



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 consequences of ‘mostly’ base isolating a building

G. Cole, H. Tatham & R. Jury

Beca Ltd, Wellington.

ABSTRACT

Following the 14 November 2016 Kaikoura earthquake, urgent shoring of the Rankine Brown Building’s North and South lift shafts was required. The building suffered unique damage in this earthquake due to the earlier base isolation retrofit of all but the two lift shafts across the isolation plane. Both lift shafts are self-supporting structures from a gravity perspective within the wider building which supports the shaft for lateral seismic loading. The damage to the lift shafts at the isolation plane meant that shoring was required to support the weight of the lift shafts above the damage zone. Until this work was completed, public access to the wider building could not be granted.

Maintaining the support of these internal lift shafts presented a variety of challenges. While reinstatement works were being completed, the shoring needed to be able to accommodate horizontal movement in the event of a future earthquake (aftershocks were considered a real possibility). The shoring also needed to provide reliable vertical support to the shafts, without vertical movements, while the damaged areas were removed. Construction space was limited, and the client required the building to be re-opened on a very tight schedule. This paper details the challenges faced and solutions implemented by Beca and others in the months immediately following the Kaikoura earthquake. The damage sustained indicates the likely consequences if ‘secondary’ structural and ‘non-structural’ elements are not addressed when applying a new structural system to a building.

1 INTRODUCTION

The 14 November 2016 Kaikoura earthquake (Stirling 2017) impacted a number of Wellington buildings. However, the damage encountered at the Rankine Brown building was highly unusual and in need of immediate attention. The damage can be primarily attributed to the detailing found around the building’s two lift shafts, as described below.

Paper 20

1.1 Building Configuration

Rankine Brown was originally constructed in 1960 and is a nine storey reinforced concrete building (Figure 1). This comprises three podium floors and six tower floors. The flooring is mostly post-tensioned waffle slab, with large central columns and smaller perimeter columns.

The building was base isolated via retrofit in 2002 by another consultancy. The base isolation added lead-rubber bearings (LRBs) to the central columns and provided a movement detail to the perimeter columns. This isolation was installed just below the first floor, and included details to permit horizontal moment for the structural elements crossing the isolation plane. However, the two lift shafts in the building were not retrofitted to accommodate movements across the isolation plane.

