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Aspects of Design for the Base Isolated Christchurch Justice and Emergency Services Precinct

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ABSTRACT: The Christchurch Justice and Emergency Services Precinct is the first of the central government Anchor Projects to be base-isolated as part of the Christchurch central city re-build. This city-block development houses both the Ministry of Justice law courts and an amalgamated Importance Level 4 command centre facility for key emergency services.

The project is unique in New Zealand in that four seismically separated two-way steel moment-frame buildings benefit from the isolation effects, where the isolation plane is provided at the underside of a common first floor podium level. The base-isolation system design had performance targets at both the 2500 year (ULS) and 7500 year (CLS) return periods, with the final scheme comprising of 50 large diameter lead-rubber bearings and 82 flat-plate pot-bearings.

The isolation system was designed to the NZS 1170.5 ULS spectrum, however a site specific study also provided uniform hazard spectra for a range of return periods. This paper summarises the design approach that met relevant local standards, incorporated the benefits of non-linear time history verification, and provides an outline to how similar complex isolation projects in New Zealand can be addressed in the future.

1 INTRODUCTION

The Christchurch Justice and Emergency Services Precinct was 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.

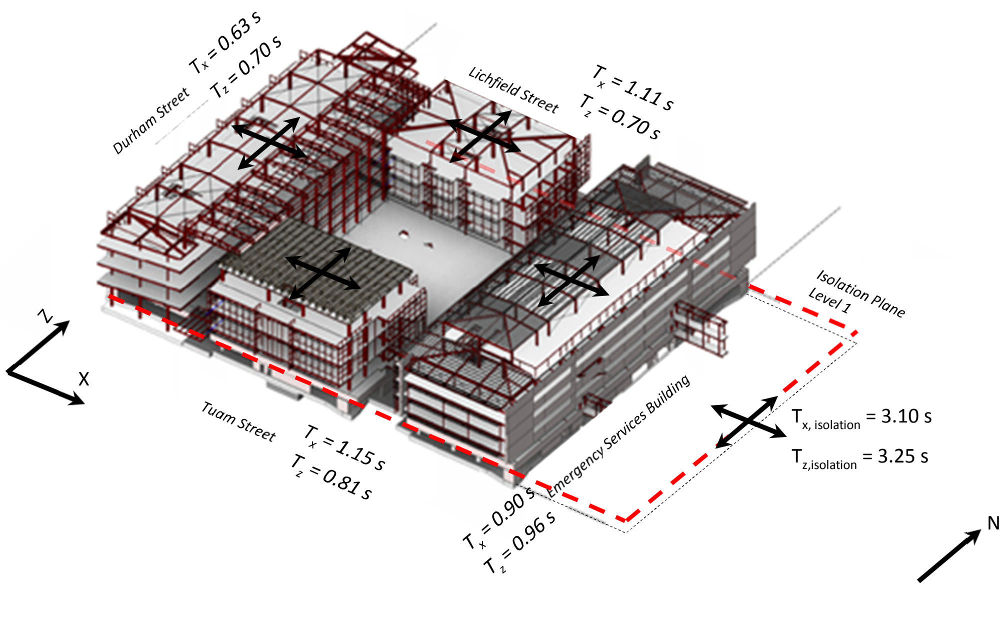


Figure 1. Overview of the Christchurch Justice and Emergency Services Precinct. Individual fixed-base building periods and isolated system periods are shown.

The primary building development comprises of both the Justice Precinct (Ministry of Justice, custodial services) and Emergency Services Building (ESB - Civil Defence, St John Ambulance, NZ Fire Services, South Communications). The four linked buildings which house the above facilities surround a central courtyard, and are isolated on a common first floor podium. The Ground Floor is therefore not isolated, and all service connections and isolation plane movement allowances are defined at the underside of the First Floor. 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 defined as an Importance Level 4 facility, and utilises dual Moment-Frame and Buckling Restrained Brace lateral force-resisting systems.

