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CHRISTCHURCH JUSTICE AND EMERGENCY SERVICES PRECINCT CASE STUDY

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ABSTRACT

The Christchurch Justice and Emergency Services Precinct is the first significant central government Anchor Project to be built in Christchurch since the 2010/2011 Canterbury Earthquakes. The project brings together all Justice and Emergency services in one purpose-built, base-isolated precinct in central Christchurch. It is the largest multi-agency government co-location project in New Zealand's history and consists of four seismically separated two-way steel moment resisting framing structures supported on a common first floor base isolated podium. The building is an NZS 1170 Importance Level 4 structure and performance based design has been used to ensure adequate building performance at 500 year (SLS2), 2500 year (ULS) and 7500 year (CLS) return periods. This paper summarises the design approach adopted for the project to meet the relevant local and international standards, incorporating the benefits of non-linear time history validation and composite construction.

Introduction

The Christchurch Justice and Emergency Services Precinct is the first significant Anchor Project, defined in the Christchurch Central City Development Unit 2012 city plan, to start construction as part of the Christchurch central city re-build. With a construction budget of circa $300 million, and a gross floor of 40,000 m^2, it is the largest multi-agency government co-location project in New Zealand's history. The project will bring together accommodation requirements for the Ministry of Justice, New Zealand Police, Department of Corrections, St John New Zealand, New Zealand Fire Service, Ministry of Civil Defence , Emergency Management and other agencies. Fletcher Construction Company is leading the construction team.

The project is designed as an AS/NZS 1170.0 (Standards New Zealand, 2002) Importance Level 4 facility. This is the level required for aspects of the Police and Emergency Services within the precinct. Contained within the complex there will be an Emergency Operations Centre (EOC) that will provide coordinated interagency support to the community in the event of public emergencies. The minimum requirements for the Justice components of the project are generally Importance Level 3, but application of Importance Level 4 loads will mitigate the risk of building damage and improve functionality in the event of a large earthquake.

The buildings incorporate low damage design principles. Multiple buildings accommodating the Justice and Emergency Services programme are to be located with a common podium / courtyard area (refer Figure 1). This podium and the buildings above are base isolated, with the isolators contained within the ground floor.

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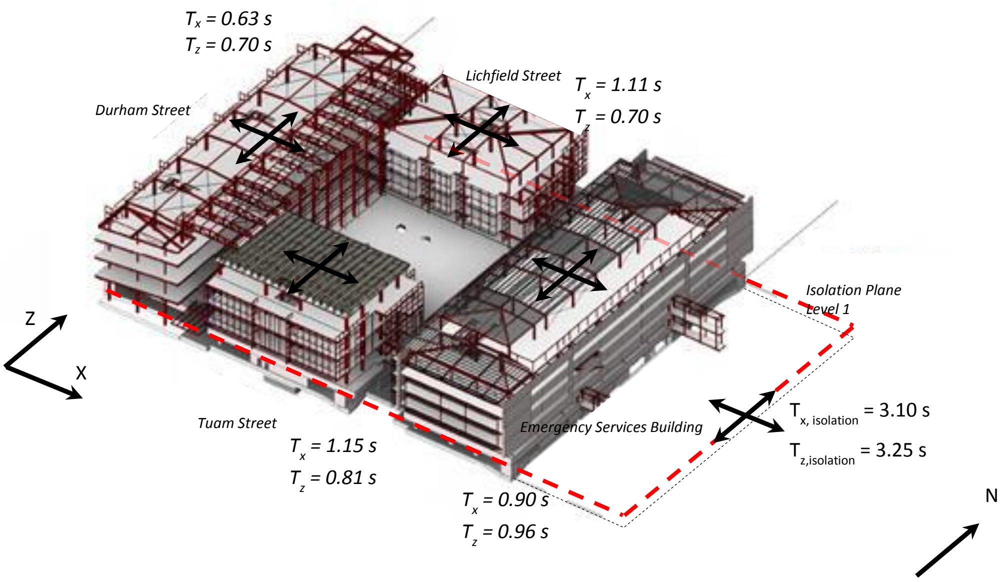

Figure 1. Overview of the Christchurch Justice and Emergency Services Precinct. Individual fixed-base building periods and isolated system period are shown.

The development also includes an Emergency Services car-park building (not discussed further here) which under AS/NZS 1170.0 (Standards New Zealand, 2002) is also defined as an Importance Level 4 facility, and utilises dual Moment-Frame and Buckling Restrained Brace lateral force-resisting systems.

