Re-treading the rubber that hits the technical road

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Beca Ltd., Wellington

2017 NZSEE Conference

ABSTRACT: The first building in the world to be built on lead-rubber bearings (the former William Clayton Building in Wellington) has recently been extended and re-furbished 35 years after it was designed. It was a commercial imperative that the opportunity be taken during the re-development to deliver a structure that meets the full current seismic building standards if doing so was feasible financially. With our decades of experience in designing new base-isolated buildings, this would seem to be a relatively easy task. In fact, the progressive introduction of many new factors and theories, performance objectives, opinions and design methodologies, in the absence of authoritative and internationally-accepted guidelines for the design of base-isolated structures, made this task a real challenge. This paper describes some of those challenges and the outcome. A refurbished building, double the size of the original, that achieves 100*%NBS* at Importance Level 3 with only 50% of the base-isolators replaced.

1 INTRODUCTION

When the New Zealand "Loadings" Standard 1170 was finally published in 2004, we were very proud that we had a state of the art approach to seismic design, and we had addressed most of the issues that had come to light in the previous decade. The seismic zoning was now undertaken by reference to a hazard factor contour map that corrected the rather interesting one of the previous standard, and we even had a Near Fault factor introduced. Special Studies were allowed for situations where it could be shown that strict adherence to the Standard would not be appropriate. Our wise colleagues on the Standards committee decided that base isolation did not need to be addressed specifically because the principles and required performance prescribed in the Standard could be applied to this technology as well as it could be applied to conventional structural forms. We practised what would later be described as force-based design, and we stretched ourselves incrementally along the path to designing more seismically-resilient new structures. The few clients who wanted to seismically-strengthen existing structures were commonly reluctant to have this done to more than "two-thirds of Chapter 8" (NZSS 8 1965) which was not a particularly challenging level in retrospect.

In the last decade we have had the rise of displacement-based design, the assessment of existing buildings, and the market pushing for some older structures to be strengthened up to 100% New Building Standard, and even higher if it will lead to a higher return. Independent peer reviews of strengthening schemes are increasingly becoming the norm. Some other countries have introduced sections in their codes for the design of base isolation systems, and guidelines for these systems are under final preparation in New Zealand.

In today's changing scene, it is therefore a challenge to be asked to both assess the William Clayton building in Wellington (the first building in the world to have a lead-rubber base isolation system more than 35 years ago), and to design additions for it to meet a market wanting such a building to reach or exceed 100 % New Building Standard.

In this short paper, we briefly traverse some of the interesting decisions we needed to take in our journey. Construction of the refurbishment was completed in October 2016.

2 THE EXISTING BUILDING

Formerly known as William Clayton House, the case study building is on Molesworth Street adjacent to the Wellington motorway. The original building was a four-storey, reinforced-concrete, ductile frame structure isolated from the foundations by 79 lead-rubber bearings in a rectangular grid approximately 16 x 5. The building was designed as a ductile frame as a belt and braces approach in the event the base isolation could not be implemented.

The upgrade involved refurbishing the existing building of approximately 8000 m^2 of net lettable floor area (NLA) over four levels by adding a further 8000 m^2 of NLA on two extra levels and extension of some floor plates to cover the full building footprint.

The additional two storeys consist of two-way structural steel moment-resisting frames. Concrete spandrels form the perimeter of the building.

Figure 1: The William Clayton Building before it was extended

3 DESIGN DEVELOPMENT

The assessment and design of the building developed and changed as the project progressed, our knowledge of the base isolation system changed and a late increase of Importance Level from IL2 to IL3 was requested by the client. This development and change is briefly summarised below.

3.1 Designing to IL2

The initial design required the upgraded building to have a design life of 50yrs with an intended use as an office complex. Therefore a classification as *normal* importance (i.e., IL2) was considered appropriate.

The first stage of the design addressed the existing seismic gap in the base-isolation system. Based on theoretical properties of the existing LRBs it was determined that the existing 150mm wide seismic gap needed to be widened to 400mm to accommodate displacements consistent with current code requirements for an IL2 building. This package was designed and detailed to building consent stage. At this time testing of the existing LRBs had not been undertaken.

