INSIGHTS FROM INTENSIVE ASSESSMENT ANALYSES – THE STEPS TO TARGETED PERFORMANCE ENHANCEMENT FOR A CHRISTCHURCH DUCTILE RC MOMENT-FRAME BUILDING

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Abstract

The re-instatement of the former Commerce Building at the University of Canterbury is an example of an intensive post-earthquake performance assessment, followed by a broad-scope seismic strengthening and performance enhancement project. The primary capacity-designed ductile reinforced concrete moment-frames suffered moderate damage during the Canterbury earthquake sequence due to unintended precast façade panel interactions. The subsequent performance assessment highlighted excessive storey drifts and significant diaphragm weaknesses under design level ground motions, with potential residual capacity issues also being accounted for in the assessment.

The assessment phase included extensive non-linear response history analyses that captured aspects of capacity and detailing noted from drawing reviews, hand-calculations and observed damage. A suite of incremental non-linear dynamic analyses was able to identify the stages of failure and limitations of the building, highlighting that the retrofit had to target both deformation control to limit associated damage to existing elements, and global strength to satisfy current code design earthquake requirements.

Now in construction, internal steel frames incorporating fluid-viscous dampers and buckling-restrained braces provide significantly reduced storey drifts, while meeting the code base-shear for the 1000 year return period ground motion hazard. Non-linear response history analyses with inelastic parameters defined via ASCE 41-13 supplied the necessary modelling information to demonstrate that this 'undefined' system satisfied NZ code minimum requirements.

Introduction and Building Overview

The Education Building (formerly Commerce Building) at the University of Canterbury is a mid-1990s ductile reinforced concrete two-way moment-frame. Although constructed as one building it is essentially two blocks, the North Block being six storeys plus a light-weight roof and the South Block being eight storeys with a concrete slab roof level (Figure 1). The blocks are connected by a central floor area that contained lifts and stairs, lobby spaces and plant rooms. The floor system throughout the whole building uses 200mm hollowcore units with nominal 70mm topping slab reinforced with non-ductile 665 cold-drawn mesh, without starter bars from the moment-frame beams.

The building has an occupied basement level, however the basement walls are not connected to the lateral force-resisting system and act as retaining walls only, with the water-table typically 1.5m above the basement floor level. The foundations are a series of individual capacity-designed pads with tie-beams matching the frame layout, set on a significant and competent gravel layer.

The moment-frames are consistent and regular over the plan and height of both blocks, with 450 x 800 mm deep beams, 500 x 700 mm deep columns and joint regions generally well detailed and essentially meeting the current Concrete Structures Design Standard (NZS 3101:2006). During the 2010-2011 Canterbury earthquake sequence the moment-frames suffered limited damage due to the precast concrete façade panel system either not moving freely within their slotted connections, or having insufficient gaps between panels. On most faces of the building the panels locked-up with one-another or within their fixings, providing an unintended shear panel system that limited the movement

of the building and protected the moment-frames. Many of the panels and fixings were significantly damaged but none lost their support, likely due to the short duration of strong ground motion in each event.