2 BUILDING AND STRUCTURAL DESCRIPTION

2.1 Structural Steel Two-Way Moment-Frames

The primary lateral force-resisting system above the isolation plane is a two-way structural steel moment-frame. The primary moment-frames were Capacity Designed as a Category 2 structure per NZS 3404 (Standards New Zealand, 1997), however under ULS demands they are intended to remain essentially elastic while CLS isolation displacements could induce demands in the superstructure that develop $\mu = 2$ response. Category 2 welded steel beams with reduced beam sections (RBS or 'dog-bone sections') designed according to SCNZ EQK1003 (Cowie, 2010) provided at each end were used for the moment-frames. The 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 around the perimeter of each building.

Concrete filled hollow steel sections (also referred to as concrete filled tubes, CFT) were used to improve the efficiency of the two-way moment-frames, as well as provide passive fire resistance. While these 750 mm diameter columns were designed as Category 3 elements following NZS 3404:1997, 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). To satisfy the steel material requirements for the Category 2 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 should be noted that standard hollow-sections available in New Zealand do not strictly satisfy the material requirements for Category 1, 2 or 3 structures, although Grade J2 is known to meet Type 6 limits when tested.

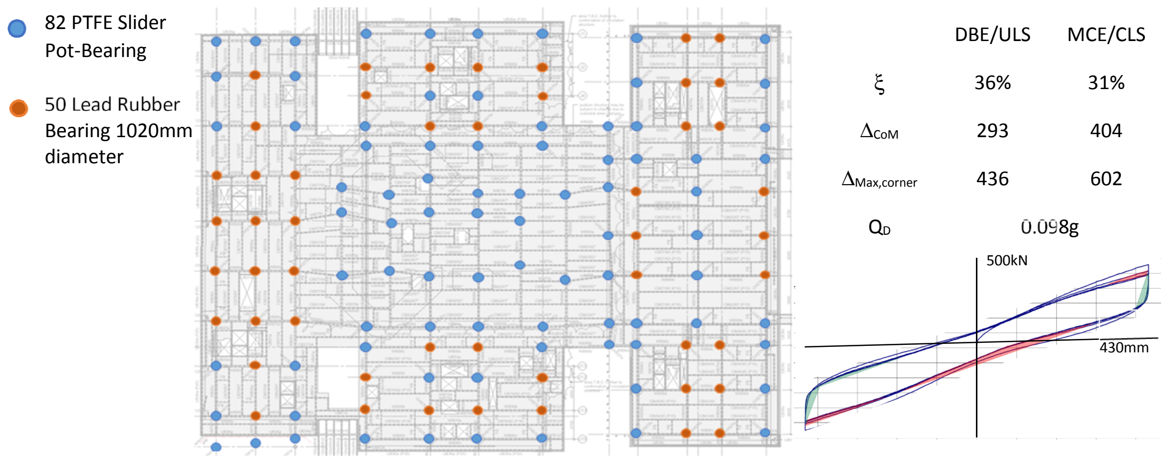
Figure 2. Examples of (a) Reduced Beam Section and collar detail for the moment-resisting steel frame (b) typical cruciform joint located over the Lead-Rubber Bearings.

2.2 Isolation Plane and Transfer Diaphragm

The isolation plane transfer grillage also used structural steel using welded beams, bolted to fabricated cruciform joints (Figure 2). The joint region was also 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.

The isolation units themselves sit on top of circular reinforced concrete columns which are cast in-situ on top of a reinforced concrete raft slab.

The base isolation is provided by 1020mm diameter Lead-Rubber Bearings (LRB) distributed under the main seismic column lines in the footprint of each tower (Figure 3). Flat-plate pot-bearing PTFE sliders were used under the primary frame columns where significant uplift forces were expected and throughout the courtyard extent. 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. Maintaining the Capacity Design applied from 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 pot-bearing sliders were used introduced to avoid the potential damage that might occur if LRB units in these locations were subject to excessive tension forces.



