PRELIMINARY OBSERVATIONS OF THE EFFECTS OF THE 2010 DARFIELD EARTHQUAKE ON THE BASE-ISOLATED CHRISTCHURCH WOMEN'S HOSPITAL

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SUMMARY

The Christchurch Women's Hospital, completed in March 2005, is the only base-isolated building in the South Island of New Zealand. The displacement capacity of the base-isolation system and the super-structure ductility capacity are designed to meet 2000-year return-period demands. Detailed structural evaluations after the 2010 Darfield Earthquake revealed damage only to sacrificial non-structural components at the seismic gaps. Because the structure is not instrumented, basic design information and ground motion records from nearby sites are used to estimate the responses to the main shock and a large after-shock. Results from this modelling effort are used to corroborate reports of structural response from staff present at the time of the main shock and aftershocks. Issues meriting further investigation are related to the local site conditions, soil-structure interaction, super-structure dynamics, interaction with the adjacent structures, and large-deformation effects.

INTRODUCTION

The isolation of dynamically-sensitive objects from vibratory environments via compliant and damped supports is common in machinery and automotive applications. Because the damaging effects of earthquakes are primarily related to the horizontal components of ground motions, the principle of vibration isolation can be applied to structures using interfaces that are horizontally compliant yet vertically stiff enough to support the weight of the structure. Seismic isolation decouples the response of isolated buildings from ground motions via laterally-compliant bearings that de-tune the building's natural period from the dominant periods of the ground motion and absorb seismic energy via damping mechanisms or materials. Challenges in the protection of buildings from earthquakes using seismic isolation bearings pertain to the needs for large vertical-to-horizontal stiffness ratios, large damping forces, compact geometries, and a very high level of reliability. These challenges were met with the 1975 invention of the lead-rubber seismic isolation bearing (LRB) in New Zealand [1,2,3,4]. The large vertical-to-horizontal stiffness ratio of LRB's is achieved by laminating multiple layers of rubber sheets (approximately 5 mm thick and with a Poisson's ratio close to 0.5) with steel plates (approximately 3 mm thick). A vertical cylindrical lead core within the bearing provides most of the energy-dissipation properties via inelastic shear deformation. After a period of development in New Zealand, Japan, Taiwan, and the U.S [5,6,7]., lead-rubber seismic isolation bearings were first implemented in 1978 in the four-story William Clayton building, Wellington, N.Z. [8]. By 1993 the numbers of buildings with base isolation were seven in New Zealand, eleven in the U.S., and fifty-seven in Japan [4].

Emergency preparedness studies following the 1989 Loma Prieta earthquake revealed that 83 percent of California's hospital beds were in seismically inadequate structures and 26 percent of the hospital beds were in buildings at risk of collapse during an earthquake [9]. Around that time seismic isolation was selected for new California hospitals including hospitals in San Bernardino [10] and the University of Southern California [11,12]. Nevertheless, after the Mw 6.8 17 January 1994 Northridge earthquake, 23 hospitals in the earthquake-affected region were inoperable [9]. The damage sustained by hospitals in the Northridge earthquake led to passage of California State Senate Bill 1953 [9], amending the Alfred E. Alquist Hospital Seismic Safety Act of 1983, and requiring the replacement or retrofit of seismically deficient acute-care facilities by 2008 and of all buildings at hospitals with acute-care facilities by 2030.

The LRB isolated Hospital of the University of Southern California is instrumented via the California Strong Motion Instrumentation Program with 24 uni-directional accelerometers within the structure and a tri-axial accelerometer to measure free-field response near the site. Response measurements taken from the USC Hospital during the Northridge earthquake provided the first opportunity to evaluate the field performance of a large base-isolated building experiencing seismic responses in the inelastic range [11]. The displacement capacity of the LRB isolation system of the USC hospital is 260 mm and meets demands having a 450 year return period [11]. The yield displacements of the LRB's is approximately 20mm and the peak displacement of the isolators during the Northridge earthquake was approximately 36 mm (ductility = 1.8) with a period of motion slightly longer than 1.0s [11]. (The USC Hospital is 36 km from the Northridge earthquake epicentre.) Subsequent system identification of the linear and non-linear dynamic characteristics of the USC Hospital illustrate that calibrated inelastic models are capable of reproducing observed responses [12]. The super-structure acceleration responses were just 50% of the base acceleration levels. Super-structure story drifts were 30% of the maximum code-allowed limits [12]. Hospital staff reported “gentle swaying” motion during the earthquake, and no toppling of contents. The overall conclusion from the USC Hospital studies were that this structure “performed well” in the Northridge earthquake, which demanded less than 15% of the isolation system's displacement capacity.