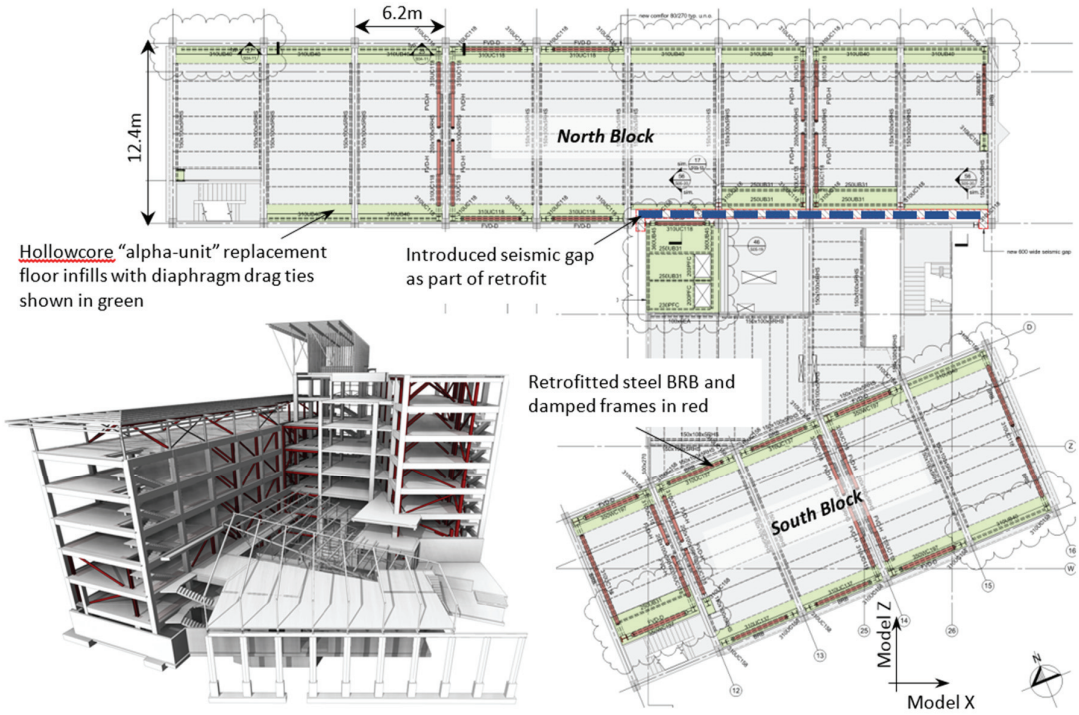


Figure 1. Perspective and plan views of the primary structure including strengthening structural steel and supplemental damping elements (red) and new floor diaphragm edge infills (green).

Assessment of the Building and Expected Performance

As part of the overall university campus post-earthquake building assessments, this building was the focus of an extended series of seismic capacity and performance reviews. These progressed from fundamental capacity and detailing checks, through to linear dynamic and finally non-linear dynamic models. Early reviews identified that the weak-beam strong-column mechanism was reliable up to the design basis ground motion, however a simple lateral mechanism assessment would have left some key structural weaknesses untested by satisfactory means. The non-linear assessment drew from the learnings of the previous phases, however rather than using the earlier reviews to simplify the modelling it is suggested that these are better used to direct additional details to include the non-linear models.

With the imminent adoption of the ASCE 41-13 (2014) backbone and performance limit-state guidance into the upcoming revision to the New Zealand assessment guidelines (NZSEE, 2016) this project provided a timely demonstration of the benefits obtained from a non-linear dynamic approach to assessment. Much of the new guideline draft focuses on promoting assessment using a simple static hand-calculated method, however with recognized non-linear modelling guidance soon-to-be available within a New Zealand document, it is hoped that many of the perceived hurdles that have limited the uptake of non-linear response history (NLRH) analysis in New Zealand will soon be overcome.

For this project the primary moment-frame elements used conforming backbone curves developed from ASCE 41-13, and performance limit-states also derived from this document. The progression from hand calculation review through to intensive non-linear analysis highlighted that fundamentals-

based review provides a means for initial identification of potential performance and capacity limitations, but that these are often insufficient to capture the complicated dynamic system interactions that are usually present in existing structures with a mixture of contributions to the lateral-force resistance, or dynamic characteristics.

A key non-linear model inclusion was the representation of the limited diaphragm capacity through the central core area using shear panel elements with degrading strength and stiffness. The diaphragm capacity had been reviewed with hand-calculations, that confirmed its limited capacity and the mesh reinforcement inability to maintain capacity if significant cracking developed. This focus on diaphragm strength also brought into question aspects of assessing residual capacity given the observed cracking and non-ductile mesh reinforcement behaviour. In some locations the mesh had fractured (generally continuous north-south running cracks at the hollowcore unit seating ends) and would require replacement reinforcement, however many cracks around the 'alpha-unit' (the first hollowcore floor unit adjacent to parallel spanning moment-frame beams) also exceeded 0.4mm. Post-earthquake assessments in Christchurch consistently identified this as correlating to mesh yield, with crack widths exceeding 0.6mm generally considered to indicate compromised mesh reinforcing.