Building and Structural Description

General

The structure consists of four 4 storey high buildings constructed over a common podium (refer Figures 1 - 3). Tuam, Durham and Lichfield buildings compromise the Justice accommodation and will contain the High Court, District Court, Family Court, Youth Court, Environment Court and the Maori Land Court, mediation and dispute resolution services, registry services, as well as Judicial Chambers and necessary support functions. The design includes an allowance to add an additional level on the southern Tuam building should additional court space be required in the future. Figure 3 illustrates a typical east – west building section.

Figure 2. Photograph of the building under construction

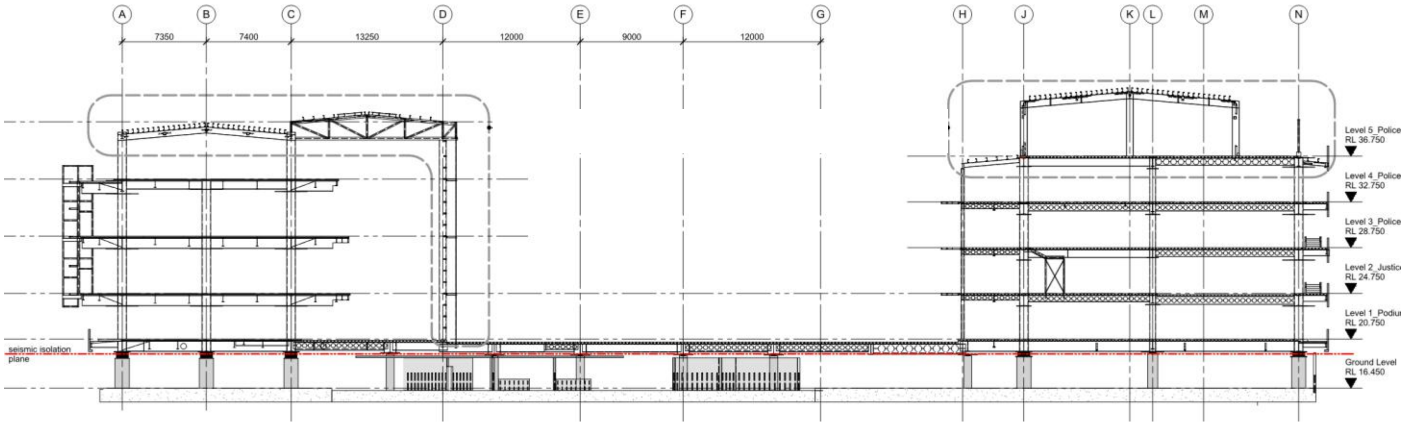

Figure 3. Typical east – west building section.

The majority of the remainder of the accommodation on the site, for the New Zealand Police and Emergency Services, is contained in a large building that occupies the central portion of the overall site, eastern side of the courtyard.

Primary Lateral Load Resisting System

Superstructure

The primary lateral load resisting system above the isolation plane for all four buildings consists of two-way structural steel moment resisting frames. The moment resisting frames were detailed as NZS 3404 (Standards New Zealand, 1997) limited ductility Category 2 seismic resisting systems. While the base isolation system was effective at mitigating yielding of the moment resisting frames under ultimate limit state (ULS) loading (2500 year return period), capacity design was undertaken to ensure resilience should the building be subject to higher loads. Under maximum considered earthquake (MCE) loading (7500 year return period) the superstructure is expected to sustain displacement ductility demands of up to 2.0.

Columns consisted of concrete filled circular hollow section (also referred to as concrete filled tubes, CFT). This construction methodology was chosen to improve the efficiency of the two-way moment-frames, take advantage of the benefits of composite concrete construction and reduce the need for passive fire protection. Typically columns were 750 in diameter with wall thicknesses ranging from 12 mm to 20 mm (refer Figure 4). The columns were detailed as NZS 3404:1997 Category 3 members. In the absence of a specific NZ Standard for CFT design the strength design was carried out using Eurocode 4 (BS EN, 2004) and the CIDECT guidelines (Bergmann et al, 1995).

Figure 4. Two way moment resisting system adopted for the Christchurch Justice and Emergency Services Precinct project

To satisfy the steel material requirements for Category 2 (i.e. for the column bases) per Table 12.2.6 in NZS 3404:1997 (and Table 2 of NZS 3404.1:2009), the CHS column material specification was Grade S355J2H. It is noted that standard G350LO hollow-sections available in New Zealand do not satisfy the material requirements for Category 1, 2 or 3 structures, although Grade J2 is known to meet Type 6 limits when tested.

