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A CASE STUDY OF DISPLACEMENT-BASED SEISMIC ASSESSMENT OF AN EXISTING HOSPITAL BUILDING IN TIMARU, NEW ZEALAND

Umut Akguzel[1], Weng Yuen Kam[1], Matt Cameron[2] and Samir Govind[3]

ABSTRACT

In this paper, a practical example on the quantitative seismic assessment of an existing concrete shear wall hospital building located in Timaru, New Zealand is presented.

In the assessment, instead of traditional force-based approach (FBA), displacement based approach (DBA) is employed. DBA procedure is recognized to be a reliable and practical assessment tool which can be implemented in the hand analysis easily. It provides a clear understanding of the probable inelastic deformation mechanisms and available displacement ductility of the existing structure.

Following a particular emphasis on the current seismic evaluation practice in New Zealand based on the New Zealand Society for Earthquake Engineering (NZSEE) guidelines, the DBA procedure used for the assessment of the Timaru Hospital Clinical Services Building (CSB) is explained in detail.

In the assessment, the building's Ultimate Limit State (ULS) earthquake resistance is compared with the current New Zealand Building Code requirements for an equivalent new building constructed on the site. Afterwards, the seismic assessment procedure and results are given for the CSB, which comprises two buildings; the Main Block and the Southern Wing. These buildings were designed and constructed in approximately 1971, and structurally connected together at each floor circa 1999. The seismic retrofit options are listed to address the identified critical structural weaknesses recognizing the need to minimize business interruption, cost etc.

This paper is aimed to contribute to the current engineering practice with an actual example and to provide some discussion on the applicability of the DBA method to determine the retrofit options based on different level of demand.

INTRODUCTION

The 7.1 Mw (Moment Magnitude) Darfield earthquake occurred on 4 September 2010 approximately 40 km west of Christchurch on a previously unknown fault within the Canterbury Plains. Since the Darfield earthquake, more than 7,000 aftershocks with Magnitude (MW) up to 6.2 have been recorded by the New Zealand national seismograph network (GeoNet). This sequence of earthquakes is termed the Canterbury earthquake sequence (GNS, 2011). The most destructive earthquake of the Canterbury sequence occurred on 22 February 2011. The epicentre of was approximately 10 km away from Christchurch central business district. This Mw 6.3 earthquake caused severe structural damage in large number of reinforced concrete (RC) buildings (Kam et al., 2011), and the collapse of two RC

[1] PhD, Senior Structural Engineer, Beca Ltd., New Zealand
[2] Associate - Structural Engineering, Beca Ltd., New Zealand
[3] Technical Director - Structural Engineering, Beca Ltd., New Zealand

buildings resulting in large number of fatalities (Beca, 2011; Hyland et al. 2012).

As a result of the Canterbury Earthquakes, building owners throughout New Zealand have become more aware of seismic risk and vulnerability of their buildings. In this paper a case study is presented on the Displacement Based Assessment (DBA) of the existing Clinical Services Building (CSB) at Timaru Hospital.

The development of procedures encompassing displacement-based design or assessment approach represents a relatively recent development compared to traditional force-based approach. While it is generally considered that displacement-based methods produce more rational and less conservative assessment outcomes, it is acknowledged that most designers are currently more familiar with force-based approaches. Some key publications on the deficiencies of traditional force based approaches can be found in literature (Moehle, 1996; Priestley et al. 2007; Kam et al. 2013).

Case studies on the application DBA for existing wall structures are rather limited in literature. In this contribution, a practical example on the quantitative seismic assessment of an existing concrete shear wall hospital building hospital building located is presented. Preliminary structural calculations undertaken for assessed critical sections, mainly the shear walls, using the 'Displacement Based Method' (1) to determine the likely building performance and damage patterns, (2) to identify any potential critical structural weaknesses, and (3) to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS) and its associate earthquake risk grade.

BUILDING DESCRIPTION

The Main Clinical Services Block and the Clinical Services Southern Wing were designed in approximately 1971. The buildings were structurally connected together at each floor circa 1999 before the introduction of modern seismic code in New Zealand circa 1976 (NZS 4203, 1976). The layout of the buildings is given in Fig. 1.