Initial concepts for the retrofit scheme considered whether the diaphragms would be sufficient if allowing for some existing plastic strain at the perimeter interfaces. It was assumed that additional chased-in starter bars from the beams to the topping slab. However as the concept progressed to preliminary design it was decided that full alpha-unit removal provided not only construction access for the new steel frames, but also an opportunity to introduce a specifically designed drag-tie beam and thin spanning slab that met NZS 3101:2006 requirements.

The NLRH analyses were carried out in ANSR (Mondkar and Powell, 1979) and worked through a series of increasing ground motion intensities, essentially building up a 12-step incremental dynamic analysis (IDA) from 8% to 100% of the Design Basis Earthquake (DBE). Due to occupant numbers the building is an Importance Level 3 (IL3) structure, for which NZS1170.5 (New Zealand Standards, 2004) sets the DBE as a 1000 year return period event. At this level of demand, Life-Safety is expected to be achieved, which is also termed by NZS1170.5 as the Ultimate Limit State (ULS). The NLRH results captured not only the dynamic interaction and higher-modes of the rather flexible lateral system, but also allowed the hierarchy of capacity limitations to be well defined, particularly where influenced by the dynamic characteristics of the building. This ultimately helped guide the design sequence for strengthening and performance enhancement.

Following NZS1170.5, the envelope of the maximum results from four earthquake records were used for both deformation- and force-related checks. The record scaling satisfied NZS1170.5 requirements, and the records were selected to represent the general conditions of seismic hazard to the site including one nearby recording from the September 4th, 2010 Darfield Earthquake that contains forward directivity characteristics.

The outcomes of the IDA phase were:

- The hollowcore seating limit was reached at demands below 33% DBE. This defined the governing building capacity limit, and set the first retrofit decision point.
- The core-area diaphragm Life-Safety (ULS) shear strain limit was reached at 50% DBE. Beyond this intensity, rapid strength and stiffness degradation developed to the Collapse Limit State at 67% DBE. This provided the second retrofit decision point.
- The building flexibility and drift response limited the building to 65% DBE. P-Δ effects became significant beyond that, as shown by the rapid increase of maximum drifts to an excess of 5% at 100% DBE.
- The ductile weak-beam strong-column mechanism was maintained throughout the IDA, however an excessive number of beam elements exceeded Life-Safety (ULS) plastic rotations beyond 60% DBE.

- The overall base-shear capacity of the building was below current NZS1170.5 IL3 ULS (DBE) demands. This was also verified through static push-over analyses showing that at 2.5% peak roof drift the building could develop approximately 70% DBE base-shear.