Following the design of the seismic gap widening, testing of two of the existing LRBs was undertaken with the aim to show that the existing bearings had adequate (displacement) capacity to resist Ultimate Limit State demands assuming an IL2 building. The existing bearings are different to today's standard bearings in that they transfer shear between base-plates and rubber via four steel spigots at each interface instead of via a vulcanising bond. Based on the outcome of this testing, a design using the existing bearings was progressed that limited the overall horizontal shear displacement of the LRBs to the maximum allowable found from testing. A series of horizontal Ring-feder friction springs were specified across the seismic gap around its perimeter to provide additional damping, buffering and a limit on the horizontal displacement of the isolation system.

In particular, the Ring-feder springs would reduce the effects of impact between the building superstructure and the surrounding retaining walls for earthquake shaking larger than expected at the Ultimate Limit State. Under this configuration a ductility of approximately 1.5 was required in the superstructure frame at the Ultimate Limit State.

Construction documents were drawn up on this basis and it was concluded that the building's structure (existing and proposed) and base isolation system achieved 100 % New Building Standard (*NBS*) assuming IL2 with the additional floor loading and no change to the existing base isolators.

3.2 **Changing to IL3**

Much later in the design process, after construction had begun the design team was asked to increase the resilience of the proposed structure to meet the higher IL3 performance objectives. As construction had commenced this change needed to be justified with little or no change to the detailing of the building extension or foundation widening, as these were partially constructed.

Seismic analyses confirmed that the replacement of the perimeter LRBs with new bearings with improved characteristics was preferable to achieve the higher performance requirements reliably. The detailed philosophy for this final design is outlined below.

- The seismic gap around the building at foundation level must be large enough to accommodate superstructure displacements (including an allowance for torsion) at the Ultimate Limit State (IL3).

- The superstructure should remain largely elastic under accelerations induced by the base-isolation system at the Ultimate Limit State. The overall building capacity was determined using a non-linear pushover analysis.

- For earthquake shaking larger than that corresponding to the Ultimate Limit State the building may run out of free travel (seismic gap) at some point beyond the Ultimate Limit State. At this point the building may impact with the retaining walls.

- The assessment was based on a $S_p = 0.7$ and a 5% torsional eccentricity allowance as per ASCE 7-10.

- At Ultimate Limit State displacements, including torsion, the shear displacement of existing bearings must remain below 250mm due to the uncertainty of behaviour of these bearings beyond this point.

- For displacements beyond the Ultimate Limit State further testing was carried out on the existing bearings with the aim to confirm that these bearings are capable of continuing to resist vertical load at 350-400mm displacement. Loss of lateral load carrying capacity at displacements larger than 250mm is considered acceptable. This testing was successful, however, had the testing indicated that the bearings were not capable of maintaining vertical load carrying capacity beyond 250mm a concrete or rubber buffer was to be installed in the seismic gap to limit displacements. The Ring-feder springs proposed under the IL2 option were not required in the IL3 scheme.

- All the new LRBs were designed to have a horizontal shear displacement capacity of 400mm to ensure the LRBs continue to perform up to the point of impact of the superstructure with the retaining walls on the outside of the seismic gap.

After completing further analysis on the basis of the above philosophy we concluded that with the replacement of 39 of the existing bearings with new bearings capable of 400mm of displacement the building could achieve IL3. As for the IL2 design the philosophy remained that sufficient seismic gap must be available for Ultimate Limit State displacements. However, impact with the surrounding walls for larger displacements is acceptable and the superstructure can survive the subsequent short-duration "shudder" due to both the existing and extended structure having ductile detailing.

On this basis, we considered that the intent of NZS 1170.5, where a building must have a reasonable margin on collapse beyond the Ultimate Limit State, was met.

Figure 2: The completed building

4 THE ISSUES

To settle on the philosophy described above we had to consider a number of issues for which there was no clear guidance given by the available standards. We therefore had to consider each item in conjunction with the peer reviewer and agree our approach.