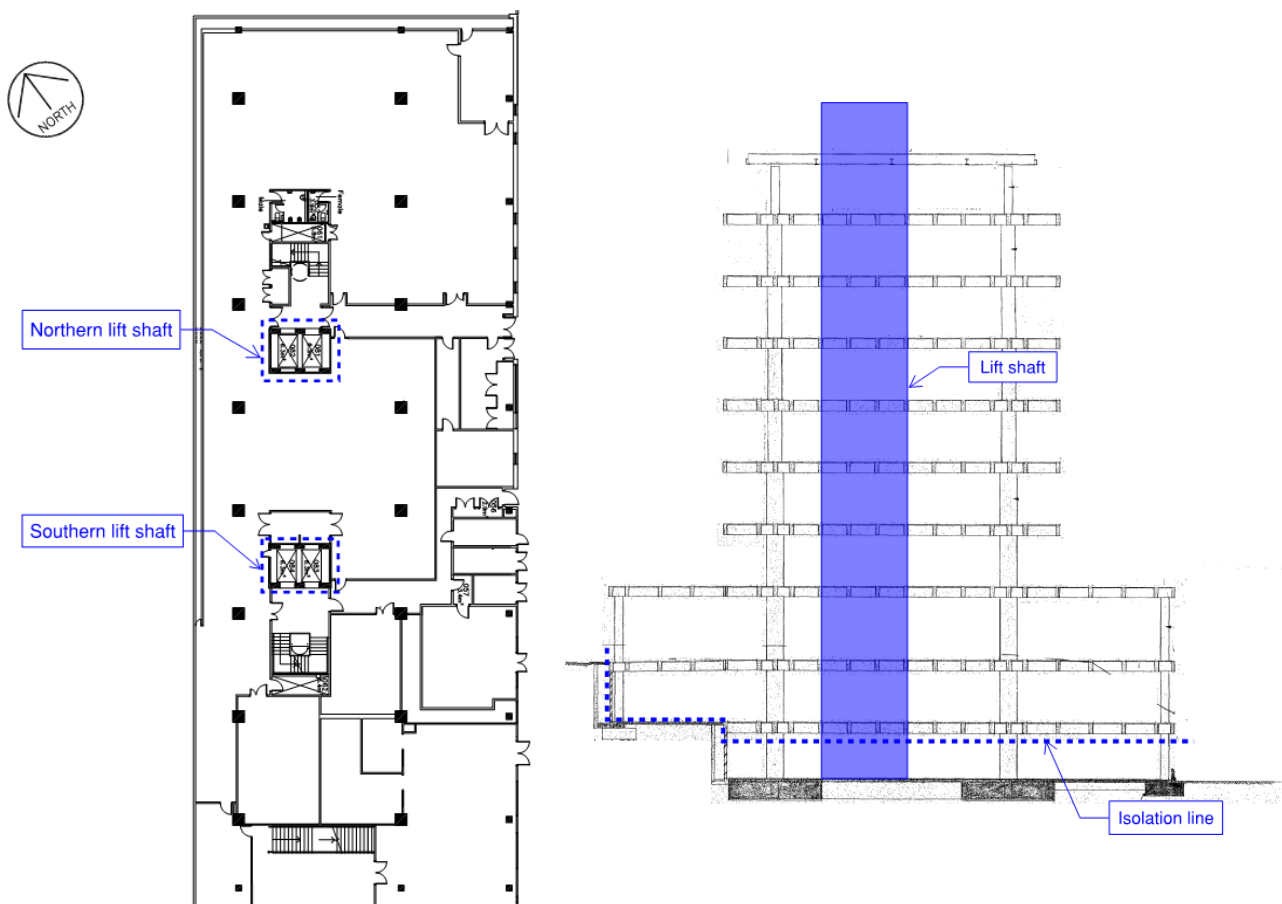


Figure 1: Building ground floor plan and transverse section

The lift shafts are self-supporting structures from a gravity perspective, but rely on the wider building for lateral support. The north and south faces of each lift shaft's structure includes a precast concrete beam at each floor, supported by three columns between each floor (Figure 2A). The gaps between these columns form the lift doorways. The two outer columns contain voids similar to conventional hollow core flooring and are connected top and bottom to the floor level beams by steel flats located on each column centreline. The central column is solid and includes six grouted, post tensioned tendons. The east and west walls of the lift shaft are light weight timber, and are not structural components.

