Aspects of Design of the Base Isolated Christchurch Hospital Acute Services Building

A. Thompson¹, C. Mackenzie²

ABSTRACT: Christchurch Hospital’s new Acute Services Building (ASB) is one of the government anchor projects in the Christchurch central city rebuild following the Canterbury Earthquakes. The 60,000 m² NZS1170 Importance Level 4 facility will house multiple clinical functions and ward accommodation. This paper summarises the key project constraints and concepts considered, which led to the selection of a 10 storey capacity designed steel moment frame – base isolated over a thick reinforced concrete raft foundation. The particular focus of the paper is a body of work undertaken to inform the selection of the most appropriate isolation system type, and its refinement to achieve the optimal performance possible within the project constraints.

The study drew from published research and vendor enquiries, supplemented with parametric studies using the ASB non-linear response history analysis model. This enabled generalised recommendations from the literature to be specialised to this real structure; with its specific contributing seismic hazard, subgrade conditions and other design constraints. Of particular interest is the extent to which superstructure performance (as measured by deformations and spectral floor accelerations) is influenced by the characteristics of isolator activation and non-linearity in response, for similar overall damping. The more commonly cited relationship linking higher drifts and accelerations with higher damping at the isolation plane holds, however is shown to have high sensitivity to the hysteretic model.

Specifications were developed for an isolation system comprising lead rubber bearings and flat sliders, and an alternative system using adaptive curved surface sliders. High quality tender submissions were received from the market for both systems.

1 INTRODUCTION

Christchurch Hospital’s new Acute Services Building (ASB) is one of the key projects in a significant capital investment in healthcare infrastructure across multiple campuses in the Canterbury Region. It is programmed for completion in 2018. Clinical and acute services (including an emergency department, operating theatres, intensive care and high dependency units) are located in a three level podium structure. Above this there are 6 levels of ward accommodation split across two linked towers with a rooftop helipad. There is provision for a future third tower.

This paper presents some background on the building design and local seismic hazard for context, and then focuses on the development and refinement of the isolation system specification. The outcomes are presented in the form of a case study for this specific building type and hazard environment. Nevertheless the outcomes may be informative for other projects, and raise some points for consideration.

2 BUILDING AND STRUCTURAL DESCRIPTION

2.1 Structural Steel One-Way Moment Frames over Raft Foundation

The primary lateral force-resisting system above the isolation plane uses one-way structural steel moment-frames with welded I sections. The frames are capacity designed category 2 moment frames to NZS 3404:1997 (Standards New Zealand, 2007). For the main frames, 1200 deep column sections vary between 326 and 661kg/m, and typical beam sections range from 175 to 317kg/m.

Gravity frames use welded columns and simple un-propped composite construction. Cellular secondary beams are used and typically span 13.5m or 11.5m. Beams have been made quite deep (rather than being optimised for minimum depth), in order to reduce response to footfall induced vibration and to ensure that large cells can be provided for reticulation of services (rather than cells being too small to be useable by large services or ductwork). Slabs are composite steel decked.

¹ A. Thompson, Holmes Consulting LP. Email: andrewt@holmesgroup.com
² C. Mackenzie, Holmes Consulting LP. Email: chrism@holmesgroup.com
The structure is isolated over a 1.4m thick reinforced concrete raft, using a mix of lead rubber bearings and flat sliders. Isolators are located directly below the first suspended level, supported on circular concrete columns. The structural raft sits on a natural gravel raft 5-10m thick, overlying sands and silts, with Riccarton Gravels at a depth of 20m. The natural raft is important for ground performance, reducing the risk of liquefaction related ground damage, and makes the site particularly well suited to shallow foundations.

Given the seismic hazard, a 3 to 3.5 s effective SDOF isolated period (ULS) with moderate damping was expected to give good performance. Therefore before any strength calculations were made, a superstructure period of 1 to 1.2 s was targeted. The superstructure design ductility was set at 1.25 for ULS demands, however following the outcome of the design process they are expected to respond essentially elastically at the ULS. CLS isolation system displacements would induce some ductility in the superstructure.