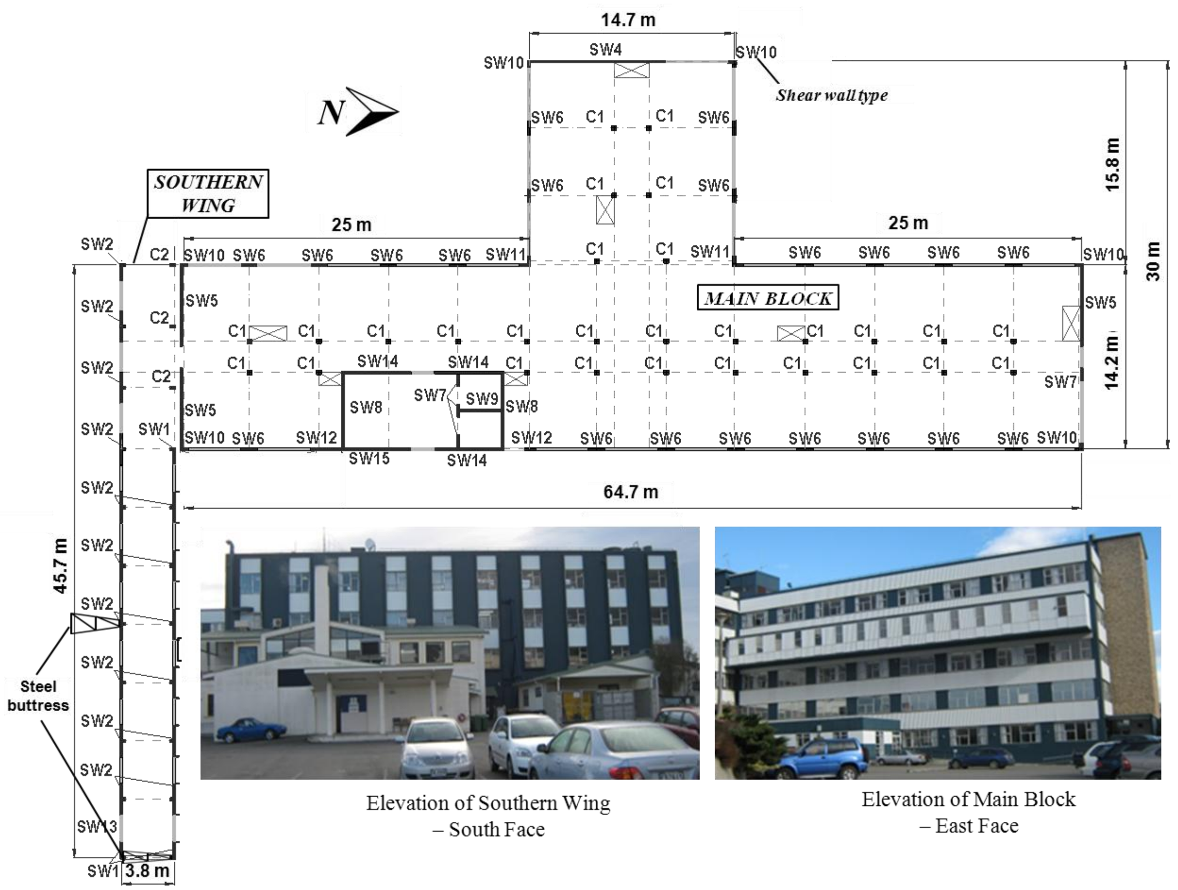

Figure 1. Main Block and Southern Buildings Layout

The Main Block is a tee shape in plan and comprises 5 storeys with an underground services tunnel and roof top plant rooms with approximate floor area of 1150 m2. The lateral load resisting system in both directions comprises a combination of reinforced concrete shear walls and reinforced concrete spandrel frames (coupling beams) around the perimeter of the building. The reinforced concrete shear walls are generally located around lift shafts, stairwells or at interfaces with other buildings. Reinforced concrete frames are located internally, but are assumed to act as gravity load resisting elements only.

The Southern Wing is a 4 storey structure that is relatively rectangular in plan and includes a basement services tunnel. It should be noted that an external steel buttress frame and internal steel bracing has been added to the Southern Wing, presumably to seismically strengthen it, when it was structurally tied to the Main Block, circa 1999. The structural system comprised of RC frames at regular centres with shear walls and concrete floors. In the longitudinal direction lateral loads are resisted by shear walls and a combination of slender concrete frames across with recently added steel braces.

For both buildings, the roof and floors comprise a reinforced concrete slab, which acts as a structural diaphragm to transfer lateral loads into the lateral load resisting elements. The foundations comprise shallow reinforced concrete pad and strip footings under the reinforced concrete shear walls, spandrel frames and internal frames. While the coupling beams within the perimeter reinforced concrete spandrel frames are not detailed with traditional seismic load resisting coupling beams (i.e. not diagonally reinforced), it is recognised that these deep beams will still contribute to the lateral strength and stiffness of the building. In Fig. 2., a blueprint of a typical reinforcement detailing for a wall and coupling beam extracted from the original structural drawings are shown (SCDHB, 1971). The likely design standard for The Main Block and Southern Wing buildings was NZSS 1900:1965 representing pre-seismic design practice.