Beams consisted of Category 2 G300 welded sections which varied in depth from 700 mm to 900 mm. Section sizes were governed by deflection considerations so the beams were detailed with reduced beam sections (RBS or 'dog-bone sections') designed according to SCNZ EQK1003 (Cowie, 2010). Beam-column joints were fabricated using continuous external collars, designed using the guidance in SCNZ CON1002 (Cowie, 2009), and web thru-plates. This allowed for consistency in the fabrication process and clear means for providing moment connections to the cantilever floor beams that occurred around the perimeter of each building.

Base Isolation System

The base isolation system is of a combination of Lead-Rubber Bearings (LRBs) and flat-plate pot-bearing PTFE sliders. It is noted that other low damage design systems including the use of fluid viscous dampers (FVDs) and buckling restrained braced frames (BRBFs) were considered during the concept and preliminary design phases. These low damage design systems were considered on the basis that they have been extensively tested, successfully used in California, Japan and Taiwan; and have performed very well in previous earthquakes. Base isolation was ultimately adopted on the basis that it provides a very high level of structural performance and is effective at mitigating seismic damage to superstructure elements and building contents.

LRB's (1020mm diameter) were distributed under the main seismic column lines in the footprint of each tower while the PTFE sliders were used under the primary frame columns where significant uplift forces were expected and throughout the courtyard extent (refer Figure 5). The decision to use sliders at all courtyard locations allowed for uncertainties in the final landscaping gravity load distribution, while also enhancing the level of energy dissipation for the isolation system as a whole.

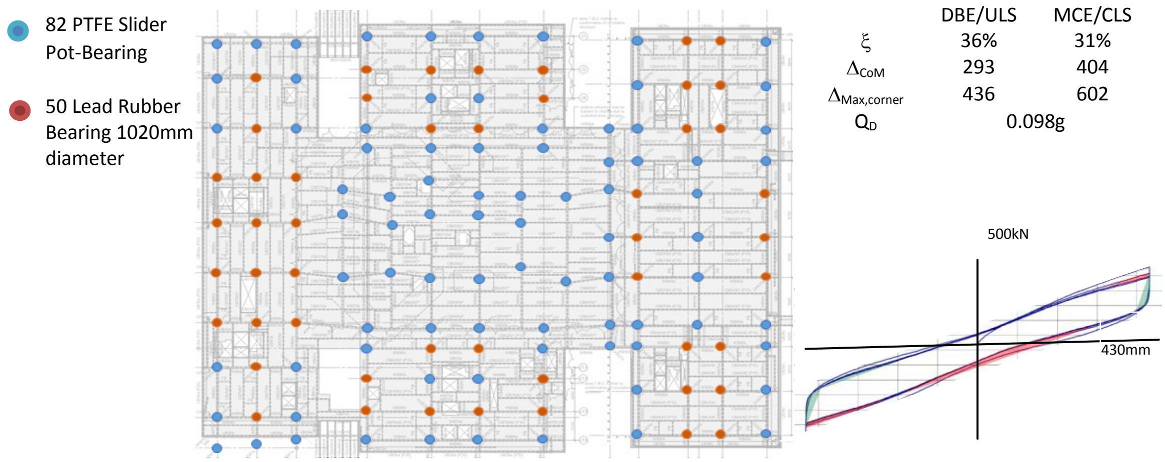

Figure 5. (a) Plan view of the base isolation transfer grillage with locations of Lead-Rubber Bearings and PTFE flat-plate slider pot-bearings (b) Summary of isolation plane equivalent viscous damping and displacement (c) Prototype LRB hysteretic shape

Maintaining capacity design principles contained within NZS 3404:1997, significant potential axial tension forces in the two-way columns were accounted for, particularly around the corner regions of each building, so PTFE sliders were used to avoid the potential damage that might occur if LRB units in these locations were subject to excessive tension forces.

The isolation plane transfer grillage also used structural steel using welded beams, bolted to fabricated cruciform joints (Figure 6). The joint region was grout-filled inside the curved stiffener plates forming each quadrant around the joint allowing a direct compression strut from column-base to isolator load-plate to form through the joint. The structural slab across the Level 1 isolation diaphragm is a concrete-steel deck composite slab, with allowance made for the variable and heavy super-imposed dead loads in the courtyard region and earthquake induced diaphragm transfer forces between the 4 buildings above.

Figure 6. Typical Lead-Rubber Bearing detailing used at the isolation plane

Isolation Plane Detailing

Structural elements crossing the isolation plane were detailed to accommodate the anticipated MCE isolator displacements (630 mm, 7500 year return period event). Referring to Figure 7 lift shafts and stairs were cantilevered off the level 1 slab. All critical building services, ground floor partitions and ceilings were detailed to accommodate the SLS2 isolator displacements (380 mm, 500 year return period event). Special attention was given to fire rated elements to ensure that their fire rating was not compromised following an SLS2 earthquake (this also applied to all fire rated walls in all levels of the building).