The key issues that influenced the design and overall philosophy were:

- Site subsoil class on a varying site
- Apply a Near Fault Factor or not
- Properties of existing lead-rubber bearings, particularly their displacement capacity
- Selection of Structural Performance Factor for building and base isolation system
- Selection of structural damping for building and base isolation system
- Treatment of accidental torsion
- Constraints on size of "rattle" space.

Each of these are discussed in more detail below.

4.1 Site subsoil class on a varying site

The building sits on the side of the Wellington Motorway which runs along a widened natural gully. In this particular area, the depth to bedrock changes both across the site and along Molesworth Street towards the city. Whether the site subsoil classification is C or D makes a big difference to the *% NBS* "score" when assessing any building.

It is problematic to try to apply the criteria for site subsoil classification detailed in NZS1170.5.

To resolve this issue, an opinion was obtained from a specialist subconsultant, GNS Science, who deployed SPAC and Nakumura techniques to determine the site subsoil classification.

The subconsultant's recommendation that the site be considered '*C*' rather than '*D*' was adopted, although they noted based on the testing it was close to the threshold for D. Note that it would be contrary to NZS 1170.5 to adopt a site subsoil class between 'C' and '*D*' – even if the site testing suggests this to be appropriate.

4.2 Apply a Near Fault Factor or not?

The building is approximately 150 m from the fault zone of the Wellington Fault. For a new building with a comparable first natural period as the case study building, this would mean that a Near Fault Factor of approximately 1.2 should be applied. However, the base-isolated structure was checked using displacement-based design principles. The demand and capacity spectra for the base isolation system of the building presented in Figure 3 below have been derived from the design spectrum of NZS 1170.5 for buildings on Class C soil in Wellington City. Given that we estimated the average equivalent viscous damping to be 25% of critical, the elastic 5% damped displacement demand spectrum was scaled down using a factor *B* (Priestley, et al., 2007) to get the 25 % damped demand spectrum, where:

$$B = \left(\frac{0.07}{0.02 + \beta_{eq}}\right)^{\alpha}$$

As per the reference, the recommended coefficient α for sites located close to major faults is 0.25, and this was used instead of the Near Fault Factor which scales up the design loading for a Force-based design methodology. The use of this approach is consistent with the NZTA Bridge Manual 2010 which does not use a Near Fault Factor on the late Nigel Priestley's advice. This is consistent with the concept that a Near Fault Factor is applied to the Demand side for a Force-based design, whereas it is taken care of on the Capacity side for Displacement-based design.

Later following discussion with the peer reviewer it was agreed that both the Near Fault Factor and the reduced damping equation would be applied. This was considered to be the most conservative option.