A number of other smaller base-isolated structures were in the area affected by the Northridge earthquake. The Los Angeles County Fire Command and Control Facility (38 km from the epicentre) sustained only non-structural damage where the seismic gap had been filled with grout. The Los Angeles County Emergency Operations Center (40 km from the epicentre) sustained no damage during the Northridge earthquake. Two 3-story residential buildings

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on spring-and-damper isolation systems experienced an increase in acceleration across the isolator level [13].

The only seismically-isolated building in Kobe at the time of the 17 January 1995 Hyogoken-Nanbu (Kobe) earthquake was 30 km from the epicentre. Recorded acceleration levels at the top (6^{th}) floor were 25% to 30% of the recorded base accelerations, and were less than 10% of what they would have been in a comparable fixed-base structure [14].

Since the 1995 Hyogoken-Nanbu earthquake and the 1994 Northridge earthquake, base-isolation systems have become increasingly popular elements of seismic resistant design. Notably, Japanese construction firms were quick to adopt seismic isolation solutions. The number of isolated structures in Japan rose from about 100 in 1995 to about 600 in 1998 [15]. The adoption of seismic isolation in the U.S. has been slower to take hold, partially due to stringent code requirements [5].

In the M7.9 Wenchuan China Earthquake of May 12, 2008, isolation system displacements in three similar apartment buildings in Wudu were inhibited by trim fascia, utility pipes, and side-walk slabs. The side-walk slab reaction forces at one of these apartments caused torsional motion and associated hairline cracking in the building above the interface [16].

Construction of seismically isolated buildings entails flexible linkages in the mechanical systems crossing the isolation interface to allow for large isolation displacements. Increasing the deformation capacity of an isolation system, therefore, comes with some extra costs, both in terms of the isolators themselves and in terms of detailing the mechanical system. Furthermore, if the elements and connections in the super-structure are not designed for ductile behaviour, should an earthquake's demand exceed the isolator's capacity, isolator failure could potentially lead to significant structural damage. In light of the large isolator displacement demands associated with long-period pulses observed in the near-fault ground motion records, observations of the response of base-isolated structures to long-period excitations is of great interest.

THE CHRISTCHURCH WOMEN'S HOSPITAL

The Christchurch Women's Hospital is a 20,000 sq.m, 45-bed facility which includes 14 birthing suites, neonatal wards, and out-patient surgery units. The Hospital opened in 2005 and remains the only base isolated structure in the South Island. The decision for seismic isolation was based on experiences of seismically isolated structures in the Northridge and Kobe earthquakes (the USC Hospital in particular), the recognition that the last M 8 rupture of the Alpine fault was almost 300 years old, and that a significant portion of the seismic hazard in Christchurch is associated with distributed seismicity in the Canterbury plains [17,18].

The hospital building is regular in plan and elevation. The footprint is 76 m x 32 m; the height is 33 m; and the building has nine levels, including the “Ground”, “Lower Ground” and “Basement” (the base isolation gallery). The lower foundation comprises a massive concrete raft (1.2 m thick) and a perimeter retaining wall. The raft foundation bears on a 5-7 m thick layer of dense consolidated gravel over layers of soft silt [18]. The upper foundation (sometimes called the transfer slab or frame) above the isolation plane is a two-way grillage of 1.0m deep beams.

The super-structure of the Christchurch Women's Hospital consists of a perimeter frame of reinforced concrete cast-in-place columns with pre-cast beam elements and shop-welded steel vee-bracing in the lower four levels. See Figure 1. The top two levels contain building service equipment and a three-day supply of water in four 10,000 litre tanks. The 1997 Uniform Building Code (UBC) contains provisions for base-isolated buildings and was therefore adopted for the design. The frame, cladding and stairs were detailed to inter-story drifts corresponding to a ductility of 1.8. The 1997 UBC specifies that seismic isolation bearings should not take tension loads. To meet this requirement, steel caissons, 1.2m in diameter, were driven to a depth of 6m and anchored to the slab foundation. A post-tensioned cable from the base of each caisson to large transfer girders were designed to prevent uplift and any associated tension in the seismic bearings [17,18].