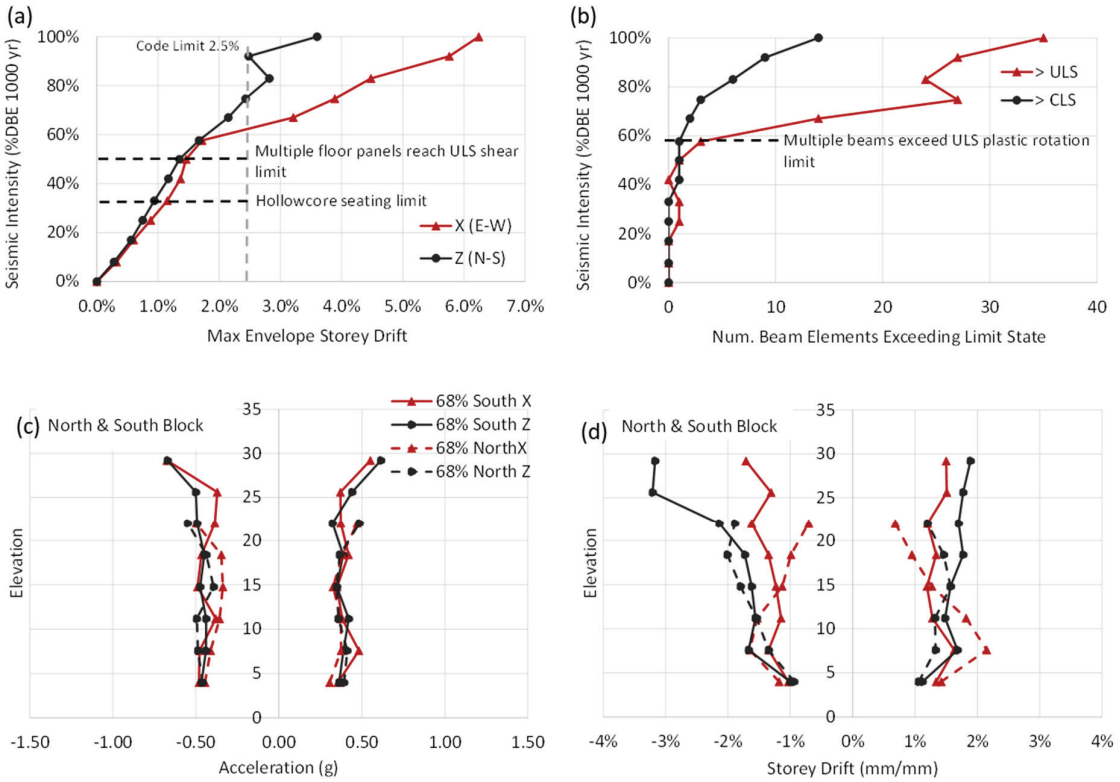


Figure 2. (a & b) Incremental non-linear dynamic analysis result summary with key performance points noted (c & d) floor acceleration and storey drift maximums at 68% DBE.

It is noted in Figure 2c that the peak floor acceleration profiles were quite low due to the very long (X = 2.0 and Z = 2.3 seconds) fundamental periods of the original building.

Development of a Performance-Based Retrofit Scheme

Two key steps were taken to improve the seismic performance of the building before any supplemental system design was started. Firstly, the addition of hollow-section steel members bolted to the face of the transverse concrete beams provided back-up seating to the hollowcore floor units at all levels where the MCE drift of the retrofitted structure exceeded 0.5%.

The central core diaphragm capacity was then reviewed against a range of potential strengthening options, including rebuilding the core floors. The decision to separate the North and South Blocks (Figure 1) was made based on the cost of floor replacement and limited prospects for successful FRP strengthening performance, given the likely slab cracking, localized rotations and out-of-plane deformations around the moment-frame beams that the strips would have to accommodate. A 600 mm seismic gap was assigned immediately north of the moment-frame along the north edge of the core area, such that the North Block was left as a simple rectangular building. The South Block could benefit from the moment-frame on this gridline remaining with the core, to help restrain the in-plan asymmetric response.

With these primary limiting factors accounted for, the final retrofit scheme provided both seismic performance enhancement based on displacement-focused decisions as well as force-based

considerations to meet NZ code minimum requirements, discussed below. A combination of fluid viscous-damped (FVD) steel frames, and buckling-restrained brace (BRB) steel frames provided significant drift reductions with final peak DBE storey drifts down to 1.5%, that were reasonably even in distribution over the building profiles.

Displacement-Based Aspects of Designing for Improved Performance. The fundamental point of entry into the retrofit design process was to determine a satisfactory limiting ductility for the existing reinforcement concrete moment-frames. While the post-earthquake assessment had found very little indication of damage in the frame elements, some cracking in the potential beam-end plastic-hinge regions had been observed. It was a conscious decision to maintain a nominal to moderate ductility demand on the beams with the retrofit scheme in-place. Using the simple frame equations by Priestley et al. (2007), the yield drift in each direction (for both buildings) was found to be 1.6% (N-S) and 0.8% (E-W). Clearly the shorter bay-length E-W frames determine the maximum drift to be targeted if the ductility demand is to be limited, and with the target DBE drift at 1.5% the expected maximum ductility demands were approximately $\mu = 1.0$ and 1.9, respectively.