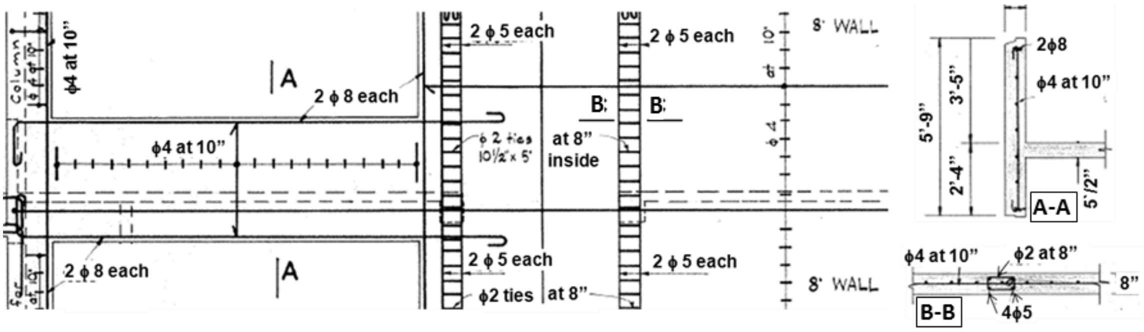

Figure 2. Reinforcement detail of a typical wall and coupling beam elements

SEISMIC ASSESSMENT ASSUMPTIONS

The key assumptions made during the assessment are given in below in Table 1.

Table 1 Summary of Seismic Assessment Assumptions

Item	Assumption	Comments
Importance Level	Importance level 4 (IL4) – Post disaster function facility	Design for earthquake with approximately 2500 years of return period. Based on emergency department and surgical wards being located within the building. Assessment assuming a reduced importance level of 3 (major structure) has also been carried out which is corresponding to approximately 1000 years of return period.

Table 1 Summary of Seismic Assessment Assumptions (cont`d)

Soil Type	C – shallow soil as per NZS1170.5 (2004)	Existing borelog and geological maps indicate a depth to rock of approximately 14m. Assumes no liquefaction or landslide threat exists on site.
Steel grade Concrete strength	f_y=300MPa f'_c=40MPa	Probable material strength based on information in original specification and material property performance factors in NZSEE guidelines.
Seismic Weight	Main Block = 43,000 kN Southern Wing = 6,500 kN	Seismic weight for the Main Block and Southern Wing. A portion of live load, L, has been included to determine the seismic weight.
Diaphragms	Rigid diaphragms	In-situ reinforced concrete slab (125 mm thick) is adequately tied to all lateral load resisting systems
Accidental Eccentricity / Torsional Effect	Inelastic torsion check is carried our based on Paulay (2001) and NZSEE (2006) recommendations.	Structural element adjusted to account for torsional eccentricity calculated by comparing location of centre of mass and centre of stiffness in each primary direction. Inelastic torsional eccentricity/effect is mitigated by minimising centre of mass and centre of strength of the system.

DISPLACEMENT-BASED SEISMIC ASSESSMENT

Key Steps of the Assessment

The main features of the methodology with emphasise on critical aspects is summarized in the following.

Step 1: Governing inelastic mechanism and post-elastic displacement properties (Fig. 3): The appropriate limit states (e.g., via stress-strain material constitutive properties) were adopted to calculate the probable axial-flexure and shear capacities of the critical sections to determine (i) non-ductile mechanism response based on the assessment of the hierarchy of strength (Akguzel et al., 2011) and (ii) member plastic rotation capacity from moment-curvature analysis. Conventional shear strength equations which include degradation of concrete shear contribution due to flexural ductility demand also used (Fig. 3c).

The capacities of the floor diaphragms and their connections to the structural walls are also checked to ensure that they can distribute the inertia forces adequately in accordance with assumptions made in the assessment.

Step 2: Calculate the post-elastic displacement (deformation) capacities (Fig. 4): For each wall elements in the CSB that are contributing to lateral resisting system, yield displacement, Δ_y, plastic hinge length, L_p, plastic curvature, ϕ_p, plastic displacement, Δ_p, at h_{eff} and ultimate displacement capacity Δ_u, can be calculated by the relationships given in Ref. 15.