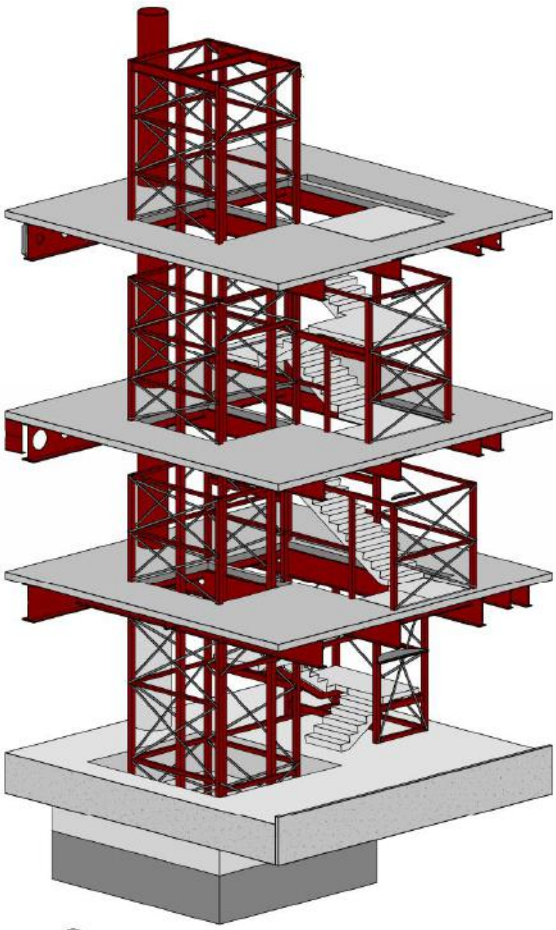

Figure 7. Typical stair and lift core structure

Substructure

The isolators are supported on circular cantilever cast insitu reinforced concrete columns which are founded on a reinforced concrete raft slab (refer Figure 3). Ground improvement in the form of ex-situ soil-cement mixing was adopted for the project to mitigate potential liquefaction issues.

Secondary Structure

Gravity Structure

The secondary gravity structure consists of ComFlor 80 composite metal deck with 175 mm thick topping slab supported on cellular secondary beams. Both the composite metal deck and cellular beams were designed as unpropped elements for the construction phase to minimise the construction program and avoid the need for back propping. The building contained significant cantilevers (i.e. > 5.0 m) and a comprehensive floor vibration assessment was undertaken in accordance with the SCI P354 Footfall Harmonic Analysis method to ensure adequate in service performance (refer Figure 8). Cellular steel beams were adopted for the project both to reduce steel tonnage and to provide room for service reticulation.

