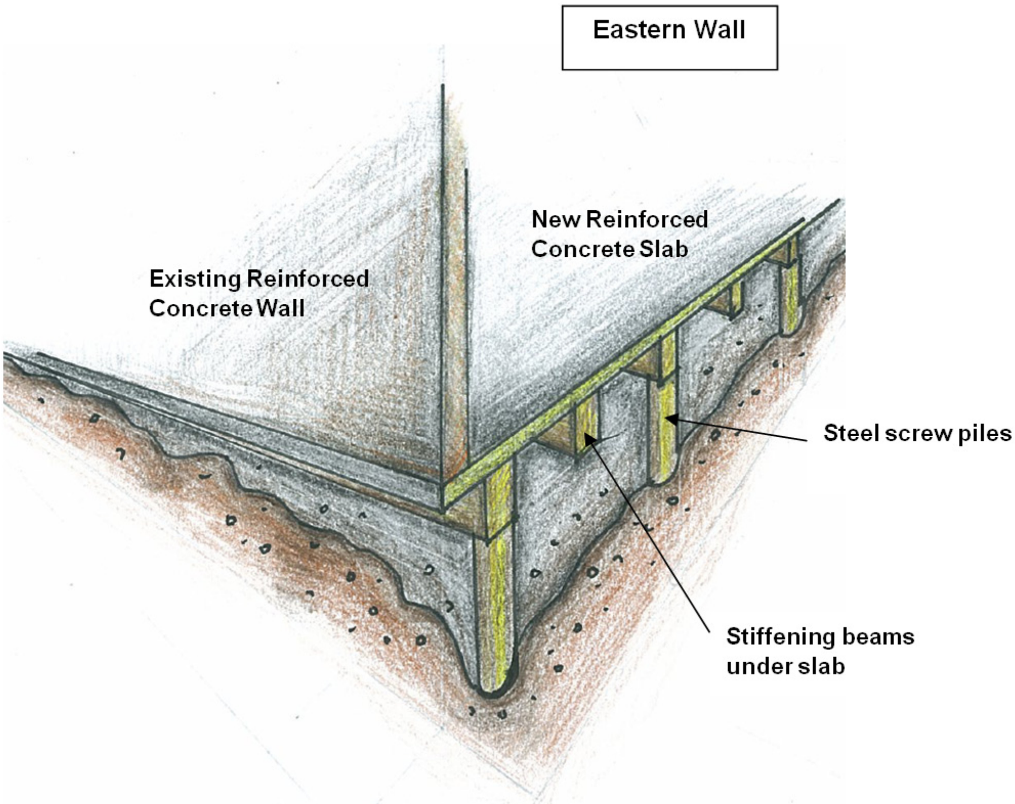

Figure 10. Damage mitigation solution, for the occurrence of lateral spread.

4.3 **Waffle Slab**

The existing slab in the hall was severely cracked and had settled irregularly to the extent that the floor sloped noticeably. This slab and the footings beneath the walls were replaced, and the new floor was constructed with small stiffening beams under it in a 2.5 x 5.0 m grid. This created a raft like slab system. The increased floor stiffness results in a reduced floor curvature in the event of differential settlement, therefore a reduced risk of future disruption of the hall parquet floor. The beams also act as ties, helping to hold the entire slab together and so to reduce movement of the Eastern wall towards the river. Furthermore, the beams enable the slab at the Eastern end of the hall to span clear of the ground onto the piles in the event that the ground drops away beneath the floor, see Figure 10 for a representation of this effect graphically.

If there was ever a need to re-level the floor (due to a very large earthquake), the beam system would make it possible to adjust and level portions of the floor without demolition of the slab. A local hole would need to be cut and the floor lifted through either pressure grouting or local jacking.

Adjacent to the hall, the slab and walls around the stage area did not require reconstruction. Therefore there is a higher risk of future settlement for the stage area than the adjoining hall area. To counter this, a timber support packer was detailed under the new stage, to make it relatively easy to re-level in the future (either raise or drop portions or the whole stage if necessary), see Figure 11.

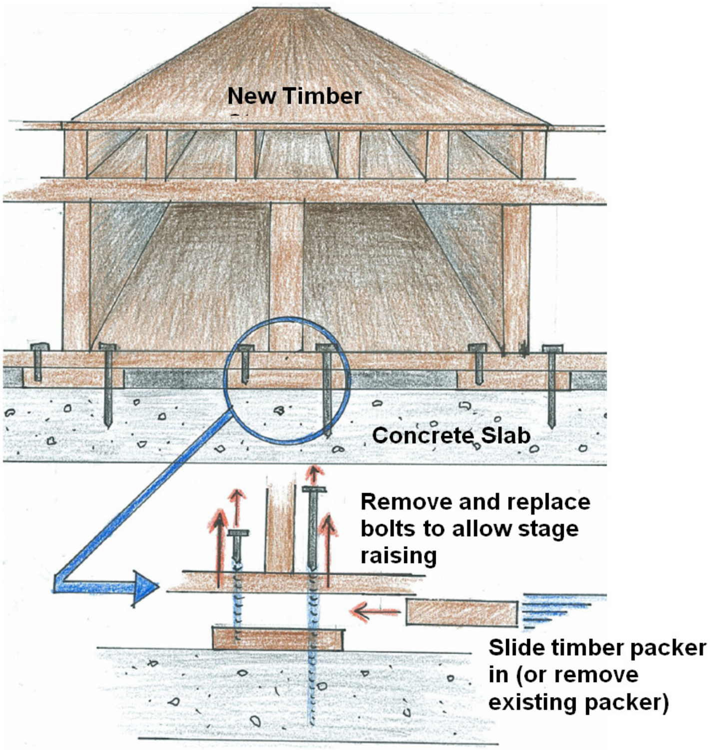

Figure 11. Stage rehabilitation solution, in the occurrence of ground settlement from a seismic event.

5 CONCLUSION

After undergoing significant differential settlement and damage from the Canterbury earthquakes, the church building was restored to close to its previous condition. The project was delivered successfully, by retrofitting and repairing the existing structure, rather than demolishing it. This resulted in a reduction in the overall project cost, project timeframe and impact on the Church. The elevated risk of settlement at the site during a Serviceability Limit State seismic event was mitigated through the removal of mass, strategic positioning of piles and careful detailing. The overall building aesthetic and heritage have been preserved for the church membership. One building saved!