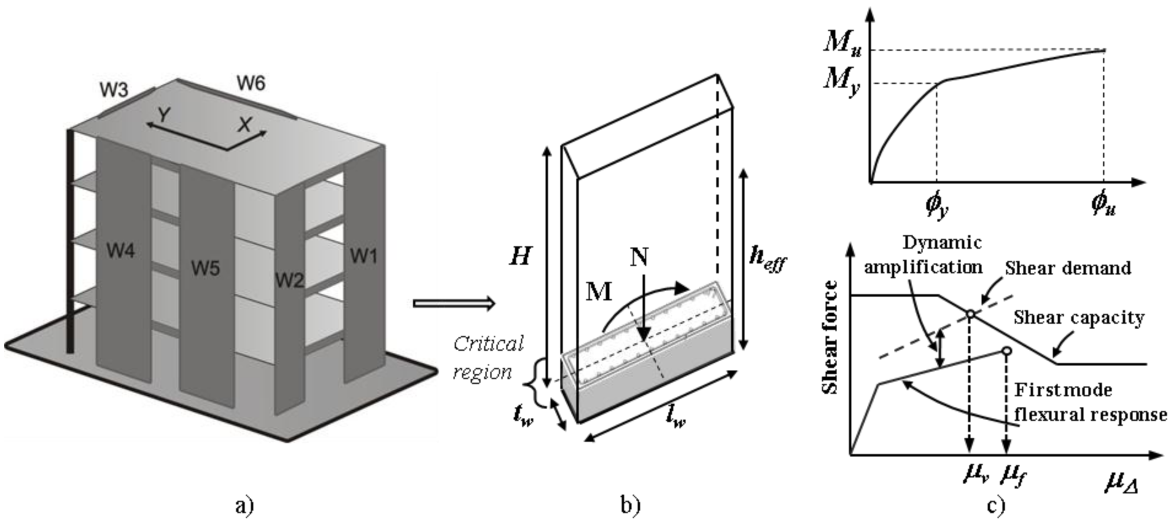

Figure 3. DBA procedure: a) Shear wall building; b) typical cantilever wall; c) flexural and shear capacities of critical section

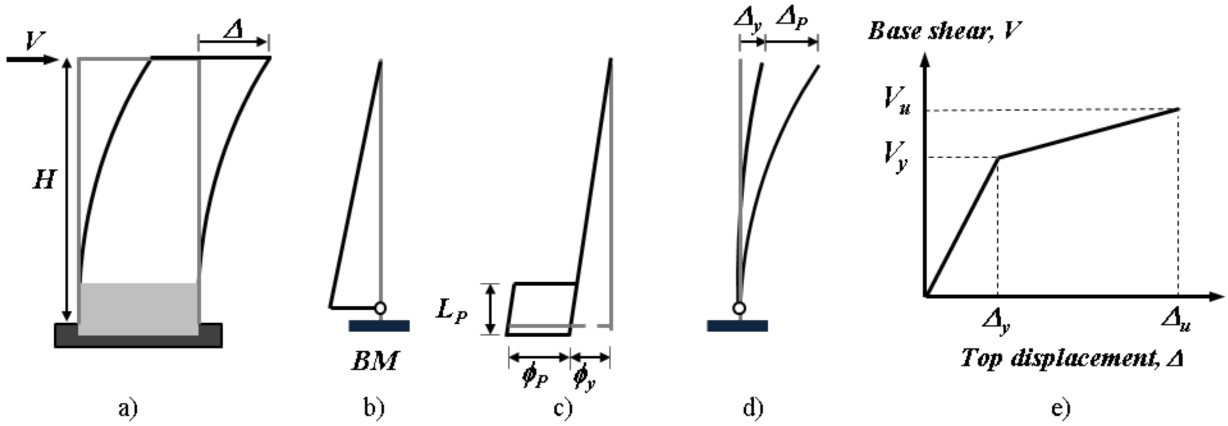

Figure 4. Shear wall element properties: a) deflected shape; b) bending moment diagram; c) plastic hinge length, yield and plastic rotation; d) yield displacement; e) bilinear hand pushover curve

Step 3: Estimate the lateral strength and achievable ductility of the system: Since the CSB comprised of multiple cantilever walls, the force-deflection response of all walls can be aggregated to provide the total response (V_{prob} vs Δ) from which the overall structural effective stiffness, k_{eff}, and global displacement ductility capacity μ_u can be calculated. This is reasonable for a system with rigid diaphragm assumption. A generic plot is shown in Fig. 5a.