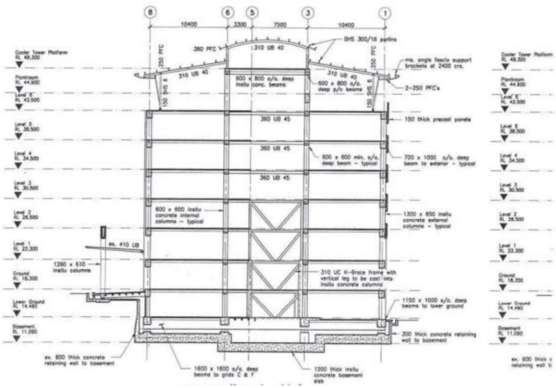

Figure 1: Elevation view [18].

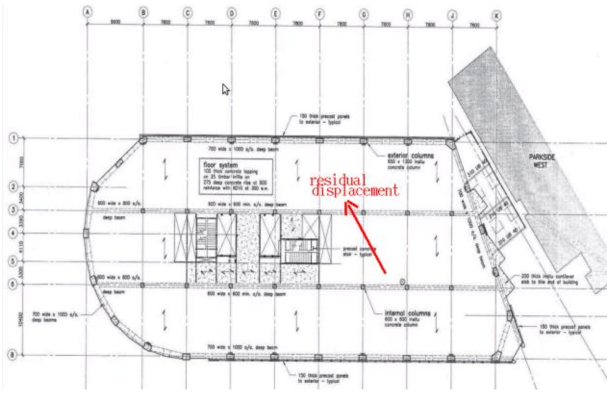

Figure 2: Plan view [18].

The seismic isolation system has 41 LRB's (830 mm x 830 mm x 275 mm) made with natural rubber. Twenty-five of the LRB's support the perimeter frame columns and the remaining sixteen bearings support interior columns. Four pot bearings carry gravity loads under the vee-braced frames. See Figure 2. The isolation system has a +/-420 mm displacement capacity corresponding to a 2,000 year return period [17,18]. Each bearing was tested prior to installation.

The base-isolated Christchurch Women's Hospital building connects to the adjacent Parkside West building through seismic joints, with sacrificial wall panels, ceilings, and floor mats.

Observations of the Effects of the 4 September 2010 Darfield Earthquake on the Christchurch Women's Hospital

All area hospitals remained operational during and after the main shock (4/9/2010 4:36 am), including the Christchurch Hospital Emergency Department. Backup generators for Christchurch and Burwood Hospitals were operational within one minute of losing power. Utility power to these facilities was restored within 80 minutes. Although the earthquake caused no serious injuries to hospital occupants, night staff remained on duty throughout the morning. Morning shift staff were advised to use discretion in deciding to travel to work.

A 25mm residual displacement in the isolator system in the north direction was observed during inspections on 14 September 2010 and 1 November 2010 (See Figure 3.) This residual displacement is roughly parallel to the interface between the Women's hospital and the adjacent Parkside West building as shown in Figure 2.

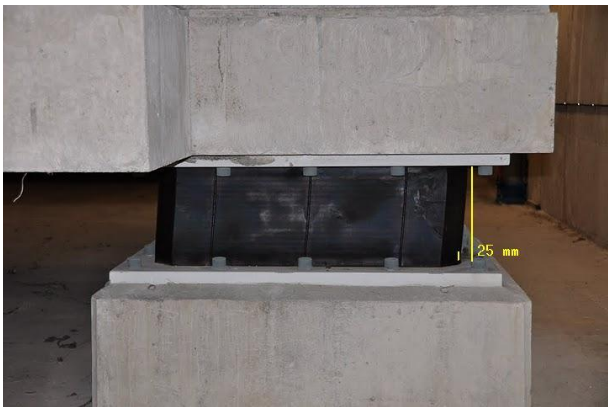

Figure 3: Residual Displacement of LRB at column F-3. (photo F.M. Turner)

Figure 4: Damage at the seismic gap between the Christchurch Women's Hospital and the Parkside West Building at Level 5 (photo W.Y. Kam)

A one-inch water pipe crossing the isolation interface had been installed without the required flexible coupling to allow for large differential displacements. This water pipe fractured during the earthquake. Evidence of sliding across the seismic gap was evident the morning after the earthquake and indicated that the peak displacement across the isolation interface was about 40 mm in the north-south direction.