A further consideration was that large drifts at DBE demands would result in a number of structural vulnerabilities associated to beam reinforcement strains and hollowcore seating. Hollowcore damage due to beam twist across the transverse span of the building, and frame elongation at the two-way frame corners (Fenwick et al., 2011), provided targets for limiting deformation incompatibility-induced damage and beam reinforcement plastic strains. A further factor that potentially affected the beam reinforcement strain capacity was foundation settlement-induced beam rotations. Floor surveys gave indications of some foundation settlement (although not confirmed to be earthquake induced), so limiting the potential beam reinforcement plastic strains also provided a means to not exacerbating any such initial conditions for the retrofitted structure.

The initial re-instatement plan was to re-use or replace the precast façade panels. Limiting the storey drifts was seen as a way of reducing the difficulties around movement details, particularly at corner intersections. The final façade design moved away from precast panels, however the fundamental aspects of movement joints and clearances still had to be reviewed for both DBE and MCE (150% DBE) movements.

With all of these factors reviewed, the target 1.5% drift was carried into the direct displacement-based design process. The first outcome from this process was that significantly more than 30% equivalent viscous damping would be required, which could not be provided by the FVDs alone. This brought the BRB frames into the design, from which a target reduced period to meet the design displacement with an estimate for the combined equivalent viscous damping value was found. The required BRB frame stiffness was evaluated, and from this the brace and supporting frame sizes developed.

Although the existing limited ductility reinforced concrete moment-frame would likely reduce the potential for residual drift concentrations, the BRB frames were designed to a maximum ductility $\mu = 3$ so that potential residual frame offsets were reduced. This decision is an extension from previous research into dual-system approaches to limiting residual drifts in BRB frames (Pettinga et al, 2007). Given the brace geometry within the existing concrete frames and the likely yield drift of the braced frames (~ 0.5%), this target BRB ductility also matched well with the chosen 1.5% drift limit.

Review of the BRB frame contribution and code displacement demand provided the updated viscous damping target in each building. Both the North and South Block buildings required 15% (east-west) and 25% (north-south) supplemental damping to reduce the DBE displacements to 1.5%. The initial distribution of damping coefficients was determined using the approach outlined by Christopoulos and Filiatrault (2006) using a trial velocity coefficient $\alpha = 1.0$, after which the distribution was refined and simplified using a revised velocity coefficient $\alpha = 0.5$.

Force-Based Aspects of Strengthening Design. The non-linear response history assessment phase had identified that the base-shear capacity of the original building did not meet current NZS 1170.5 minimums. Further push-over analyses confirmed this once the buildings were separated by the new seismic gap (Figure 1).

To even-up storey drift distributions, the buckling restrained braced frames were introduced over the bottom half of each building only. The BRB frames served to reduce the fundamental building periods and displacements, but also provided an opportunity to raise the base-shear capacity to meet the current NZS 1170.5 ULS minimum.

Validation and Review to Satisfy Code and Beyond

The New Zealand design Standards and Building Code do not contain guidance on the design and analysis of fluid viscous damped frames, therefore NLRH verification is currently the only path to demonstrating code compliance via alternative methods. Currently NZS1170.5 requires a minimum of three scaled earthquake record pairs to be used, with the maximum envelope being the only code allowed combination to evaluate these results (i.e. an average of seven or more does not exist in the NZ Standard context). For this project the verification analysis used four records, provided by the geotechnical consultant for the assessment phase.

The MCE analyses were carried out as a means to confirm the collapse prevention characteristics of the retrofitted structures. These also provided a means to specifying the capacity, velocity and stroke of the FVD units following ASCE 7-10 (ASCE Standard, 2011), and reviewing aspects of capacity design for the connections of the new steel frames.