In some cases, "critical structural weaknesses (CSWs)" may limit the displacement capacity of the overall system, hence these CSWs need to be identified. For instance, specific calculations are carried out to check the critical load path is compatible with the envisaged behaviour of the system. Specifically, potential CSWs to be checked for the CSB include: (1) horizontal diaphragm-to-wall capacity, (2) wall foundation capacity, (3) inelastic torsion stability from plan irregularity and (4) torsion amplification eccentricity for elements at the edge. The ratio of the ultimate displacement capacity and the yielding displacement gives the achievable ductility of the system, μ_{sys}. This value was checked against the material standard requirements. In case of reinforced concrete wall with non-ductile detailing such as the presence of plain bar straight splices, poorly confined boundary ends etc. a relatively conservative limit of $\mu_{sys} = 2$ can be used (Kam et al. 2013).

Step 4: Characterise the substitute structure properties: In order to characterize the nonlinear behaviour of an inelastic system with equivalent effective properties of effective stiffness, k_{eff}, and effective damping, in DBA the multi-degree-of-freedom (MDOF) system shall be reduced to single-degree-of-freedom (SDOF) system (Fig. 5c) by utilizing substitute-structure approach of Shibata and

Sozen (Shibata et al., 1976). This method uses a modified linear model of the structure and recognizes the effect of energy dissipation in the nonlinear range of response. The structural effective period, T_{eff}, is directly related to effective stiffness at maximum displacement and effective mass of the substitute system (Fig. 5a).

The identification of the probable inelastic deformation mechanism plays a crucial role in DBA by affecting the critical displacement capacity, Δ_{crc} and the expected inelastic displacement profile. Therefore, deformation of the structure is carefully determined by taking into account the findings of hierarchy of strength analysis. For CSB, the displacement profile of a cantilevered wall with flexural hinge at the base (Fig. 5b) is assumed. We note the determination of the displacement profile for an existing building depends on the assessment of the probable inelastic mechanism. This can be very challenging for complex structure with mixed inelastic mechanisms (Kam et al., 2013).

The equivalent viscous damping, ξ_{eq}, is determined based on the estimated ductility capacity and the expected mechanism using several different approaches. In this work, a base shear contribution weighed average approach to compute the achieved global energy dissipation, ξ_{sys}, is used (Ref. 15). Necessary substitute structure parameters are given in Fig. 5c. The substitute structure properties for CSB buildings are given in Table 2.

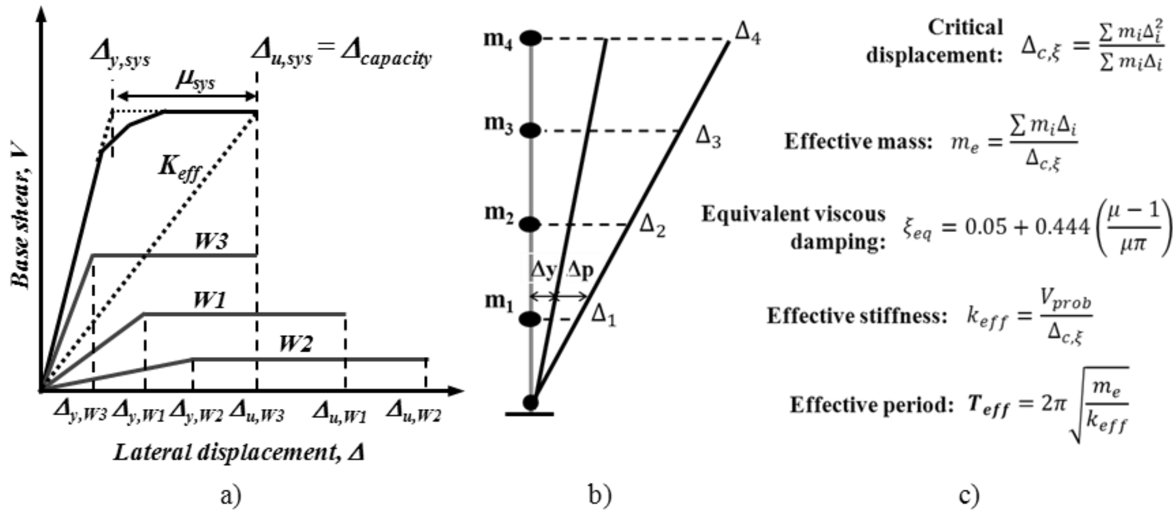

Figure 5. Structure properties: a) Schematic representation of push over capacity curves for individual wall elements of the in x-direction; b) displacement profile; c) substitute system properties (Priestley, 2007).