Damage to sacrificial components was evident up the height of the building with larger motion in the north direction. Damage at the seismic gap between the Women's Hospital and the Parkside West building, Figure 4, shows crushed concrete or grout. The residual displacement is also evident in a buckled cover plate at grade that had been grouted on the sliding edge, as shown in Figure 5.

Cracks in gypboard were visible in stairwells above the isolation level.

Personal Accounts of Motion in the Christchurch Women's Hospital

Hospital staff were given an opportunity to provide personal accounts of their experience during the earthquake.

Above level 5, motion of an air-handling unit exceeded the horizontal displacement capacity of the unit's coil-spring isolation system.

Staff on duty at level 5 (the highest occupiable level) reported rolling trolleys, items falling from shelves, clocks falling from walls, items falling out of refrigerators, and the toppling of an unsecured bookshelf (the only unsecured shelf on that level).

Staff on level 4 reported rolling trolleys and the need to hold on to hand-rails to keep from falling over during the main shock. Other staff on level 4 reported that during aftershocks the building felt as if it were “a boat on a rough sea” with a “constant rolling effect, long after the shocks” were over.

Staff on level 3 (the maternity ward) reported “furious” “side-to-side” swaying with increasing intensity, items falling from shelves, loss of electricity, the sloshing of water from a full birthing pool to a distance of 2 to 3 m from the pool, and a number of distressed staff.

A fire alarm on level 2 was triggered when electricity was restored (within a minute).

During an aftershock, staff on level 1 reported a “vibrating effect” rather than a “swaying effect.” The aftershock sequence has led to motion-sickness complaints from some staff working in levels 4 and 5.

Figure 5: The seismic moat cover plate on ground level at the South corner of the Christchurch Women's Hospital was secured along both edges. (photo W.Y. Kam).

Interpretation of Personal Accounts of Motion

The Christchurch Women's Hospital is not instrumented, but may very well have undergone greater inelastic deformation in the isolation system than the USC Hospital experienced in the Northridge earthquake. By all accounts, the motion within the Christchurch Women's Hospital during the Darfield Earthquake was more severe than reports from the USC Hospital during the Northridge Earthquake. The following sections outline an initial attempt to estimate the response of the isolation system using basic engineering data, ground motion records at two nearby sites, and a simple inelastic response analysis. An objective of this part of the work is to reconcile the observations of hospital staff with results of engineering analysis, and to indicate areas meriting further investigations.

SIMPLIFIED MODELLING

Basic Design Data

The four basic design parameters of a seismic isolation system are the yield displacement, D_y, the yield strength coefficient (the ratio of the isolation system yield force to the weight of the structure), C_y, the post-yield stiffness ratio (the ratio of post-yield to pre-yield stiffness), κ, and a viscous damping ratio, ζ. The basic isolation design data available at the time of this writing describes the relation between the base-shear coefficient (the ratio of base shear to structural weight) to the displacement amplitude, and the associated return period, as shown in Figure 6. Site-specific design spectra for the isolation system were also made available. The data in Figure 6 specifies the post-yield stiffness. Given a yield displacement and the post-yield stiffness, a corresponding yield strength ratio is easily computed, from which values for the pre-yield stiffness and the post-yield stiffness ratio can be computed. So, the design data provided reduces the problem to two independent unknown parameters: the yield displacement and the viscous damping ratio.

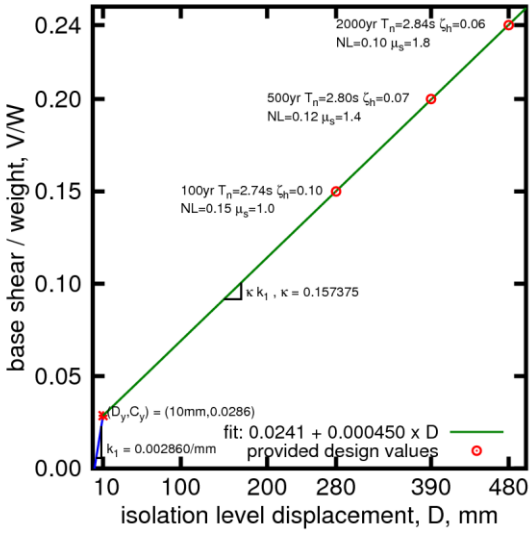

Figure 6: Base shear coefficient vs. isolation displacement for the Christchurch Women's Hospital.