2.2 Constraining the Grid

The building is relatively tall for an isolated structure, so suited distributed lateral resistance rather than localised bracing elements which would require expensive and spatially challenging interventions in the lower floors to spread the overturning. Braces or walls are also problematic for a long-term loose-fit asset if they cannot be positioned solely to the outsides, which would not have been readily achievable for a building of these dimensions.

The foundation system also suited distributed resistance to reduce localised overturning actions. Moment frames are less susceptible than multi-bay braced frames to potential differential settlement of a shallow foundation system – which includes the effect this could have on isolator hardware and system performance. A lightweight superstructure was favoured to limit the overturning actions and to make a low fixed base period easier to achieve for more effective isolation. The building was therefore still light enough that reducing the number of bearings would improve the ability to optimise the isolation system design, particularly for designs using elastomeric bearings. Therefore the grid spacing should not be too tight.
Intermediate link columns were added to longitudinal frames to stiffen them. Their close spacing introduced some complexities, however their layouts were able to be incorporated and the overall steel tonnage was similar or less than what it would have been with a closer external grid spacing. It was better to have the external grid match the internal grid, and the isolation system worked better with fewer bearings.

Figure 3: Mixed mechanism for longitudinal frames (left); overstrength moments from primary hinges (right)

### 2.3 Superstructure Detailing

Category 2 welded steel beams used reduced beam sections, designed according to SCNZ EQK1003 (Cowie, 2010). Columns were 1200mm deep and so from a transport and erection perspective, bolted connections at the column face were preferred. Gusseted moment end plates were used generally, however for the beam sections heavier than 260kg/m these were impractical and a rigid flange bolted joint was used instead.

Figure 4: Primary frame connections – moment end plates (left) with RBS were used generally

### 2.4 Isolation System

Performance specifications were developed for both a curved surface slider system and a lead rubber bearing/flat slider system (LRB) as complying alternatives. The LRB system was the successful tender. The final design consists of 79 lead rubber bearings and 49 flat sliders using PTFE on stainless steel. The sliders carry 35% of the structure weight. The characteristic strength of the system (Qd) is 0.07 as a fraction of the building weight under nominal properties. The total maximum displacement of corner isolators at MCE/CLS is 600mm.

The building has provision for a future third tower. The isolation system is tuned such that it has similar yield levels and post yield stiffness relative to the isolated weight in each case, and minimal inherent eccentricity under each case. This is done by positioning 6 sliders and LRBs in series on the lesser loaded far eastern grid. For each of the 6 additional levels constructed, there is provision to provide a bracket to plate off the slider and activate the LRB. The stiffer LRBs, along with the increased axial load on gravity column sliders under the future tower (partially offset by reduced friction coefficients from higher contact pressures) provide the necessary adjustment.

There are therefore a total of 122 column lines, and the average seismic weight per bearing is 4000kN, which is heavy enough to allow good optimisation of system performance for elastomeric bearings. The LRBs are positioned under the seismic frames, and the sliders under gravity columns. The grouping of sliders away from LRBs on a reasonably wide grid reduces the potential for interactive effects from the rise and fall of LRBs or from differential foundation movement. Two sizes of LRB were used; 1020mm diameter bearings generally, with 1120mm diameter bearings under each of the 12 tower perimeter frame corner columns for stability under high compressive loads.

Recently compiled data from tension testing permitted the use of LRBs under corner columns where net tensions develop in linear analysis (Pietra 2017). The data showed LRBs experience significant reduction in axial stiffness under tension and an ability to accommodate quite reasonable axial strains before rubber properties are significantly affected, even when units are un-displaced. Small strains redistribute frame actions and mobilise more gravity weight in this structure, enabling
the plastic frame sway mechanism to develop (verified by non-linear RHA). For this project, the use of LRBs under corner columns was therefore preferred over sliders, which for a number of reasons would have proved difficult under such large and variable compressions/tensions. Prototype and production testing of isolators included tension testing. Due to the superstructure frame configuration, the structure would be stable in the event of loss of support under a corner column. This was considered to demonstrate good redundancy.