Table 2. Summary of substitute structure properties of the CSB buildings in each direction

Block	Direction	Teff	Building Base Shear	Limiting Disp Δc	Keff
		(sec)	(kN)	(mm)	(kN/m)
Main + Southern Block	N-S	1.75	2653	40.8	65025
Main + Southern Block	E-W	2.28	2150	64.6	33282

Step 5: Comparison of displacement capacity against demand: The structural displacement demand is read from the appropriate displacement spectra characterized by different levels of equivalent viscous damping and in terms of annual probability of exceedance. The structure spectral demand, $\Delta_{d,\xi}$, at effective height equals to the product of the site hazard spectral displacement, $\delta(T)$ which is code elastic displacement for 5% damping and the spectral reduction factor K_ξ (NZSEE, 2006). K_ξ accounts for the energy dissipation contribution from hysteretic and elastic viscous damping. For far field $K_\xi = (7 / (2+\xi_{eq}))^{0.5}$ is used (NZSEE, 2006). As the New Zealand Loading Standards NZS1170.5 does not yet incorporate an explicit displacement design spectrum, the pseudo-displacement spectra ordinates, S_d *(T)* can be generated by dividing the acceleration spectral ordinates, S_a (T) by ω^2, where $\omega = 2\pi/T$ = the angular frequency. In Fig. 6, spectral displacement demands for two annual probability of

exceedance are given for the main block and southern wing.

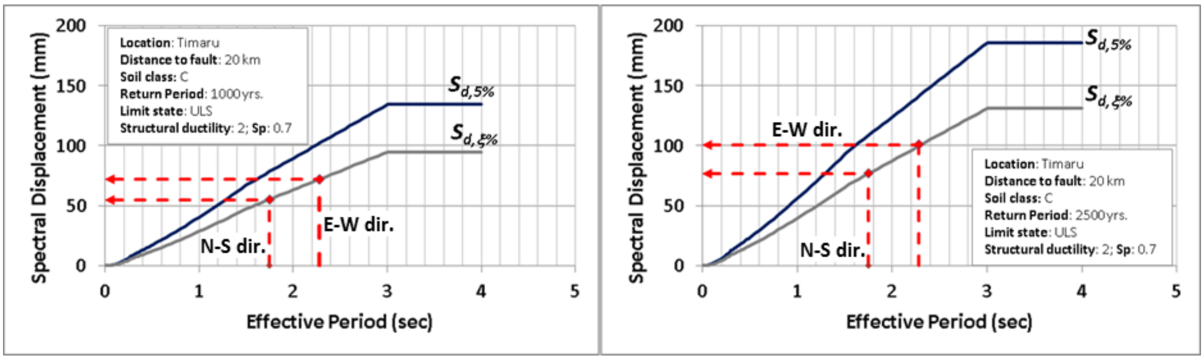

Figure 6. Structural displacement demand based on system equivalent damping ratio

Step 6: Percentage of new building standard %NBS at ULS: The seismic performance of the building is therefore the ratio between the lateral displacement capacity and the expected lateral displacement demand of the equivalent SDOF system: $\%NBS=\Delta_c/\Delta_{d,}$. It is apparent that cantilevered walls system with shear-critical inelastic mechanism, the %NBS can be directly calculated using the ratio of achievable base shear capacity and required base shear demand, as per force-based assessment procedure.

Coupling Beam Contribution

The resistance provided by the coupling beams in the CSB buildings are explicitly calculated and the associated displacement limits based on the available detailing. In accordance with the system post-elastic deformation profile, the deformation relationships between the overall wall rotation, θ_w, and the rotation at the ends of the coupling beams, θ_{cb}, can be conveniently employed to determine whether the coupling beams are yielding or not (Priestley 2007). Based on this, and hierarchy of strength within a coupling beam the lateral strength and stiffness contribution of the coupling beams is found.

The shear strength contributions due to coupling beams can be significantly high in old existing buildings with deep spandrel beams used as coupling beams. Nevertheless, note that failure of any or all of the coupling beams may not be indicative of the full capacity of the structure.

Inelastic Torsional Effects

Torsional irregularity is the most widely considered type of structural irregularity because of its seismic damage potential and complex nature. In this study, the analysis of torsional effects is performed according to Paulay (2001) and NZSEE (2006). The strength eccentricities in each direction are found from the centre of resistance, CV, and the centre of the mass, CM, of the system. The CV is determined using the probable strength of the wall elements. If the strength eccentricity exceeds 2.5% of the relevant lateral dimension of the plan, the probable strength of those elements which are responsible for the strength eccentricity is reduced to eliminate the strength eccentricity. Using the reduced hypothetical strength of the system, the displacement ductility demand is revised. This iterative procedure is based on the assumption that in the absence of strength eccentricity the response of the system may be considered to be governed primary by translatory displacements. In terms of ductile response, effects of elastic stiffness - eccentricity may be ignored. For buildings with complex configuration, it is suggested that the hand calculation approach is checked and confirmed with three-dimensional dynamic elastic analysis.