Bi-axial Inelastic Transient Response Simulation

The four isolation system parameters (D_y, C_y , κ , and ζ) are sufficient to define a bi-axial hysteretic model of the isolation system. The Park-Wen-Ang bi-axial hysteretic model [19] is used in a simplified representation in which the super-structure is considered to be a rigid mass with displacements r_x and r_y in the horizontal x and y directions. The biaxial, mass-normalized, shear forces in the isolation system, f_x and f_y are modelled by Equation (1), below:

$$
\begin{aligned}
f_x &= C_y g\left((1-\kappa)z_x + \kappa u_x\right) \\
f_y &= C_y g\left((1-\kappa)z_y + \kappa u_y\right)
\end{aligned}
\qquad (1)
$$

where g = gravitational acceleration;

u_x = dimensionless displacement, r_x / D_y;

u_y = dimensionless displacement, r_y / D_y;

and the hysteretic variables, z_x and z_y, evolve according to

$$
\dot z_x = \frac{\dot r_x}{D_y} - \left(\left(\alpha sgn\left(z_x \dot r_x\right) + \beta\right) z_x \dot r_x + \left(\alpha sgn\left(z_y \dot r_y\right) + \beta\right)\right)\frac{z_x}{D_y}
$$

$$
\dot z_y = \frac{\dot r_y}{D_y} - \left(\left(\alpha sgn\left(z_y \dot r_y\right) + \beta\right) z_y \dot r_y + \left(\alpha sgn\left(z_x \dot r_x\right) + \beta\right)\right)\frac{z_y}{D_y}
$$

(2)

where α = 0.8 and β = 0.2 and the equations of motion are:

$$\ddot{r}_x + 2\zeta\omega_n\dot{r}_x + f_x = -a_x$$
$$\ddot{r}_y + 2\zeta\omega_n\dot{r}_y + f_y = -a_y \qquad (3)$$

where $\omega_n = \sqrt{C_y g / D_y}$ is the initial natural frequency and a_x and a_y are the x and y components of ground acceleration.

Given the four isolation parameters and a set of bi-axial ground motion acceleration records, Equations (1), (2), and (3) may be solved to determine the bi-axial hysteretic response of the isolated structure. This model assumes that the structure is a rigid doubly-symmetric mass, and neglects soil-structure interaction effects and interaction with adjacent buildings.

Nearby Ground Motion Records

Two ground motion stations are near the Hospital site: the Christchurch Botanical Gardens (CBGS) 500 m from the Women's Hospital station and the Christchurch Hospital (CHHC) station, 250 m from the Women's Hospital building. The Women's Hospital lies close to the line between these two stations, as shown in Figure 7. Corrected acceleration, velocity, and displacement records at these two stations from the main shock of 4:36 am 4/9/2010 and a large aftershock of 10:32 am 18/10/2010, were obtained from The Institute for Geological and Nuclear Sciences, Wellington NZ [20]. For the main shock the PGA and PGV were 0.20g and 50 cm/s at the CHHC station and 0.18g and 40 cm/s at the CBGS station. Response spectra are plotted in a following section.

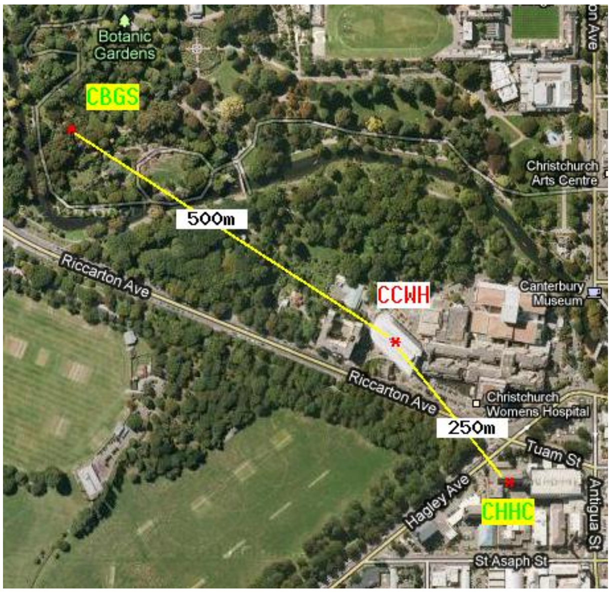

Figure 7: Location of the Christchurch Women's Hospital (CCWH) in relation to two nearby ground motion recording stations: Botanical Gardens (CBGS) and Christchurch Hospital (CHHC) [19]. (image google maps.)