3 DESIGN PROCEDURES

The design of the ASB used AS/NZS 1170.0:2002 and NZS 1170.5:2004 for general loading and modelling parameters. Site seismic hazard was determined by a special study (refer section 3.1). ASCE 7-10 Section 17 (ASCE 2010) was adopted as the primary guidance/framework for determining design actions on the isolated superstructure, sub-structure and the isolation system. The response-history procedure was used. Isolation systems and elements providing stability to the isolation system have quite specific and variable fragility and therefore require direct assessment at the MCE under ASCE 7-10 and other international standards. Under the New Zealand framework this was interpreted as the Collapse Avoidance Limit State (CLS or CALS) and the return period defined as 7500 years. New Zealand standards NZS 3101:2006 and NZS 3404:1997 were the primary material standards used for design.

The use of a mixture of local and international standards and guidance presents challenges in achieving consistency in risk, reliability and performance, and also in having designs accepted under the New Zealand regulatory framework. Some discussion can be found in (Pettinga, 2015). Whilst these issues had to be addressed for this project, this is not the focus of this paper.

3.1 Seismic Hazard

In accordance with amendments to Building Code Clause B1 in effect at the time of design (NZ BC 2011), seismic hazard was determined by a project specific Probabilistic Seismic Hazard Analysis (PSHA). The analysis was performed by URS (URS, 2014) and peer reviewed by Bradley Seismic. The study provided Uniform Hazard Spectra (UHS), de-aggregation of hazard and recommended earthquake record suites for response history analysis at each return period considered. Suites of seven records were used, as for this project three records were not considered to be enough to represent the various contributing scenarios nor provide sufficient variation. To comply with NZS 1170, the maximum response from the 7 records was used. The use of an average result (similar to US practice) would likely have been un-conservative, and despite a more detailed method being proposed by the PSHA peer reviewer, the maximum of 7 was kept.

The approximate ratio between ULS and CLS from the outcome of the PSHA was 1.25, which under the NZS 1170.5 context gives an R factor of 2.25. Whilst this value was used with NZS 1170.5:2004 spectra (with Z = 0.3) for preliminary exercises, ultimately (for compliance) the UHS were used to obtain minimum displacements and lateral load distributions (as per ASCE 7-10), and to set the target spectra for scaling records. The spectra obtained are shown in Figure 5, and were somewhat lower than the NZS 1170.5 Z = 0.3 spectra around the isolated period range of interest (i.e. 3 to 3.5 s).

![Figure 5: Site Specific Spectra and NZ Code Spectra, Sp = 1, (left); hazard deaggregation at the 7500 year return period controlling maximum isolation system displacement for 3.0s response spectral acceleration (URS 2014)](image_url)

From deaggregation, nearer source events contribute strongly to the hazard at very high return periods, as epsilon values from alpine fault ruptures become very large (Figure 5). Forward directivity contributes, and was reflected in the recommended record suites. This likely resulted in higher displacements in RHA compared with the displacement based SDOF analysis, due to the reduced influence of damping under pulse-like conditions (Priestley et al, 2007). Damping factors from ASCE 7-10 do not adjust for source distance.

Isolation system displacements are governed by the RHA. In light of the above comments on record scaling and damping, this outcome was expected. For the resulting isolation system design, superstructure minimum lateral design forces are...
therefore set by the limit of \(1.5 \times \text{the isolation system yield force}\), since the SDOF analysis again gives base shears which are below that \(1.5 F_y\) limit (even considering property modification factors). The corresponding ductility for the ULS sets between elastic response and \(\mu = 1.25\), which is reasonable.

Figure 6: Envelope of Scaled Suite of 7 Records (and example ID076_CHHC_Hospital_20100903, included in both 2500yr and 7500yr suites)

Considering the above points (and some other aspects not discussed), it is the authors’ view that the ULS and CLS objectives at 2500 year and 7500 year return periods respectively have likely been over-achieved. However, this does improve the asset’s resilience to future developments in understanding of the regional seismology, if this developed unfavourably. Reliable performance of the isolated structure is achieved at the 500 year and lower return periods. For higher seismic zones it could be difficult to achieve such an outcome if similar conservatisms were employed.