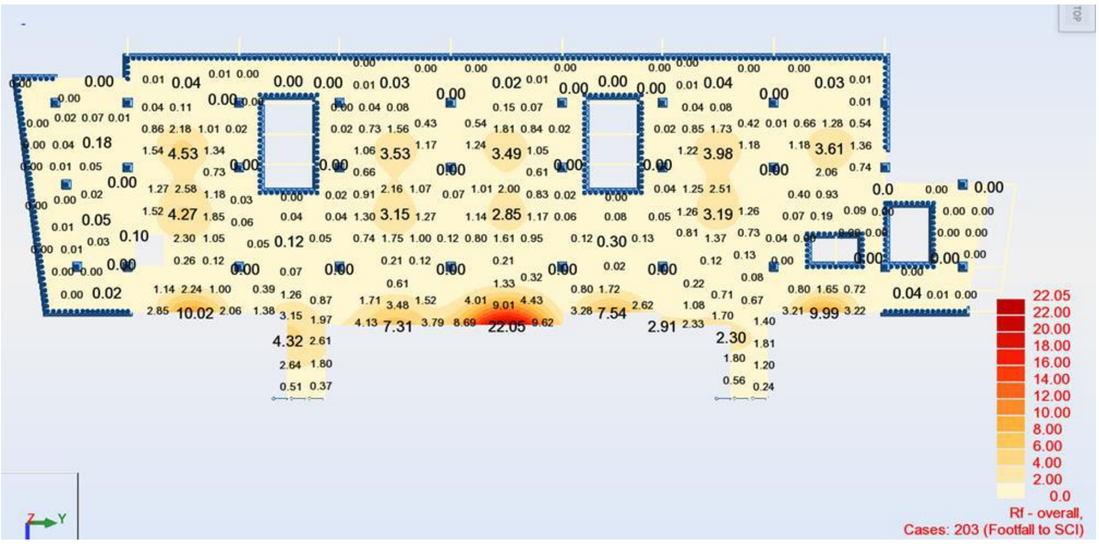

Figure 8. Output from SCI P354 footfall analysis of Durham building

Structural Steel Fire Design

Performance based fire design, undertaken by an affiliated company Holmes Fire, was utilised to demonstrate that the concrete filled steel columns, isolator units and secondary gravity system and did not typically require any passive fire protection (Holmes Fire, 2015). Figure 9 illustrates the peak temperature gradient developed through a typical concrete filled steel column. Resulting reduction in passive fire protection requirements for the steel beams in the Tuam building are shown in Figure 10.

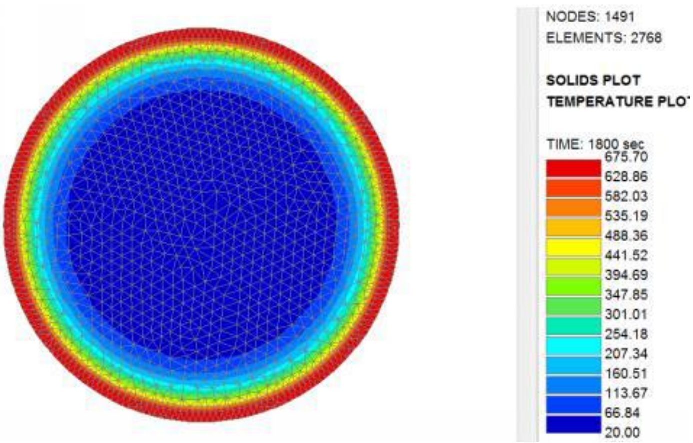

Figure 9. Peak temperature contours through typical column heated on 4 sides, at 30 minutes (Holmes Fire, 2015)

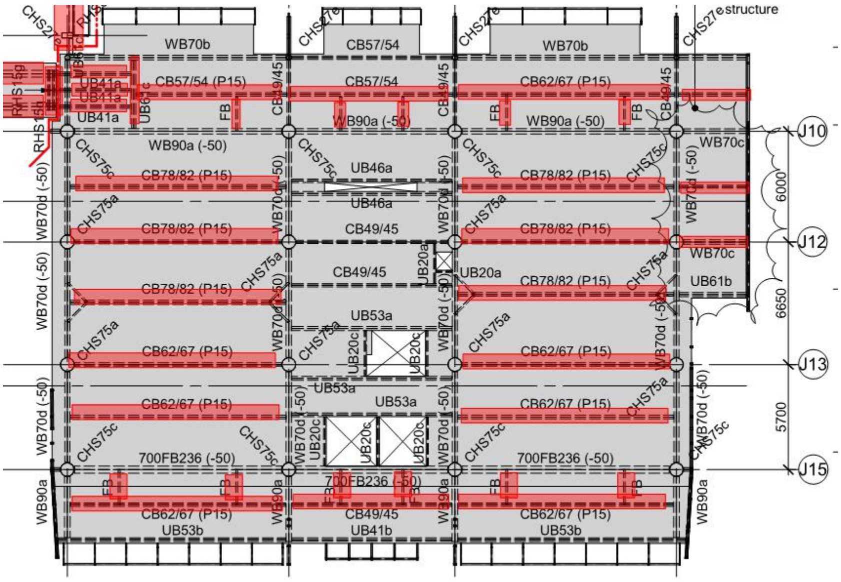

Figure 10. Passive fire protection requirements in Tuam building - red shading denotes no passive fire protection is required (Holmes Fire, 2015)

Those structural steel elements that did not require passive fire protection, and were concealed in the completed building, were typically left unpainted to minimise construction costs.

Seismic Design Methodology

New Zealand does not have a base isolation standard. A consequence of this is that there has been a range of design and verification approaches adopted by different consultants to develop base isolation systems. Without New Zealand specific guidance it is not clear how to (or at least not simple to) demonstrate New Zealand Building Code (NZBC, 2011) compliance. This has led to some confusion amongst designers and reviewers (Territorial or peers), particularly when mixing aspects of various design Codes from other countries.

Typically Chapter 17 of ASCE 7 (for example ASCE 7-10, 2010) has been adopted as means to design and detail the isolation system. However New Zealand specific design parameters such as the NZS 1170.5 (Standards New Zealand, 2004) structural performance factor, s_p, conflicting accidental eccentricity requirements; and differences in ground motion selection and scaling requirements are such that direct application of Chapter 17 of ASCE 7 is not possible. The challenge therefore remains for the designer to demonstrate Code compliance. The only complete means until now, and for the foreseeable future until NZSEE sponsored base isolation guidelines (currently being developed) are introduced and recognised by local authorities, has been to use non-linear time history analyses as a means of meeting the NZ Building Code as an alternative solution.