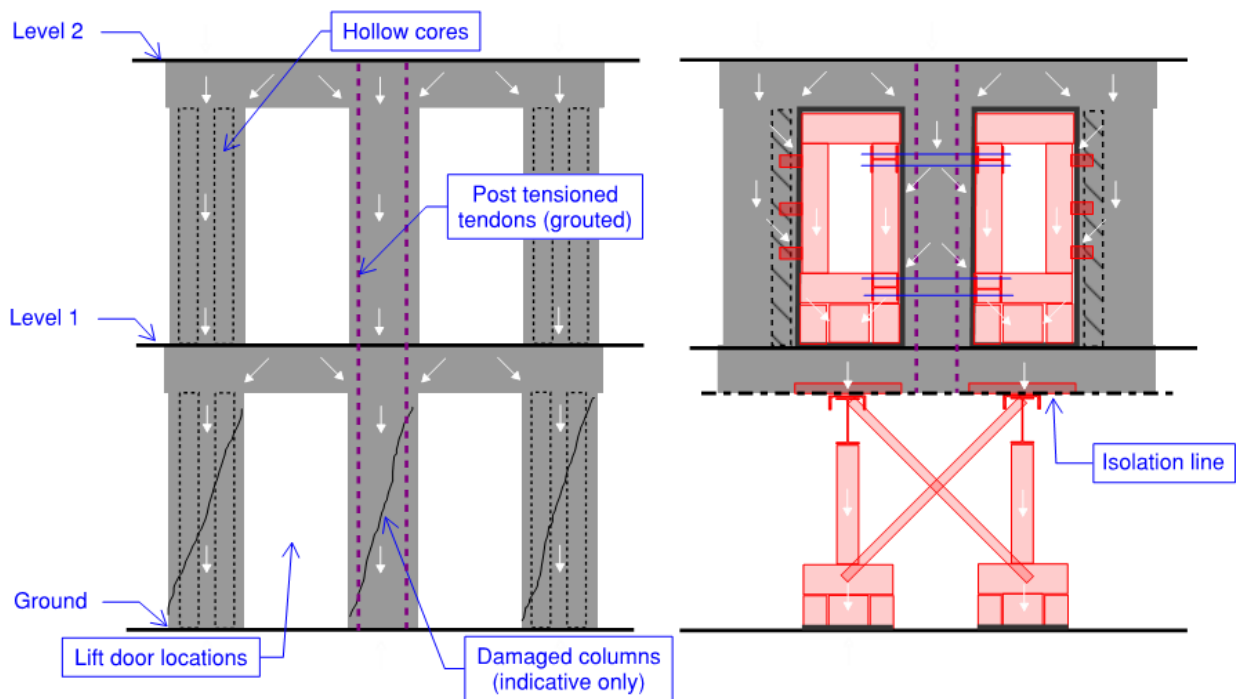


Figure 2: A) Structural configuration of lift shaft. B) temporary shoring. White arrows show gravity loadpath.

1.2 Damage from the Kaikoura Earthquake

Prior to reviewing the drawings, the damage observed in Rankine Brown following the Kaikoura earthquake did not cause much concern. Large diagonal cracks could be observed in GIB linings at the ground floor around the lift shaft, however the lift shaft appeared to have gravity support by the surrounding RC frame structure at each individual floor. However, once the drawings were reviewed, the importance of these lift columns was realised. After removal of the GIB linings, a large single crack was observed in all twelve concrete columns at ground floor (six in each lift shaft). The damage was consistent with a shear failure in each column, due to the two lift shafts effectively trying to resist the building's entire base shear across the isolation plane. Minimal flexural cracking was observed.

1.3 Remediation Solution

Beca's solution was a multi-phase approach, including:

1. An expedited design and construction of temporary shoring which made the building accessible to the public and staff (Figure 2B), but without lift access. This phase was completed (including design, fabrication and installation) within 16 weeks.
2. Design and construction of a new permanent isolation detail for the lift shafts, reinstating lift access while the building remained occupied.
3. Completion of a Detailed Seismic Assessment and strengthening of the rest of the structure to achieve our client's desired %NBS rating.

This paper focuses on the challenges faced and solutions provided in the first phase of this project.

2 TEMPORARY SHORING DESIGN

Our client required that the building be able to be used as soon as possible, and had a specific deadline of the end of February 2017 to achieve this. In order to meet this timetable, a ‘temporary’ solution was required. This solution provided a new reliable gravity structure to support each lift shaft which could also withstand aftershocks or other seismic activity. However, the lift shafts themselves would remain largely unusable until further works were undertaken. This was due to some steel being installed within the lift shaft, which prevented normal lift operation.

2.1 Design

The provided temporary shoring comprised two floors of structural steel (refer Figure 2B). Where the shoring steel crossed the isolation line at the underside of the Level 1 concrete beam, a sliding surface was provided. To facilitate this, the bottom of the Level 1 concrete beam was armoured with a steel plate. PTFE (Teflon) was attached to the surface of this plate to reduce the induced friction in an earthquake event.

Below the sliding line, polished stainless steel contacted directly with the PTFE. The stainless steel was embedded into a PFC (orientated with its flanges pointed downwards). The PFC was welded on top of a 610UB. This configuration allowed East-West movement by sliding along the armoured Level 1 beam, and North-South movement by sliding along the PFC/UB.