3.2 Lateral Load Distributions, and RHA Verification

The response-history procedure in ASCE 7-10 Section 17 permits design by verification, subject to the design actions exceeding a set of minimum actions. For this design, the minimum lateral load distributions were used for member sizing and non-linear response history analysis (RHA) used as a subsequent verification. The design base shear from Section 3.1 is \(0.115x W_t\), which ASCE 7-10 applies in a triangular distribution.

The results from the RHA were used to inform the design, and it was found that stepping down the sections in the upper levels to the minimum strength required (whilst still achieving the period target of 1.2s), resulted in higher global overturning moment ratio \(M/V\), and disproportionately higher drifts and accelerations in the middle and upper stories. This was somewhat sensitive to the isolator element model (discussed in Section 4). It was therefore elected to keep similar section depths in the upper levels, and minimise their excess in strength for capacity derived column actions by the use of reduced beam sections (although excess strength was still retained).

Through the course of design, updates to ASCE 7 were balloted which included significant changes to Chapter 17. Proposals include a dependence on isolation system damping when setting the lateral force distribution, similar to that proposed by (York and Ryan, 2008). Overturning moments are increased for systems with high damping (and high periods), and so excessive system damping is penalised. Experiences on this project support the proposed provisions on the basis that they were shown to be required for strength, and the result of their implementation is that they also improve the stiffness characteristics of taller structures, particularly if RHA is not being conducted.

4 ISOLATION SYSTEM SELECTION

A study was undertaken to inform the selection of the most appropriate isolation system type, and its refinement to optimise performance within the project constraints. The study drew from published research and vendor enquiries, supplemented with parametric studies using the ASB non-linear response history analysis model (RHA). Modified non-linear modelling elements were developed to model response of curved surface sliders with varying radii/friction, including their response to varying axial load and influence of retainer ring contact. This section presents a summary of some of the key aspects:

1. System damping across the range of return periods;
2. Influence of stiffness and damping on displacements and superstructure performance;
3. Influence of initial stiffness and low displacements hysteresis on superstructure performance;
4. Discussion of outcomes, and development of performance specifications.
4.1 Damping versus return period

Isolation systems use damping to reduce system displacements. The disadvantage of damping, is that it increases overturning moment, superstructure accelerations and drifts relative to the base shear measured across the isolation plane. Floor spectral response also increases. This effect has been previously described (for example in Kelly 1999). For systems which use hysteretic damping, and whose response is generally bilinear, the disadvantage is that at low displacements (low return period events) the effective viscous damping is high and unwanted. The damping is only wanted to control displacements at high return periods where moat dimensions and isolator stability is affected.

To illustrate this effect, simple 2-DOF matrix analysis was performed and an example structure (based on the ASB building) analysed across a range of return periods. Figure 8 shows the model schematic. The superstructure had a 1.2s period and 2% damping. Only the first system mode from the 2-DOF analysis is used. The three isolation systems tested were a simple curved surface slider (FPS), the lead rubber bearing/flat slider system (LRB), and an adaptive double pendulum system (FPS-D). Note that the LRB system has two slopes due to a proportion of the weight on sliders with stiff initial response. Real LRB hysteresis depends on a range of factors including confinement of the lead core (Kelly, 2001). The traditional method of defining the yield displacement of an LRB (used above) characterises large loop area at large displacements reasonably well, however will tend to overestimate the initial stiffness. The actual yield point (from test) is difficult to define with a bilinear element. The low return period damping will be sensitive to these assumptions.

The resulting isolation system damping and displacements are indicated in Figure 8. Systems which maintain lower damping and higher displacements at the low return periods might be expected to give good performance. Such systems include adaptive curved surface sliders (i.e. double or triple pendulums), true viscous damped systems, and also lead rubber bearing and slider combinations, provided that there is relatively low weight on sliders. The extent to which LRBs limit damping at low displacements would depend on the element model and confinement of the lead core (Kelly 2001). 