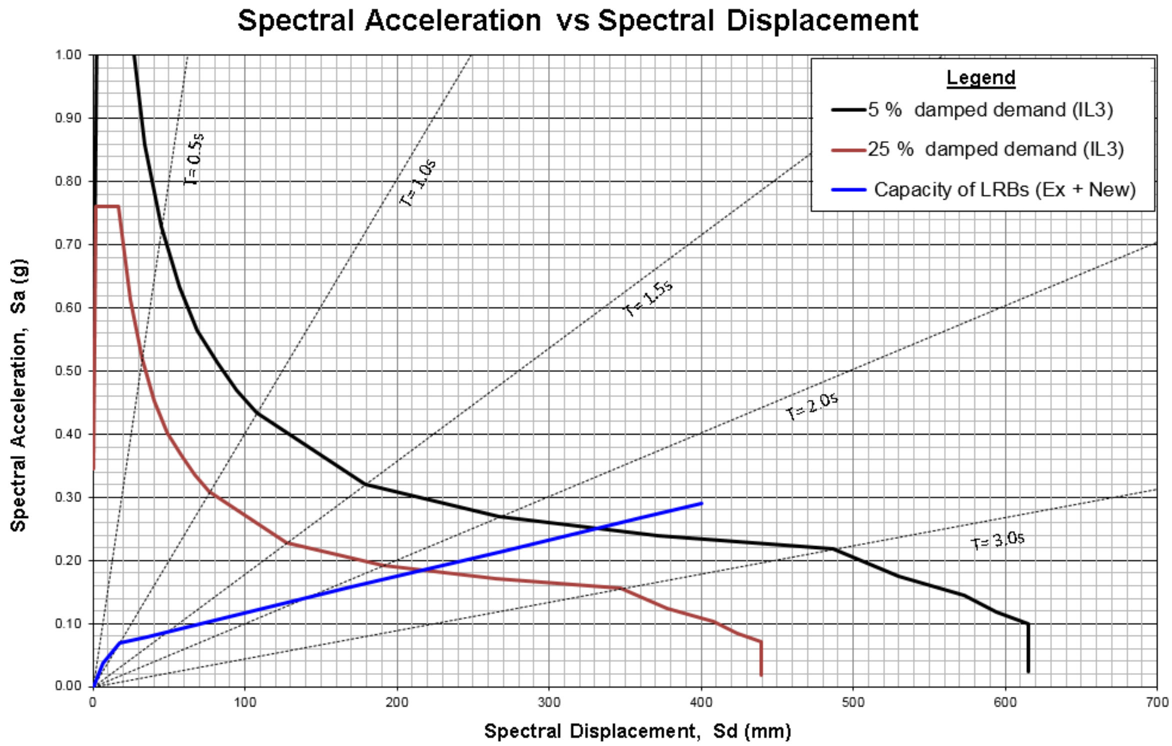

Figure 3: Demand and Capacity Spectra for the Base Isolation System

4.3 Properties of existing lead-rubber bearings

Although the existing bearings were found to be theoretically capable of not exceeding internationally-accepted norms for lead rubber bearings under the proposed design loads and displacements, it was recommended during the original IL2 design that a pair of the existing bearings be physically tested. This would show whether the bearings have the same characteristics as when tested nearly 40 years ago, and whether the now-superseded spigot connection at their top and bottom surfaces would perform satisfactorily under the increased demand.

This testing was conducted in Malaysia and one of the bearings was deliberately tested to failure. On the basis of this testing it was decided that the existing bearings have a maximum displacement capacity of 250mm, governed by the lightest vertical load case. While some rolling of the bearing was exhibited under the lightest vertical load at less than this displacement, this was not considered to affect the bearings' performance.

This limit of 250mm was used as the basis of the IL3 design. The IL3 design replaced all the perimeter bearings where the lightest vertical loads occurred and limited the displacement of the existing bearings to less than 250mm at the Ultimate Limit State including the effects of torsion. For displacements larger than those predicted at the Ultimate Limit State further testing was proposed on two more bearings to show that the existing bearings continue to carry axial load up to the maximum available seismic gap of 400mm.

It was expected that the bearings may exhibit some drop off in lateral load capacity at displacements larger than 250mm, due to the spigot connections allowing the bearing to roll. Our analysis indicated that with the new bearings designed to have a capacity up to 400mm the base isolation system had sufficient lateral stiffness without the existing bearings. Therefore, provided that the existing bearings continued to resist vertical loads up to 400mm our design criteria would be satisfied.

The testing was completed in mid-2016 only a few months before the building was due for completion. The testing was successful and the results indicated that the bearings continue to carry axial loads for displacements up to 400mm and also continue to carry lateral loads. This in spite of the fact that the bearings exhibited significant roll as shown in the photo below.

Figure 4: Existing lead-rubber-bearing test at 300mm displacement

4.4 Selection of Structural Performance Factor for building and base isolation system

A typical new base isolated structure in New Zealand is designed using an Sp factor of 1.0. In the new build context this may not have a significant impact on the overall design.

However, in the context of an existing building with a limited rattle space we felt that the selection of an Sp factor of 1.0 was too conservative. We therefore elected to complete the design using an Sp factor of 0.7. This was chosen in line with the principles and criteria outlined in the current New Zealand Loadings Code NZS 1170.5 where buildings with a ductility of greater than 2 can be designed/assessed using an Sp of 0.7. Both the superstructure and base isolation system have a ductility of greater than 2 so Sp=0.7 was used for both the design of the superstructure and base isolation assessment and design.