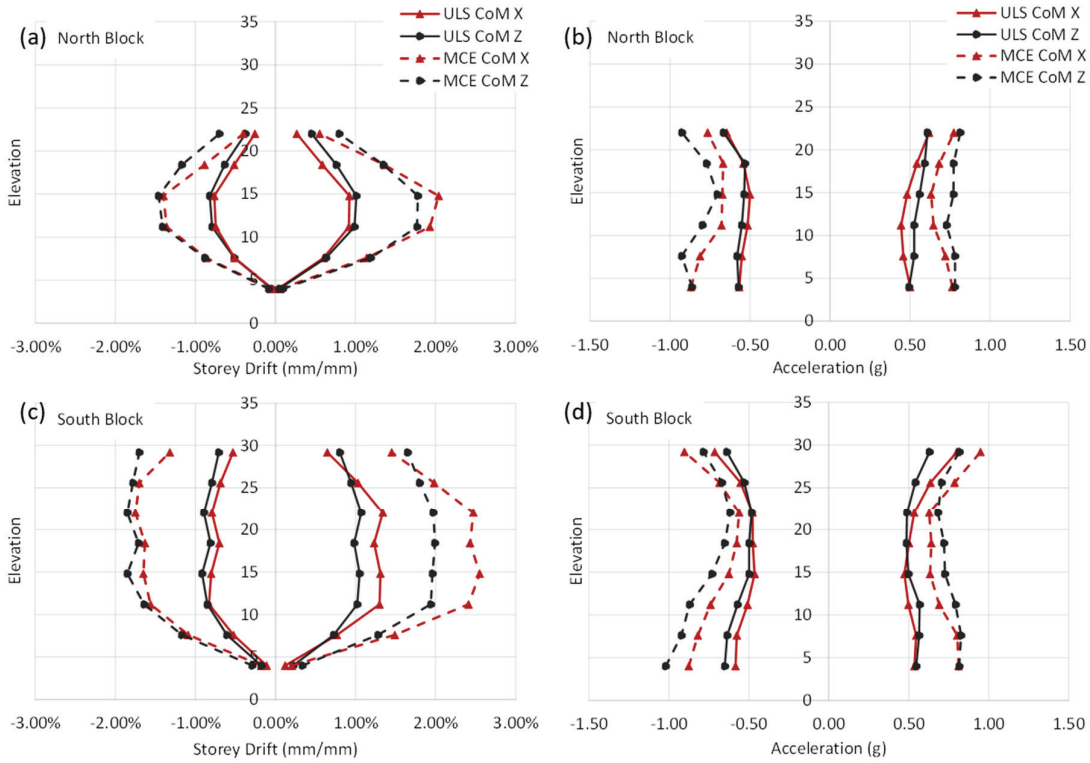


Figure 3. Final maximum diaphragm centre-of-mass 'CoM' storey drift (a & c) and acceleration profiles (b & d) at Ultimate Limit State (1000 yr RP DBE) and MCE (150% DBE).

Retrofitted Performance Summary

The retrofit performance was evaluated using the traditional metrics of storey drift and floor accelerations, along with evaluation of the various ASCE 41-13 performance limit-state parameters.

As identified in Figure 2, the drift limit was the fundamental aspect of performance that needed to be addressed through the addition of the new damped or braced steel frames. The target DBE peak storey drift was 1.5% and as the plots in Figure 3a & c show, this was achieved (noting that the corner vector drift not shown here was at 1.5%, the peak X or Z values were less than this). The MCE drifts were also well controlled and below 3% in the worst case. The implication from this is that the existing reinforced concrete frames would maintain their gravity load carrying capacity, however it is possible that at some levels the hollowcore units would lose their original seating and require the catch-beams installed under them.

The peak floor accelerations in Figure 3(b) & (d) are generally consistent over the height of each building, with a limited higher-mode roof level 'kick' noted. This information was used as input to the Parts & Portions design information passed on to the mechanical and services engineer.

The performance limit-state evaluation confirmed that the retrofitted buildings were able to meet the ULS Life-Safety requirements. The limit-state envelope (combined from all earthquake records) in Figure 4(a) provides an overview of the distribution of peak damage, which was confined to the reinforced concrete moment-frame beams and BRB elements.