The gravity support that was provided by the ground floor steel acted in the centre of each previous lift door lintel. This provided the required East-West sliding tolerance. However, this also meant that a new load path was required to transfer the lift shafts’ axial loads from the columns above into the centre of the concrete lintels. The Level 1 concrete beam was not sufficient to transfer this load alone, so additional Level 1 steel was provided.

At Level 1, load was transferred from the external columns through three RHS stubs per column. These stubs extended into existing hollowcore cavities, so the cavities were grout filled. The central column on each face of the lift shaft could not have stubs due to the high concentration of reinforcing in this column (including multiple post tensioned and grouted strands). Load from this column was instead transferred via friction with multiple tensioned MacAlloy bars. Load could also be transferred via bearing from the Level 2 concrete lintel, however this load path was not considered sufficiently reliable.

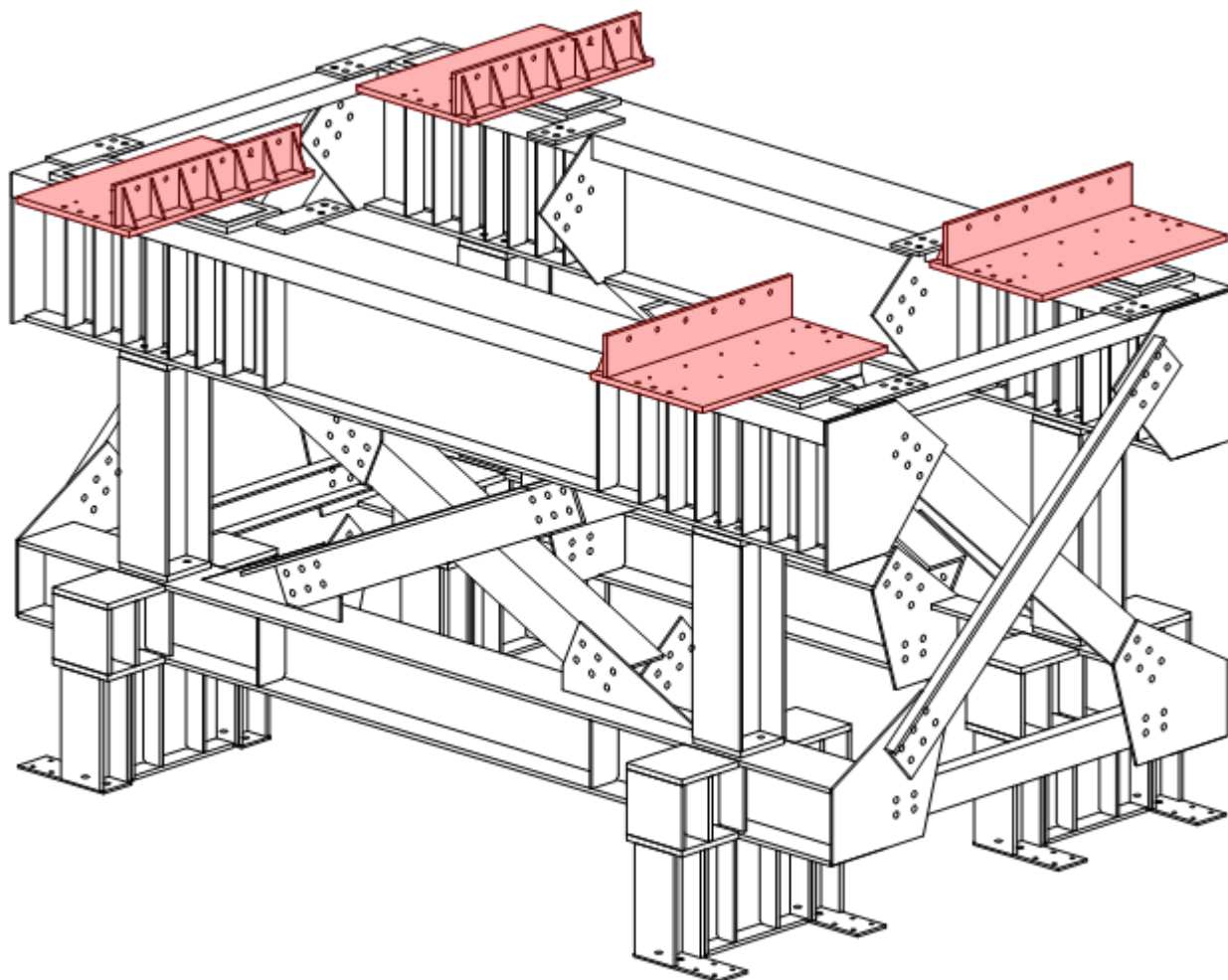


Figure 3: Render of North lift shaft steel below level 1 (MJH Engineering). Steel plate armouring above the isolation plane is shown shaded. Existing structure not shown.

2.2 Specific site challenges

The design was subject to several challenges, namely:

- The shoring had to be transported through the existing building. This resulted in the need for site assembly of most of the shoring. All welding was performed offsite, and bolted connections were adopted to minimise any hot works on site.
- Shoring of the lift shaft was designed and staged so that personnel were never required to be within the lift shaft until after the shaft had been secured. This was a key health and safety consideration.
- Ground floor shoring steel was preloaded to prevent the lift shaft dropping when the damaged columns were removed. This involved installation of flat jacks connected via a manifold to evenly raise each lift shaft.
- The design and construction timeframe was very short. To minimise delays, the steel contractor's shop drawings were adopted as the official steel design documentation (Figure 3). Beca documentation showing the intended construction sequence was produced separately.
- Allowance was made for how the permanent solution could be installed. This included considering how an acceptable clearance could be maintained between the new structure and the temporary steel over the course of the installation period (to accommodate horizontal base isolation displacement).