	DBE/ULS	MCE/CLS
ξ	36%	31%
Δ_{CoM}	293	404
$\Delta_{Max,corner}$	436	602
Q_D	0.098g	

Figure 3. (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

3 DESIGN APPROACH

Without a New Zealand specific base isolation design Standard, there have been a range of design and verification approaches employed to develop base isolation systems. Until guidance applicable to New Zealand is available, it is not clear how to (or at least not simple to) demonstrate New Zealand Building Code (NZ BC, 2011) compliance. Typically, designers have resorted to using recognised international references such as Chapter 17 in ASCE 7 (for example ASCE 7-10, 2010). However this has presented difficulties for designers and reviewers (Territorial or peers) around mixing different aspects of various design Codes from other countries. Further to this, with base isolation not being recognised as a standard solution in New Zealand Standards, the problem has remained, and still does, for the designer to demonstrate Code compliance. The only complete means until now, and for the foreseeable future until the 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 to 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-degreeof-freedom approximation, however the overall design intent was to satisfy the fundamental requirements of NZS 1170.5 (NZS 1170.5:2004). 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 plan dimensions.

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 $1.5Q_d$.

At the outset of the project there was significant uncertainty around the long period spectral shape for Christchurch. A site specific hazard analysis was performed by URS (URS, 2013), and the resulting spectra used to define the ratio between the 7500 year return period (the agreed Collapse Limit State, CLS, earthquake event for the project) and 2500 year Standard defined Ultimate Limit State (ULS) earthquake events. While the basic isolation design was carried out using the NZS1170.5 defined IL4 (R = 1.8) spectrum (see Figure 4), a Maximum Considered Earthquake (MCE) event was needed to finalise the maximum design displacement in keeping with ASCE 7-10. The MCE event was assumed to be equivalent to the CLS event commonly referred to within the AS/NZS context. Following recommendation and agreement with the client the MCE return period was set at 7500 years, with the CLS:ULS ratio equal to approximately 1.25 (R = 2.25) around the isolation period of three seconds.

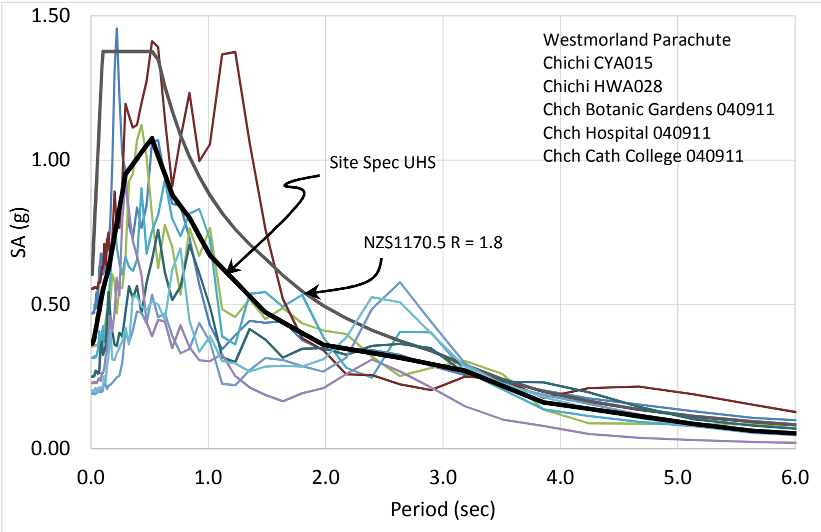


Figure 4. Primary earthquake record components (ULS) with scaling to match the site specific UHS. Comparison provided to the NZS1170.5 spectrum used for the isolation system design: Site Class D, R = 1.8 spectrum, Z = 0.3, Sp = 0.7, μ = 1.0.