Simulation Results

Using a range of values for D_y (5 mm < D_y < 10 mm) and a set of values for ζ (ζ = 0.05, 0.10, 0.15) a series of simulations were executed with ground motion records for these two stations. Viscous damping ratios provided by natural rubber isolation bearings (without a lead core) have been reported to be about 12% to 15% [4,5,21]. From each simulation the peak displacement response,

$$max|D| = max\sqrt{r_x^2 + r_y^2} \qquad (4)$$

and peak total acceleration response,

$$max|A| = max\sqrt{(\ddot{r}_x + a_x)^2 + (\ddot{r}_y + a_y)^2} \qquad (5)$$

were calculated. In addition, the period of motion was computed from the average displacement up-crossing rate for the largest 15 cycles of the response. Example results from this parameter study are given in Figures 8, 9 and 10 for the 4/9/2010 4:36 am ground motion record from the CHHC station. Assuming a value of 15% for ζ, the peak isolator displacements from the CHHC record of 4/9/2010 4:36 am would be about 170 mm. The peak floor accelerations would be about 0.145g and the period of motion would be about 2.05 s. For the CBGS station record from 4/9/2010 at 15% damping the corresponding values are 110 mm, 0.085g, and 1.6 s. For the aftershock of 18/10/2010 10:32 am the corresponding values are 15 mm, 0.044g, and 1.1 s for the CHHC station and 12 mm, 0.038g, and 1.0 s for the CBGS station. This model indicates that the isolator displacements from the main shock could have been two to three times larger than measured at the USC Hospital during the Northridge earthquake. These isolation-level displacements could have been more than one-third of the ultimate capacity. This result would be consistent with staff reports from these two events. Residual displacements computed from this simple model were much smaller than those observed in the actual isolation system.

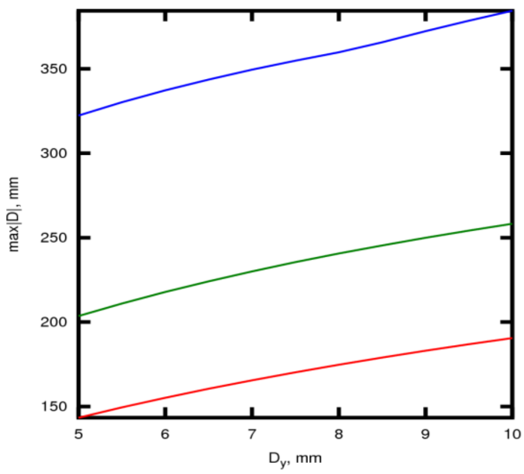

Figure 8: Modelled peak isolator displacements for different values of isolation yield displacements and ζ = 0.05, 0.10, 0.15 (red).

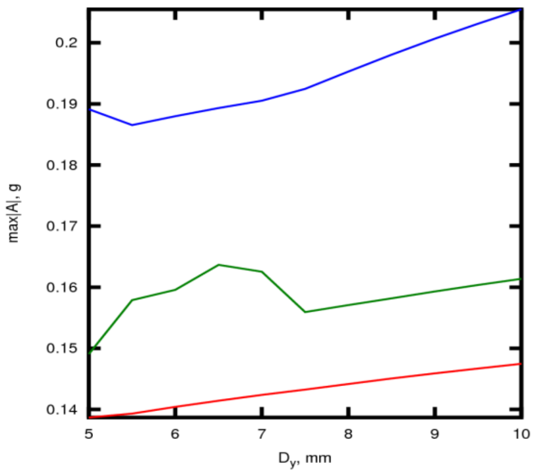

Figure 9: Modelled peak total response accelerations for different values of isolation yield displacement and ζ = 0.05, 0.10, 0.15 (red).