4.5 Selection of structural damping for building and base isolation system

Damping capacity of the current base-isolation system was determined based on the weighted average of the experimentally determined capacities of both existing and new LRBs. The results of three sets of physical tests were available for the assessment of the existing bearings. The first set of tests were undertaken in 1980 on bearings that were randomly selected from those manufactured for the building (Robinson & Tucker, 1981). The recent two set of tests were undertaken recently using bearings removed from the building and a spare bearing that had never been installed. In the 1980 physical tests, the bearings were subjected to dynamic loading and tested to a maximum shear displacement of 110mm, while the bearings in the recent two sets of tests were subjected to a maximum shear displacement of 250-400mm. However, the recent tests were quasi-static because of the limitations of the testing equipment.

When sheared horizontally, the plastic deformation of the lead core and material damping of the LRBs dampen motions. LRBs dissipate energy principally as heat through plastic deformation of the lead core. The mechanical properties of the lead core are continually recoverable from one cycle to the next if tight confinement is maintained. Displacement-dependent damping due to plastic deformation of the lead core (hysteretic damping) is typically determined by measuring the area within the force-displacement loops from the results of quasi-static tests as an equivalent viscous damping ratio. Damping arising from repeated elastic straining and internal friction in the rubber (and lead core) could ideally be determined from dynamic physical tests (Chopra, 1995).

A 5% of critical viscous damping of the structure was considered justifiable for the design of the building in addition to the quasi-static hysteretic damping of the LRBs. It should be noted that the damping value adopted for the existing LRBs was in good agreement with the critical total damping measured from the 1980 dynamic tests.

4.6 Treatment of accidental torsion

In the absence of a local base-isolation standard, a 5% accidental eccentricity was considered for the design of the building in line with current practice in New Zealand and internationally-accepted base-isolation design standards (ASCE 2010, EN 2004). Provided the symmetry of the base-isolation system of the building in terms of both strength and stiffness in both principal directions, consideration of a 10% eccentricity as per NZS 1170.5:2004 was considered too severe for the design of such a regular building with a reasonably symmetric distribution of seismic mass due to dead load.

4.7 Constraints on size of "rattle" space.

100*%NBS* at IL3 implies that the seismic resilience of the structure meets the intent of both B1 of the Building Code and NZS 1170.5:2004 for an Importance Level 3 building. That is, the building's structure will have a significant margin of resilience above the 1000-year return period design-level (Ultimate Level State) earthquake. We note that a new base-isolated structure designed in New Zealand today may have seismic gaps sized to accommodate displacements expected during a 2500-year return period event.

Our analysis indicated that even with the widened seismic gap, which was widened to a practical maximum of 400mm, we were not able to achieve a gap to meet expected displacements expected during a 2500-year return period event. We therefore expected that the building may hit the surrounding retaining walls in shaking larger than the 1000-year return period event.

To assess the effect of the building hitting the surrounding retaining wall a series of 2D inelastic time history analyses on a typical building cross section were completed. The time-history analyses indicated that closing of the seismic gap may occur in events larger than the 1000-year return period event and these will induce momentarily large accelerations in the superstructure. However, the ductile detailing of the superstructure is considered capable of absorbing these accelerations without collapse. On this basis, we considered that the intent of NZS 1170.5, where a building must have a reasonable margin on collapse beyond the Ultimate Limit State, is met.

5 CONCLUSIONS

In the last decade we have honed our thoughts on what material factors we use when assessing a structure, and whether we should stick to the factors we use for new building design. In our particular case, a number of the Force-Based Design provisions of NZS1170.5:2004 have been challenged when we were faced with extending a base-isolated building. The common approach in New Zealand now to base isolation design is to apply Displacement-Based Design principles.

When an existing building is extended substantially, and it already contains ductile elements (a base isolation system) with reasonably predictable characteristics, there is a good case for minimising the Structural Performance Factor adopted. In addition, there seems to be no reason not to allow the base isolation system's gap to close up briefly for intensities of shaking above the Ultimate Limit State level, provided the impact is minimised and the performance objectives for a building of the chosen importance can be shown to have been met.

Although the testing was limited to four bearings, lead-rubber bearings manufactured more than 30 years ago seem to have retained their properties with time.

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