For this project ASCE 7-10 provided the guidance to design the isolation system using a single-degree-of-freedom approximation, however the overall design intent was to satisfy the fundamental requirements of NZS 1170.5 (Standards New Zealand, 2004). Therefore the design spectrum for the isolation system followed NZS 1170.5 and the accidental eccentricity of the isolation plane was set as 10% of the plane dimensions. Non-linear time history analyses was then used to validate the isolation system design.

The isolation system nominal characteristic yield coefficient (Q_d) is 0.1g, and governing design base-shear coefficient from ASCE 7-10 for the isolated super-structure was 1.5Q_d.

Seismic Loading

A probabilistic seismic hazard analysis (PSHA) was commissioned for the project. This work was undertaken by URS (URS, 2013) and was required for the following reasons:

- The isolated period of the structure was greater than 1.5 seconds and therefore the NZS 1170.5 code spectra did not apply (NZ Building Code, 2011)
- NZS 1170.5 did not provide guidance on earthquakes with return periods in excess of 2500 years.
- To quantify ground motion amplification effects anticipated at the site, and the proposed base isolation system. This included the development of nonlinear spectra.
- To provide recommended ground motions records for use with the non-linear time history analysis.

Figure 11 illustrates the primary site specific uniform hazard spectra generated for the 2500 year return period ULS earthquake and compares this with the NZS1170.5 code spectra. While the basic isolation design was carried out using the NZS1170.5 defined IL4 (R = 1.8) spectrum, a Maximum Considered Earthquake (MCE) event was needed to finalise the maximum design displacement in accordance with the ASCE 7-10 design methodology. The MCE event was assumed to be equivalent to the CLS event commonly referred in the AS/NZS context. With the MCE return periods set at 7500 years, the CLS:ULS ratio was equal to approximately 1.25 (R = 2.25) around the isolation period of three seconds.

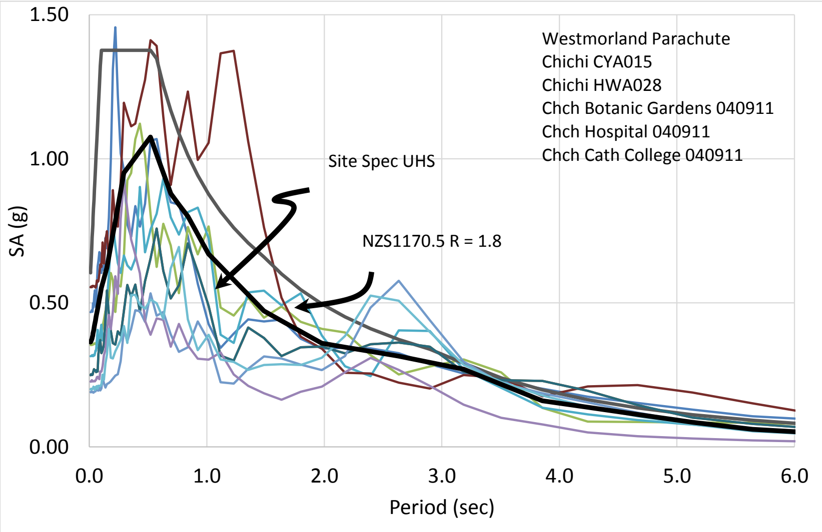

Figure 11. Primary earthquake record components (ULS) with scaling to match the site specific UHS. Comparison provided to the NZS1170.5 spectrum used for the isolation system design: Site Class D, R = 1.8 spectrum, Z = 0.3, Sp = 0.7, μ = 1.0.

The site specific Uniform Hazard Spectra developed by URS were then used to scale the corresponding ground motion record suite (seven records per return period) provided by URS as part of the PSHA. The AS/NZS 1170.5 scaling procedure was used and the resulting record spectra are compared to the 2500 year return period UHS and Code design spectrum in Figure 11. The scaling was set using S_p = 1.0, thus providing a final measure of conservatism in the verification analyses while using the UHS as the scaling target.

It is acknowledged that when designing the LRBs a reduced S_p = 0.7 could have been used recognising the resilience of the LRB units. In this instance an S_p = 1.0 analysis would still be required to determine PTFE slider pot bearing sizes and rattle space widths.