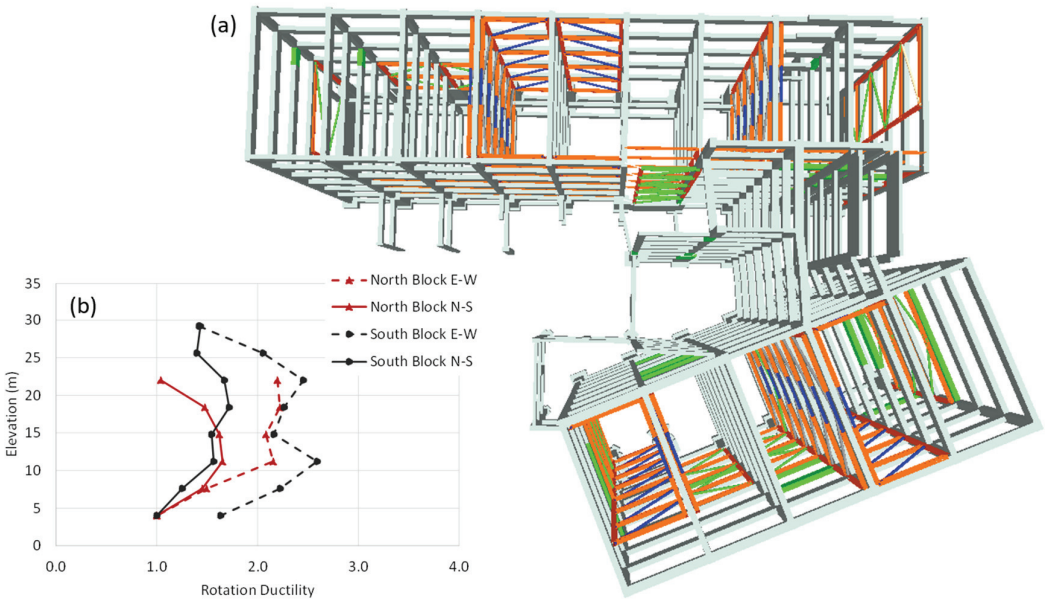


Figure 4. (a) Damage envelope from ANSR showing the ASCE 41-13 damage limit states at DBE demand (green ≥ Immediate Occupancy, the blue elements shown are the fluid viscous dampers which do not correlate to damage limit state colours). (b) Distribution of maximum reinforced concrete beam rotation ductility for each building in the principal directions.

The damage exceeded ASCE 41-13 Immediate Occupancy limits, but did not exceed Life Safety limits. Figure 4(b) provides the distribution of rotation ductility demands in the concrete beams. With reference to the expected ductilities, at the target 1.5% drift discussed earlier, it is noted that the peak ductilities exceed the targets however these were localized and generally across a floor level the ductility was within 15% to the expected values noted earlier.

In Summary

The assessment of the former Commerce Building at the University of Canterbury provides a case-study that demonstrates the benefits of a progressively more intensive and detailed evaluation of structural seismic performance, particularly in-light of the upcoming adoption of ASCE 41-13 non-linear analysis guidance into a New Zealand document.

The initial review and hand-calculation capacity checks demonstrated the basic seismic frame capacity design hierarchy. However these highlighted that the limited diaphragm capacity of the precast hollowcore floor system would interact with dynamic behaviour of the two offset building wings and that such response could not be adequately captured without non-linear response history analysis. An incremental approach to reviewing performance over a range of ground motion intensities allowed the key targets for performance enhancement to be clearly identified and a hierarchy for the retrofit sequence to be defined. This hierarchy information, presented to the Client, was an aid to setting the retrofit target at 100% of current Code demand.

Hollowcore seating and diaphragm capacity between the two wings were the early governing factors in the building capacity, and separating the building into two separate blocks provided a means to simplifying the subsequent retrofit scheme. This decision was helped by knowledge that a combination of supplemental viscous damping and limited additional stiffness provided by buckling-restrained braces, a unique retrofit combination in New Zealand, could control the drift performance without having to account for the building interaction across the core floor area.

The final hybrid-system retrofit provided an involved but complete means to raising the buildings' capacity to 100% of current Code demand, and through the use of non-linear response history analysis at DBE and MCE it was demonstrated that the overall new system also satisfied the fundamental performance requirements for a new building designed to NZ Standards.

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