MEANING OF THE %NBS SCORE AND RELATIVE RISK

For this assessment, the building's Ultimate Limit State (ULS) earthquake resistance is compared with the current New Zealand Building Code (NZS3404:1997; NZS3101:2006) requirements for an equivalent new building constructed on the site. This is expressed as a percentage of New Building Standard (%NBS). In accordance with the Building Code, new buildings classified as 'major' (Importance Level (IL) 3) structures are required to have an ULS strength capable of resisting an earthquake with a return period of 1000 years and new buildings classified as 'post disaster' (IL4) structures capable of resisting an earthquake with a return period of 2500 years (NZS1170:2004). This is the 100%NBS level assumed in this study, as the building has been assessed to both IL3 and IL4 requirements.

The likely ultimate capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines (NZSEE, 2006), which provide guidance on calculating a modified Ultimate Limit State (ULS) capacity of the building for use when a Quantitative analysis is required. This ULS capacity can then be compared to the demand of a new building to determine a %NBS score.

Earthquake-Prone Buildings (EPBs) are defined in Section 122 of the New Zealand Building Act 2004 as buildings whose ultimate capacity will be exceeded in a moderate earthquake and would be likely to collapse causing injury or death or damage to any other property (NZ Building Code 2004). A building can be considered to be potentially Earthquake Prone, as defined by the New Zealand Building Act, when it has a score less than or equal to 33%NBS. If the score is greater than 33%NBS but less than 67%NBS, the building can be considered to be Earthquake Risk as defined by the NZSEE guideline document.

The following table by NZSEE provides a rough order grading system for existing buildings, as one way of interpreting the assessed %NBS building score. The Earthquake Prone (%NBS<33%) standard is low and it can be seen that Earthquake Prone buildings have more than 10 times the risk of collapse compared to an equivalent new building. For buildings that are potentially Earthquake Risk (67%>%NBS>33%), the risk of collapse may be considered 5 to 10 times greater than that of an equivalent new building. Broad descriptions of the life-safety risk can be assigned to these building Grades accordingly.

The New Zealand Society for Earthquake Engineering (which provides authoritative advice to the legislation makers, and should be considered to represent the consensus view of New Zealand structural engineers) classifies a building with an earthquake strength more than 67% of the new building standard as "Low Risk", and having "Acceptable (improvement may be desirable)" building structural performance.

Table 3 Relative Earthquake Risk

Building Grade	Designation		Percentage of New Building Strength (%NBS)	Approx. Risk Relative to a New Building	Risk Description
A+			>100	<1	low risk
A			80 to 100	1 to 2 times	low risk
B			67 to 80	2 t 5 times	low to medium risk
C	E/Q RISK		33 to 67	5 to 10 times	medium risk
D		E/Q PRONE	20 to 33	10 to 25 times	high risk
E			<20	more than 25 times	very high risk

ASSESSED SEISMIC PERFORMANCE

Table 4 below provides a summary of the expected seismic performance for shear wall elements, which were the structural elements of the building that governed the seismic performance. Assessed structural elements (such as diaphragms, foundations etc.) that have score greater than 100% have not been included in the table. The score shown reflects an assumed Importance Level of IL4 – 'post disaster' use; the scores shown in brackets reflect an assumed importance level of IL3 – 'major' structures. Additional behavioural observations revealed by the assessment are given in the following.

Table 4 Summary of Seismic Performance of Structural Systems

Building	Direction	Controlling Behaviour/Element	*%NBS*
Main Block and Southern Wing	North/South	Ductile failure of reinforced concrete walls in in-plane flexure	54 (75)
	East/West		64 (88)

The flexural failure of the walls is expected between the ground and first floor levels, with limited damage expected to the perimeter coupling beams above this level. It is noted that moderate localised damage could be expected to the reinforced concrete coupling beams between the reinforced concrete walls in the each direction under the above scenario, however this will not influence the expected seismic performance of the reinforced concrete walls.

Detailed displacement-based seismic assessment indicates the building is torsionally unbalanced due to irregular distribution of mass, stiffness and strength on plan. The presence of torsional irregularity of the Main Block is basically stemming from the reinforced concrete shear walls placed to unfavourable positions in the floor plan such as shear walls around the lift shafts placed near the edges of East-South direction of the plan. Therefore, the elements with highest displacement demand and most critical to the inelastic torsion check are the shear walls at the outermost West and North sides of the building.

Detailed capacity assessment of the Southern Wing has also shown that in the East/West direction main lateral bearing system consists of in-plane flexural strength of reinforced concrete shear walls. These elements contribute excessively to the torsional irregularity in the Main Block and Southern Wing buildings due to the increased eccentricity stemming from high stiffness to mass ratio.