The site specific Uniform Hazard Spectra developed by URS were then used to scale the corresponding record suite (seven records per return period) provided as part of the hazard analysis. The AS/NZS 1170.5 scaling procedure was used, with the resulting record spectra compared to the 2500 year return period UHS and Standard design spectrum in Figure 4. 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.

4 ANALYSIS AND VERIFICATION APPROACH

4.1 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 towers 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 tower periods (to ensure reasonable separation between the fixed-base and isolation periods) and ULS axial loads. A subsequent preliminary Capacity Design of the frame elements also provided axial loads which, with reference to the earlier definition of CLS superstructure ductility demands, corresponded to CLS loading conditions to verify the isolator unit stability in a 7500 year return period earthquake.

Using these axial loads the isolation plane design was carried out using a SDOF approximation per ASCE 7-10. The 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 tower 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. A full Capacity Design was used to meet AS/NZS 1170.5 ULS requirements following NZS 3404:1997 (although ductile response is only anticipated for ground motions exceeding the 2500 year event), and also used to review the isolator design axial loads. With some iteration, the final isolation plane design was simplified to specify the same LRB dimensions and properties under each building. This was due to the axial demands being somewhat similar, and recognition that the apparent design eccentricity across the isolation plane was relatively small. This enabled the design to conclude faster, and significantly reduced the prototype development and verification test costs, as well as final delivery times.

4.2 Non-linear Response History Model Definition

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

The non-linear response 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. Beam stiffness values were reduced by 5% to account for the RBS as each end, while the CFT columns were given increased effective stiffness values based on the equations provided in EC4. The column stiffness increase was typically 130% of the bare steel section. Through specific detailing of the composite slab around the beam ends the stiffening effects of composite action are expected to be limited, and for conservative estimates of superstructure drift were not included in the time history model.

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.

The non-linear response 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. A comparison of the resulting isolator definitions is given in Table 1.

Table 1. Design and bounding values used for the isolation system verification

	Lower Bound (Nominal x 0.8)	Nominal Design Values	Upper Bound (Nominal x 1.2)
LRB Elastic Stiffness, K_u (kN/m)	7140	8930	10720
LRB Yielded Stiffness, K_r (kN/m)	890	1120	1340
LRB Yield Force, F_y (kN)	105	130	157
Isolation Characteristic Strength, Q_d/W	7.9%	9.8%	11.8%
Pot Bearing Slider Coeff Friction (High-speed)	8%	10%	12%

5 PERFORMANCE VERIFICATION

Results from the nominal ULS non-linear response 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 response history results were being used as the final design verification check.

5.1 Global Performance

When using response history results the primary aspects for review of the isolation system are the maximum displacements at the extreme corners where torsional response has its greatest influence, and the maximum axial loads sustained by the isolator units. With the four independent towers on the common isolation plane an in-depth investigation into the maximum axial forces transferred through the courtyard portion of the isolation plane was carried out using a modified model that separated (using gap, hook and shear link elements) the courtyard diaphragm from the individual building diaphragms at Level 1. This aimed to confirm that the rigid diaphragm assumption, which would typically be used in the model, was appropriate and that the axial forces could be sustained along with bending moments and shears in the beam grillage. Superstructure floor displacements, accelerations and storey drifts were also used to confirm that the performance targets had been met.

Figure 5, Figure 6 and Figure 7 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 buildings 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 IL4 designated facility in this development (the Justice Precinct buildings need only satisfy IL3 to AS/NZS 1170.0).