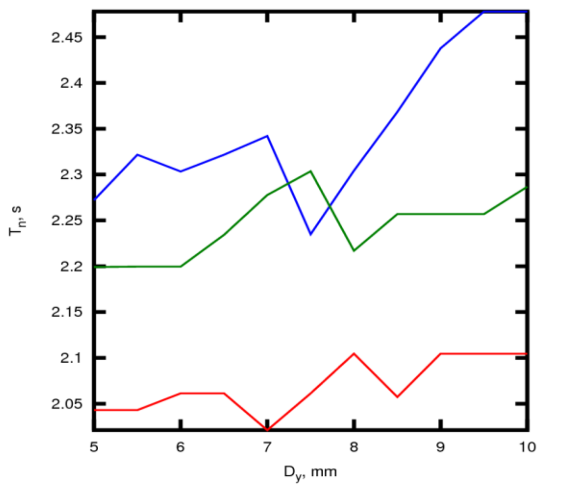

Figure 10: Modelled period of motion for different values of the yield displacement and ζ = 0.05, 0.10, 0.15 (red)

Return Period of Modelled Responses

Given the base isolation system parameters from the previous analysis, and the response to the main shock and the aftershocks for the CHHC and the CBGS station records, it is possible to generate constant ductility response spectra for these parameters. These constant ductility spectra are computed using the same type of model for smooth hysteretic behaviour that was utilized in the base-isolation response analyses [21]. The spectra for the main shock records of 4/9/2010 4:36 am have a constant ductility of 20, a post-yield stiffness ratio of 0.16, and a viscous damping ratio of 12%. Spectra for the 18/10/2010 aftershocks have a ductility of 2. The red lines are for the CHHC station and the blue lines are for the CBGS station. The green line represents the 5% damped elastic response spectrum for the CHHC station. All spectra are the geometric mean of spectra of the orthogonal horizontal components. Note that the large spectral amplitudes at a 3 second period for elastic response are substantially suppressed by the hysteretic and viscous damping in the constant ductility response spectra. It should be noted here that the design spectra presented in Figure 11 may under-represent the hazard at long periods arising from the alluvial deposit underlying most of Christchurch.

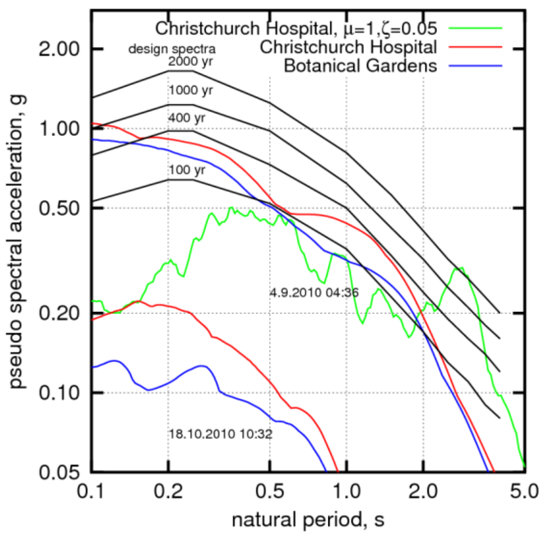

Figure 11: Design spectra and response spectra for the main shock and an aftershock.

The results of this analysis indicate that the response amplitude of the Christchurch Women's Hospital to the main shock of the Darfield Earthquake has a return period of approximately 100 years.

Sloshing and Sensitivity to Small Amplitude Motion

Two notable observations from staff in the Christchurch Women's Hospital relate to the sloshing of water from the full birthing tub and motion-sickness symptoms. The birthing tub is essentially cylindrical with a diameter of 1,080 mm at the bottom and 1,180 mm at the top. The tub is 650 mm deep. Measurements of sloshing frequencies at various depths of water are consistent with George Housner's 1963 analysis [23], as shown in Figure 12. A more recent analysis for sloshing periods for spherical tubs is shown as well [24]. Housner's formulation includes a prediction for slosh amplitudes. The formulation predicts slosh amplitudes as shown in Figure 13 for lateral floor displacement amplitudes of 100 mm, 150 mm and 200 mm. For a sloshing period of T_s and a floor motion period of T_f the effective lateral water displacement amplitude is roughly $(T_s\ /\ T_f\ )^2\ /\ (\ 1\ -\ (\ T_s\ /\ T_f\ )^2\ )$ times the floor displacement amplitude. For a floor period of 2.1 seconds and a slosh period of 1.2 second, this comes to about 50% of the floor displacement amplitude

Using Housner's 1963 analysis Figure 13 shows that the estimated floor displacement amplitude of 200 mm would result in a significant volume of water leaving the tub and supports observations from staff at the time of the main shock.