In the North/South direction the lateral load resistance system of the Southern Wing maintained by out-of-plane flexural resistance of the existing reinforced concrete shear walls, internal and external steel buttress frames. It is important to note that, among these resistance mechanisms, assessment results has shown that, the external steel buttress frame is the most reliable in terms of lateral load carrying capacity under seismic action. Other elements exhibited low load carrying capacity due to unfavourable out-of-plane flexure failure mode forming at the base of the shear walls.

The reinforced concrete foundations in the East/West and North/South directions are likely to achieve approximately 100%NBS. The limiting mechanism in the foundation was assessed to be bearing failure of the underlying soil. The reinforced concrete floor slab is likely to achieve approximately 100%NBS in both the North/South and East/West directions with the limiting mechanism being dowel-friction action at the floor and wall interface.

OUTCOME

Our investigations outlined the following seismic retrofit options to address the structural weaknesses identified in the seismic assessment, recognizing the need to minimize business interruption, cost etc. These options are listed below:

Implementation additional of shear walls to the structural system to reduce torsional irregularity: Structures are generally exposed to torsional irregularities due to the seismic effects. Plan geometry of structures should be symmetrical to reduce the torsional irregularity coefficient. In order to reduce these effects shear walls can be added on the outer North and West sides of the Main Block building.

Undertake selective weakening of the perimeter concrete spandrel/coupling beams: Selective weakening of the reinforced concrete coupling beams (to the perimeter concrete frames) could be carried out to minimise the likelihood of localised damage to the coupling beams during a significant earthquake. This would involve making a vertical cut to the coupling beams at the reinforced concrete wall interface. This selective weakening would not reduce the expected seismic performance of the building.

Modify stairs to increase their performance by introducing sliding joints: Modifications could be made to the stairs and their supporting elements allowing the stairs to slide during a seismic event. This would reduce the likelihood of damage to the stairs (although some damage is still possible), and could help to protect escape paths for occupants.

Strengthen reinforced concrete shear walls and gravity columns using carbon fibre wrapping: To enhance the displacement ductility capacity and the shear capacity of the walls and columns. For the critical gravity-load path, the additional ductility and shear capacity provides further seismic resilience to the building.

Further consider the effect that the Southern Wing has on the building performance: The Southern Wing is shown to contribute significantly to the torsional irregularity and hence the seismic performance of the building as a whole. It is also noted that the Southern Wing is only 3.8m wide and is not considered functionally efficient space (it used to be a 4 storey link corridor to a building that was demolished some years ago). Therefore an option is to demolish the Southern Wing Block to reduce the seismic demand on the Main Block. The area can be subsequently redeveloped for a more purpose—built facility.

Consider serviceability limit state performance of the building: Given that the CSB is currently intended to provide post disaster care to the community, it may be desirable to determine the return period of earthquake that the onset of structural damage would occur. This could be done by determining the %NBS (and corresponding return period) assuming a structural ductility of the system of $\mu_{sys} = 1$.

These items were used as the basis for determining subsequent concept seismic retrofit schemes, which was used to assess the economic viability of a seismic retrofit (and associated implications on other building elements) versus building replacement.

CONCLUSIONS

As a result of the Canterbury Earthquakes, building owners throughout New Zealand have become more aware of seismic risk and vulnerability of their buildings. In this paper a case study is presented on the Displacement Based Assessment (DBA) of the existing Clinical Services Building (CSB) at Timaru Hospital. The seismic risk evaluation of existing structures is complex and significantly different from designing a new structure.

Many of the principles outlined above are directly applicable to buildings where the principal form of lateral resistance is structural walls. The CSB seismic assessment has shown that DBA can be applied in real life to structures that are not simple in geometry like many ‘model structures’ presented in procedures or examples in relevant literature. However the designer should pay a particular attention to identify the possible limitations on the displacement capacity imposed by critical structural weaknesses (CSWs), and also the effect of torsional irregularity in the building structure, the effect of connecting structures on seismic performance and ensure that rational load paths are assumed.

In our opinion, the DBA procedure, whilst simplistic, provides a rational methodology to form a good understanding of the probable inelastic mechanism and the critical load path, and therefore a better seismic assessment outcome. Preliminary structural calculations can be carried out for a particular building form and allows the engineer to relatively quickly determine the critical elements of the building in terms of seismic performance. This can allow the engineer to identify more targeted and accurate seismic retrofit options to the building owner, allowing them to make more informed and sensible decisions when considering seismic retrofit of the building and other follow on effects.

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