The element performance assessment confirmed that the rotations did not exceed Immediate Occupation 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 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

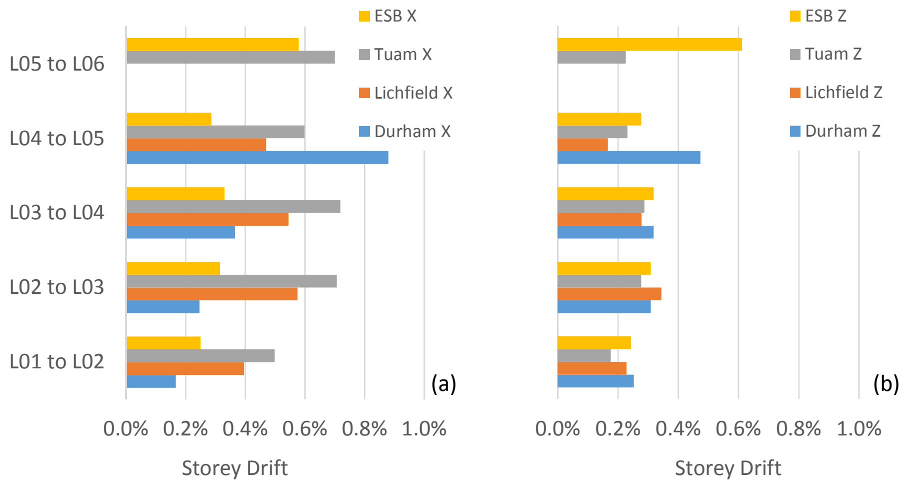


Figure 5. ULS (DBE) storey drift envelopes with nominal isolation properties (a) X-direction (b) Z-direction

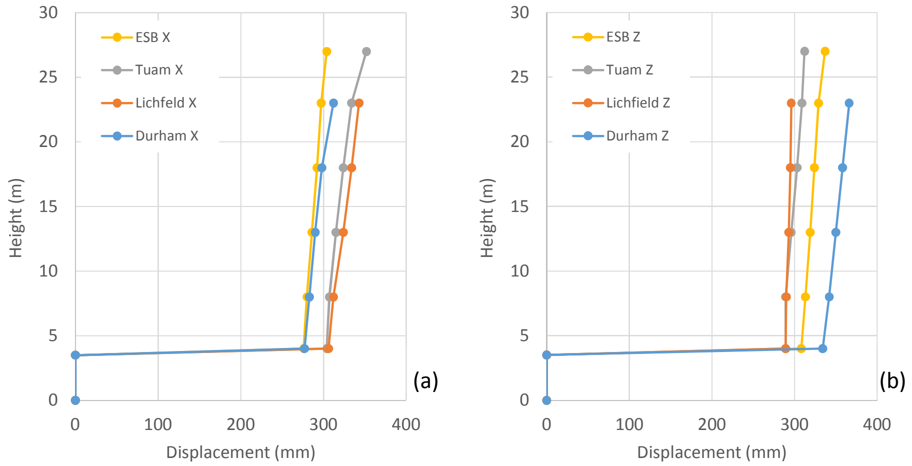


Figure 6. ULS (DBE) peak floor displacements envelopes with nominal isolation properties (a) X-direction (b) Z-direction

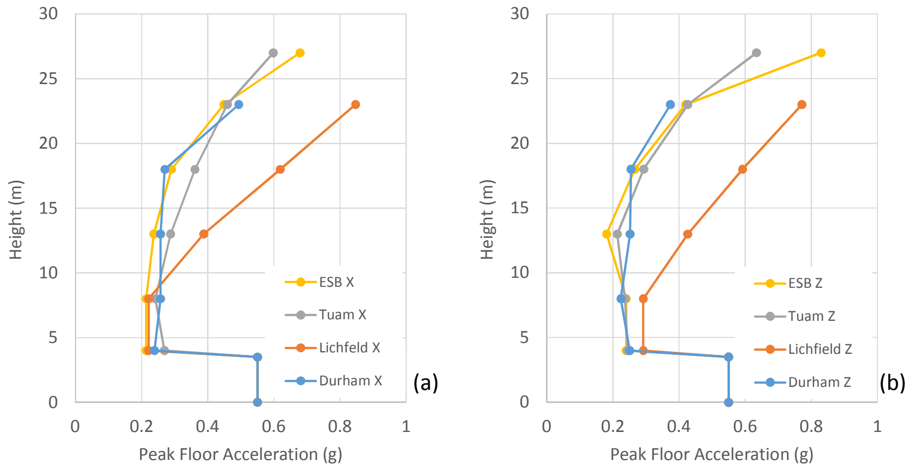


Figure 7. ULS (DBE) peak floor acceleration envelopes with nominal isolation properties (a) X-direction (b) Z-direction