4.2 Influence of damping and post yield stiffness on peak isolator displacements

A study was undertaken using the ASB RHA model to examine effect of damping and post yield stiffness, and the performance of adaptive bearings and the effect of their hysteretic model in more detail. It used the recommended suites of 7 records scaled to the UHS for the return periods of interest. The characteristic strength and the post-yield stiffness of single pendulum systems were varied – as these are the parameters that the designer has control over. The range explored was not extensive, but bounded the properties of the proposed system. The adaptive FPS-D design from the previous section is also included (which has generally similar hysteresis to the LRB system). Results are presented in Table 1.
Figure 9: Hysteresis of Isolation Systems tested (left), and their equivalent viscous damping (centre); non-linear RHA displacement results (right) at 500, 2500 and 7500 year return periods plotted over predictions from simplified 2-DOF system using the enveloped response spectra from scaled records. The vertical scale has been adjusted to fit NZS 1170.5 R factors for this plot.

Table 1: Non-linear RHA results for ASB building with varied isolation system parameters

<table>
<thead>
<tr>
<th>System Type</th>
<th>RHA Characteristics</th>
<th>Maximum Results - Diaphragm Node</th>
<th>Max Interstorey Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>FPS</td>
<td>90</td>
<td>X Dir Disp, Vectoral Disp, Residual Disp</td>
<td>X Base Shear</td>
</tr>
<tr>
<td>FPS</td>
<td>91</td>
<td>5%, 0.20</td>
<td>486, 500, 106</td>
</tr>
<tr>
<td>FPS</td>
<td>92 7500</td>
<td>9%, 0.15</td>
<td>498, 518, 76</td>
</tr>
<tr>
<td>FPS</td>
<td>93</td>
<td>9%, 0.20</td>
<td>479, 488, 63</td>
</tr>
<tr>
<td>FPS-D</td>
<td>94</td>
<td>7%, 0.17</td>
<td>499, 517, 69</td>
</tr>
<tr>
<td>FPS</td>
<td>90</td>
<td>5%, 0.15</td>
<td>403, 412, 57</td>
</tr>
<tr>
<td>FPS</td>
<td>91</td>
<td>5%, 0.20</td>
<td>385, 394, 48</td>
</tr>
<tr>
<td>FPS</td>
<td>92 2500</td>
<td>9%, 0.15</td>
<td>391, 411, 69</td>
</tr>
<tr>
<td>FPS</td>
<td>93</td>
<td>9%, 0.20</td>
<td>368, 385, 69</td>
</tr>
<tr>
<td>FPS-D</td>
<td>94</td>
<td>7%, 0.17</td>
<td>397, 409, 85</td>
</tr>
<tr>
<td>FPS</td>
<td>90</td>
<td>5%, 0.15</td>
<td>228, 242, 35</td>
</tr>
<tr>
<td>FPS</td>
<td>91</td>
<td>5%, 0.20</td>
<td>217, 229, 36</td>
</tr>
<tr>
<td>FPS</td>
<td>92 500</td>
<td>9%, 0.15</td>
<td>159, 167, 49</td>
</tr>
<tr>
<td>FPS</td>
<td>93</td>
<td>9%, 0.20</td>
<td>151, 157, 44</td>
</tr>
<tr>
<td>FPS-D</td>
<td>94</td>
<td>7%, 0.17</td>
<td>197, 203, 37</td>
</tr>
</tbody>
</table>

Peak displacements were insensitive to the isolation system properties within the range tested. Obviously this is specific to the record governing displacement response for this project. At the 2500 and 7500 year return periods, maximum displacements for each system covered a fraction of the range predicted by displacement based methods using ASCE 7-10 damping factors. The controlling record is shown in Figure 10 (unscaled), and contains pulse-like behaviour. As displacements are similar for all systems, the stiffer systems see unnecessarily high base shears as a result.

Figure 10: The governing record Erzikan 1992 Record (NGA_821) is included in 2500 year and 7500 year suites

For this site and controlling hazard it would appear to be better to keep stiffness and damping on the lower side, whilst satisfying provisions for re-centering. However whilst the damping is not particularly effective at reducing peak displacements from high velocity pulses and therefore should not be excessive, some damping is best retained as it significantly reduces resonance with basin soils and improves the risk profile in that respect (Figure 11).