Analysis and Verification Methodology

3D Linear Elastic Design Model

The modelling followed distinct design analysis and then design verification phases. Design analysis used a full 3-dimensional linear-elastic model of the isolation plane and superstructure buildings developed in ETABS. To estimate the isolator design axial loads the isolation plane was first fixed, and a modified ULS acceleration spectrum with a step-down at 0.6 seconds was used, to allow for the isolation plane equivalent viscous damping on the primary translational isolation modes. This fixed-base model allowed an evaluation of the individual building periods (to ensure reasonable separation between the fixed-base and isolation periods) and ULS axial loads.

Isolation system design was carried out using a SDOF approximation as per ASCE 7-10. Effective isolator and pot-bearing slider properties were then input into the 3-dimensional ETABs model that included the isolation plane movement with LRB and PTFE sliders. Using this isolated model the full building isolation period and an initial review of the drift performance of the superstructure was made. Each individual building was separated out from the model and using a fixed-base condition immediately above the isolation plane, the superstructure developed design was completed. The design base-shear was scaled to match $1.5Q_d$ = 0.15g. Capacity design was undertaken to ensure resilience should the building be subject to seismic loading in excess of ULS.

3D Nonlinear Time History Analysis Validation

Final design verification of the isolation system and superstructure was provided by extensive non-linear time history analyses. This verification approach provides the means to comply with the New Zealand Building Code (NZBC, 2011) as an Alternative Solution, and was used to assess not only the SLS2 and ULS performance of the system, but also the CLS demands for both the isolation system and the superstructure.

A 3D non-linear time history model was developed in ANSR (Mondkar and Powell, 1979) using backbone definitions from ASCE 41-13 (ASCE 41-13, 2013) for the superstructure. The model was essentially the same as the linear-elastic model, however full section definitions including yield moments and surfaces were defined for the beams and columns. The LRB elements allowed a simple bi-linear hysteresis using definitions for initial and post-yield stiffness, and yield shear force. The pot-bearing slider element model allowed for the coefficient of friction dependence on velocity and bearing pressure.

Non-linear time history verification was carried out for the 500 year (SLS2), 2500 year (ULS) and 7500 year (CLS) return periods. Bounding analyses at ±20% on LRB and friction properties were carried out, with upper bound ULS results being used to evaluate the superstructure performance (including floor accelerations), and lower bound CLS results confirming the isolation plane displacement demands. The SLS2 performance used nominal isolator properties.

Results from the nominal ULS non-linear time history analyses are presented here only. Even though seven earthquake records were used, the envelope of results was used to be consistent with NZS1170.5 requirements. It is recognised that this leads to a slightly conservative evaluation of performance, however it was considered that complete verification compliant with NZS1170.5 was necessary as the non-linear time history results were being used as the final design verification check.

Figure 12, Figure 13 and Figure 14 present the enveloped responses for the diaphragm centre-of-mass storey drift, displacement and floor acceleration. While there was some torsional amplification of these responses, the isolation system clearly achieves the desired effect of limiting the superstructure demands and providing low-damage structural performance. It is noted that the east-west (X-direction) drifts were typically higher for the three Justice Precinct building due to the moment-frames tending to have long spans or limited number of bays along a frame-line. The ESB results are more consistent, and this was reflective of a focus to achieve lower drifts in this building as it is the actual NZS 1170.5 Importance Level 4 designated facility in this development (the Justice Precinct buildings strictly speaking need only satisfy Importance Level 3 requirements).

The element performance assessment confirmed that the rotations did not exceed ASCE 41-13 Immediate Occupation (IO) limits even at the ULS demand level, while plastic rotation demands from the 7500 year return period runs had plastic rotations exceeding IO in a few limited locations, but were well below ASCE 41-13 Life Safety limits.

Overall the performance of the buildings was demonstrated to meet New Zealand design Standards, and satisfy performance requirements for the isolation plane even up to a hazard factor R = 2.25

Conclusions

The Christchurch Justice and Emergency Services Precinct project required challenging design and analysis solutions to ensure the full benefits of the project could be realised. Base isolation was adopted as the preferred low damage design solution for the project because it has a proven track record and provides a very high level of protection to superstructure elements and building contents.

Composite structural steel construction was used throughout the project to maximize structural efficiency. Performance based structural steel fire design was used to accurately determine scope of required passive fire protection requirements

This project utilised aspects of international building codes, but always with the intent of verification by non-linear response history analyses that provided a means to meeting the New Zealand Building Code by alterative means. In doing so the development has been demonstrated to achieve the requirements for an IL4 facility, and can maintain satisfactory performance under much larger events, up to the 7500 year return period earthquake.