- Allowance was made for how the damaged existing concrete columns could be accessed and removed.
- Allowance for gravity support that could support 2600 kN and up to 400 mm horizontal movement in an earthquake.
- Provision of a sliding surface to minimise the actions in the lift shafts resulting from the movement of the building's base isolation system. Typically, friction coefficients for friction sliders can be lower than 5%. However, the materials available in the required timeframe provided a 20% friction coefficient. This required the temporary shoring to be able to withstand substantial lateral loading.

3 DISCUSSION

The rapid construction of the temporary shoring could not have been completed without exceptional co-ordination and dedication of multiple parties. This project relied heavily on working with the contractor (Fletcher Construction Ltd), the client (Victoria University Wellington), and the steel fabricator (MJH Engineering). The common objective and imminent completion date created an environment of cooperation and mutual problem solving. This is something that most projects strive for but rarely achieve.

The specifics around previous decisions made during the 2002 retrofit are not known. In this case the implications of non-isolated lift shafts across the isolation plane were significant. Although the Kaikoura earthquake did not create life safety issues in Rankine Brown, it nevertheless led to significant disruption and retrofitting cost while the building was occupied. While we have successfully mitigated the damage that occurred in Rankine Brown, we consider this a case example of why a building philosophy should be executed completely rather than 'mostly', particularly for base isolation where the full benefits could not be realised. The repercussions for Rankine Brown may be a useful illustrate example for design engineers who need to convince clients of completely committing to a holistic design philosophy.

4 CONCLUSIONS

This paper presents a unique form of structural damage that was a consequence of not completely base isolating a building. The temporary shoring solution and the challenges that were overcome are described. It was found that the short term advantage of not isolating in some areas of the building are far outweighed by the financial cost and building downtime that occurred after the Kaikoura earthquake.

5 REFERENCES

Stirling, M.W. et al. 2017. The Mw7.8 2016 Kaikoura Earthquake: Surface Fault Rupture and Seismic Hazard Context, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol 50(2) 73-84