The most notable result from these analyses, is that for the similar displacements, the adaptive FPS design achieves dramatically lower drifts at the 2500 and 7500 year return periods, despite having similar levels of equivalent viscous damping at the peak displacement to the other lightly damped systems. Similar remarkable drift reductions are observed for 500 year return period, despite lower isolation system displacements. This result indicates a high dependence on the characteristics of isolator activation and not necessarily the overall damping as measured at the peak response/base shear.
4.3 Influence of Initial Stiffness of Isolation System

Following the above study, another set of studies were completed for bilinear isolation systems, varying the initial stiffness. York and Ryan (2008) noted that their studies and proposed k factors for lateral load distribution were based on 1 cm yield deformations, commenting that yield deformation of isolators (or their characteristics in general) would probably have some affect. The ASB study focussed on measuring and comparing drift and floor spectral accelerations rather than the deriving of equivalent static lateral load distributions, however the effects would be related. The adaptive FPS-D design was used as a benchmark, and bilinear systems with low and high initial stiffness were included (Figure 12). A bilinear model with equal damping (equal under-over area) to the FPS-D model was also included, as this has been suggested by vendors as a simpler model for practitioners to use when analysing curved surface sliders with multiple sliding regimes (similar to the FPS-D used in this study) for ULS and MCE (CLS) return periods.

Similar results are found, summarised in Table 2. It is not possible to conclude from such few results whether the equal damping model should give similar displacements, however it gives a conservative result here which seems to agree with other observations of performance.

<table>
<thead>
<tr>
<th>RHA Run</th>
<th>Return Period</th>
<th>System Characteristics</th>
<th>Maximum Results - Diaphragm Node</th>
<th>Max Cnr Col Interstorey Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>94</td>
<td>7500</td>
<td>Low (3 x Kr)</td>
<td>0.165</td>
<td>0.130 0.42%</td>
</tr>
<tr>
<td>96</td>
<td>2500</td>
<td>FPS-D</td>
<td>6.8% 0.165</td>
<td>0.123 0.38%</td>
</tr>
<tr>
<td>97</td>
<td>Eq. Damping</td>
<td></td>
<td></td>
<td>0.135 0.57%</td>
</tr>
<tr>
<td>99</td>
<td>Stiff</td>
<td></td>
<td></td>
<td>0.124 0.53%</td>
</tr>
<tr>
<td>94</td>
<td>500</td>
<td>FPS-D</td>
<td>6.8% 0.165</td>
<td>0.094 0.26%</td>
</tr>
<tr>
<td>96</td>
<td>Low (3 x Kr)</td>
<td></td>
<td></td>
<td>0.103 0.31%</td>
</tr>
<tr>
<td>97</td>
<td>Eq. Damping</td>
<td></td>
<td></td>
<td>0.100 0.37%</td>
</tr>
<tr>
<td>99</td>
<td>Stiff</td>
<td></td>
<td></td>
<td>0.107 0.43%</td>
</tr>
</tbody>
</table>
The trilinear FPS-D model significantly outperforms the other models as measured by superstructure drifts and floor spectral accelerations, despite lower isolation system displacements. Results for the 500 year return period (SLS2) are presented graphically in Figure 13 as it is an important performance objective. The trend is similar for other return periods however; even at the 7500 year return period the peak drifts for the bilinear equal damped model are 50% greater than the detailed trilinear model.

![Figure 13: Superstructure drifts and floor acceleration response spectra for 500 year return period](image)

4.4 Discussion, and Isolation System Performance Specification

4.4.1 Hysteretic Isolation Systems in Near Source Environments

For isolation systems in environments where near source motions contribute, displacements are significantly affected. Early agreement on record selection and scaling was important for this project. The authors understand that the NZSEE Base Isolation Guidelines currently under development will include dependence of damping on source distance in a manner similar to that proposed by (Priestley 2007, pp. 59), which is supported by these studies.