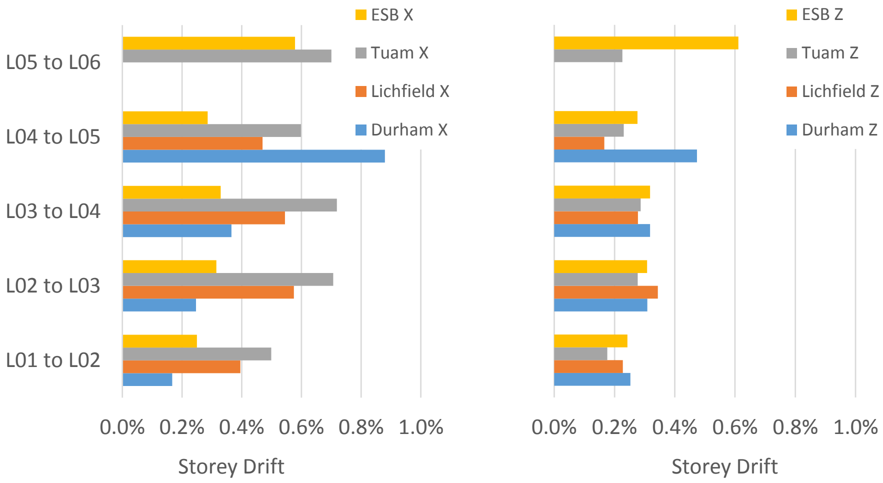

Figure 12. ULS (2500 YRP) storey drift envelopes with nominal isolation properties (a) X-direction (b) Z-direction.

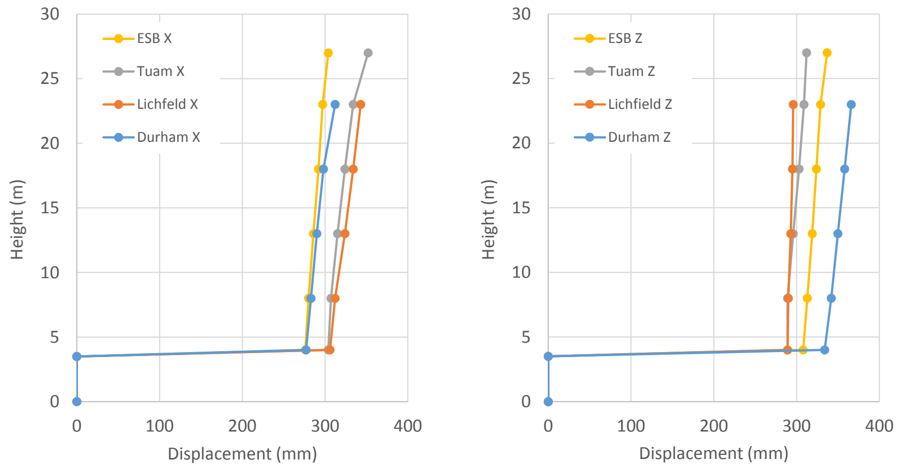

Figure 13. ULS (2500 YRP) peak floor displacements envelopes with nominal isolation properties (a) X-direction (b) Z-direction

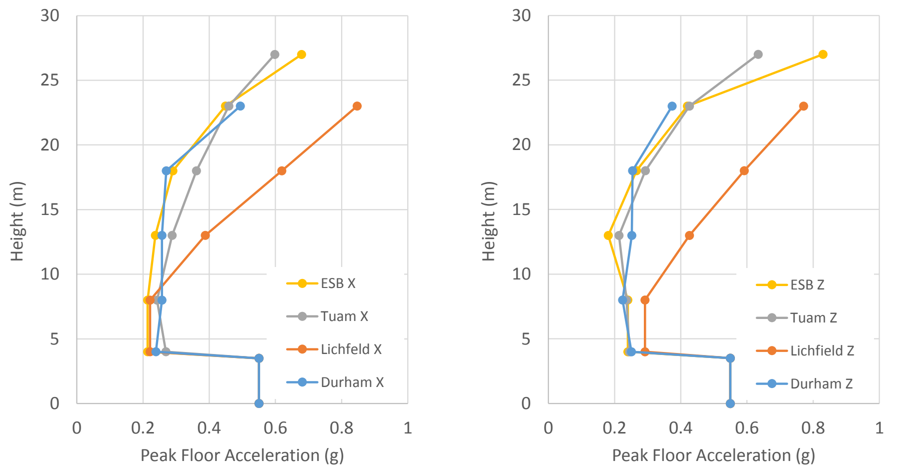

Figure 14. ULS (2500 YRP) peak floor acceleration envelopes with nominal isolation properties (a) X-direction (b) Z-direction

Acknowledgments

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