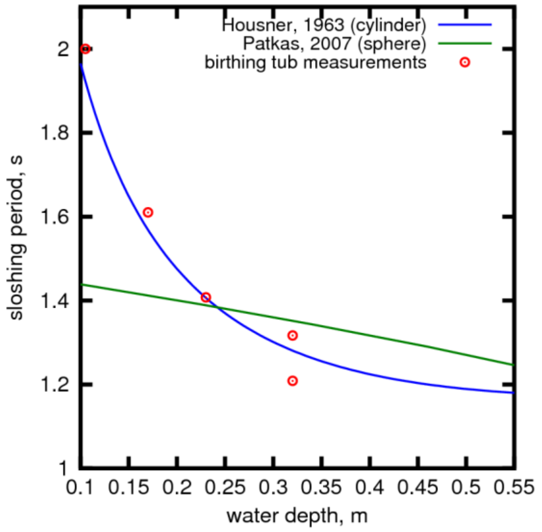

Figure 12: Dependence of sloshing frequency of birthing tubs on water depth.

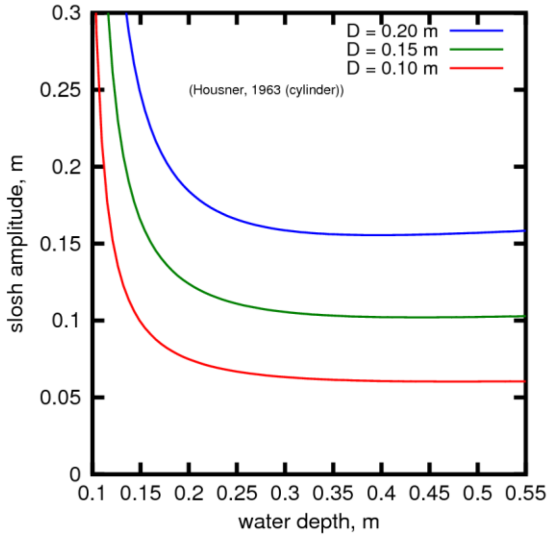

Figure 13: Dependence of sloshing amplitude on water depth and lateral floor displacement, D. Floor motion period = 2.1 s

In terms of the sensitivity to small amplitude motion, the modelled responses to measured ground acceleration records of a relatively large aftershock indicate that the peak structural accelerations are approximately 40 mg with a period of roughly 1.0 s. The free vibration portion of the response would have amplitudes of 5 to 20 mg. People are most sensitive to floor vibrations with periods of 0.3 s to 1.0s. The threshold of perception to floor accelerations at a 1.0s period is 0.3 mg for sensitive people and 1.5 mg on average. The threshold for adverse comments regarding floor vibration at a 1.0 s period is approximately 3-7 mg [25,26,27]. The model responses therefore agree with reports that aftershock motions produced adverse affects in some of the building occupants. The long duration of responses from after-shocks and the residual displacement are not explained by this simplified model. Furthermore, the cursory visual inspection immediately after the earthquake indicates that peak displacements may have been significantly smaller than predicted by the simplified model. Ongoing work in this direction will include soil-structure interaction [28,29], torsional response, and interaction with the adjacent Parkside West building.

SUMMARY

The 4 September 2010 Darfield (Canterbury) earthquake generated the first significant seismic response of any base isolated structure in New Zealand and one of the largest responses for base isolated structures world-wide.

Preliminary and simplified numerical simulations of the response of the base-isolated Christchurch Women's Hospital to the 4 September 2010 Darfield (Canterbury) earthquake indicate that the isolation system displacements could have exceeded the yield levels of the lead-rubber bearings by at least an order of magnitude. Computed responses are consistent with motions having a 100-year return period. These preliminary results are in accordance with reports of the motion of the structure and its contents as reported by hospital staff. This simplified model, which includes bi-axial inelastic behaviour of the isolation system but which does not include torsional response, super-structure flexibility, soil-structure interaction effects, or interaction with the adjacent building, does not explain the magnitude or direction of the residual displacement. Modelling interferences in the isolation interface and at the seismic gap between the Women's Hospital and the Parkside West building could help explain the observed residual displacements. Such interferences will affect super-structure responses as well. Further, incorporating soil-structure interaction effects can help explain the smaller-than-expected displacements across the isolation interface.

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