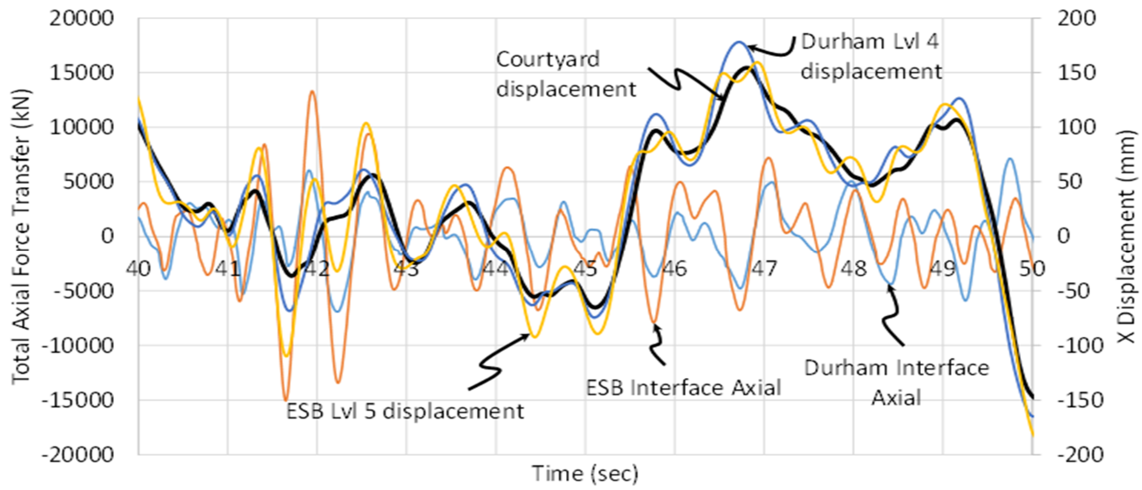


Figure 8. Response history segment of the X-direction Courtyard transfer axial forces compared with Durham building and Emergency Services Building roof displacements

Figure 8 provides an example of the type of interaction between the towers on opposite sides of the central courtyard. The Durham and ESB Interface traces refer to the imaginary separation between each tower diaphragm and the courtyard diaphragm. The total axial load crossing such interfaces between each tower and courtyard was tracked throughout each analysis, with the envelope results at each beam line being used in the design of the beam grillage and connections. The behaviour is as expected, that the transfer forces reflect the individual tower fixed-base first-mode response which is superimposed over the isolation plane response. The indication from this investigation is that a conservative but reasonable estimate of the transfer forces can be made using the fixed-base base-shear of the individual towers summed as if acting in opposite directions.

6 CONCLUSIONS

The Christchurch Justice and Emergency Services Precinct development has presented a number of design challenges, some of which have been noted in previous base isolation projects in New Zealand. Many of these are due to the lack of formal guidance for base isolation design in the New Zealand context, a matter that will hopefully be rectified with the current development of a New Zealand specific guideline.

This project utilised aspects of internationl 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 alternate 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.

Particular challenges with the design and analysis focussed on the definition of a suitable CLS (or MCE) earthquake event via site specific hazard analysis that could be used for the isolation plane performance verification. In completing these analyses the interaction of the four separate towers on top of the isolation podium was also found to be a challenge, and was demonstrated to be a significant issue for design of similar projects in the future.

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