4.4.2 Dependence on Low Displacement Hysteretic Performance and Break-away

Superstructure performance, when measured by superstructure accelerations, floor spectral accelerations and inter-storey drifts, exhibited a significant dependence on isolation system breakaway and hysteretic characteristics at low displacements. Softening systems with low base shears at the initial breakaway, and a high degree of non-linearity or continually varying stiffness appears to significantly reduce resonance of superstructure modes. This was observed even when peak displacements or base shears are reached significantly beyond the softening range, and when overall damping was similar. Isolation system displacements were also affected but to a lesser extent. This effect appears to be equally as important as equivalent viscous damping in influencing the ability of an isolation system to reduce damage to building contents (where higher damping reduces effectiveness).

Lead rubber bearings exhibit non-linear softening response, however if they are combined with flat sliders, then it is preferable that the friction contributed by the sliders is kept low by keeping the percentage of seismic weight on sliders below 30-40%. This system therefore suits heavier structures (in terms of seismic weight per bearing). Friction systems have been developed with multiple sliding regimes, and a substantial amount of research has been published, for example by Zayas and Constantinou. These systems would particularly benefit lightweight structures with low mass per bearing. Low friction coefficients are required, sometimes with very high contact pressures. Specifiers need to develop confidence that the specified properties will be achieved (for example by appropriately specified prototype testing considering scale, speed and number of cycles), and also that sufficient consistency in performance over the lifespan is achieved and that the corresponding property modifications factors are appropriate. Whilst single surface curved sliders gave poorer analysed performance, it is thought that part of this effect could be due to computer modelling, as these devices and their supports do exhibit some variability in characteristics in a real structure.

4.4.3 Development of Performance Specifications for the ASB Project

Considering the outcomes listed above, including the comment on modelling uncertainty, it was decided that the isolation system should include softening response at low displacements, as such systems are available from the market. This performance could be delivered using either elastomeric or friction systems, and so a dual specification was developed for the project. This outcome may not be the same for other projects. As a result of the study outcomes the curved surface slider specification contained some detailed requirements to ensure equivalent performance to the lead rubber bearing system. Only adaptive systems with multiple sliding regimes would comply, and articulated pucks were necessary.
High quality tender submissions were received from the market for both systems. Each system had some variances in their implementation and in their spatial/coordination requirements, however neither presented a clearly distinguishable advantage from a technical performance point of view. A lead rubber bearing and flat slider option was selected by the client following consideration of all tender aspects including technical review inputs from the design team.

5 CONCLUSIONS

Studies undertaken as part of the design of the Christchurch Hospital Acute Services Building (a 10 storey IL4 base-isolated moment frame) were presented as a case study, focussing on the refinement of the isolation system specification within the project constraints and specific seismic hazard. The study considered the basic parameters available to the designer (system characteristic strength and post yield stiffness), as well the potential benefits of softening response of lead-rubber systems – also achieved by adaptive (multi-regime) sliding systems available in the market.

Added damping at the isolation plane has a generally detrimental effect on superstructure performance (drifts and floor accelerations), however this was shown to be sensitive to the hysteretic model at isolator activation. For similar overall base shears and damping, improved performance was observed when low initial yield forces and softening response was used. Such systems have the widely understood benefit that they can provide better superstructure protection in low return period events (due to a low initial yield force). However even in higher return periods when peak displacement and base shear occurred well beyond the softening range, the benefits were shown to be substantial for this 10-storey moment frame structure. Simple single surface friction bearings (friction pendulums) performed poorly in the study compared to their adaptive counter-parts at all return periods, however the extent to which the computer model contributed to this result was uncertain. It is possible that more variation and support flexibility exists in real isolated structures than was implied by the modelled elements.

Whilst isolation system displacements at the high return periods showed less dependency on the hysteretic model, practitioners and researchers using RHA to investigate superstructure performance (e.g. drifts and floor accelerations) should be aware that the results may be highly dependent on the type of hysteretic model used, and should consider the sensitivity of the response quantity being measured to adjustments in this model. Bilinear models may be too approximate.

Other outcomes from the ASB design supported an adjustment to damping factors for SDOF displacement-based design procedures for near-source hazards, and also supported the inclusion of modifications to the lateral force distributions which increase the overturning moment for systems with higher damping.

REFERENCES