New Generation of Structural Systems for Earthquake Resistance

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ABSTRACT: The current seismic design philosophy for reinforced concrete structures in New Zealand is based on the concept that it is generally uneconomical to design a building to ensure elastic response in a large earthquake. An implication of this concept is that structural damage is accepted, as long as collapse is prevented in a major earthquake. For this reason standards allow the use of design forces that are generally smaller than those required for elastic response. This requires the critical regions of the structure to be adequately designed for ductility and for energy dissipation. In New Zealand, ductile design has been achieved since 1976 by selecting a suitable mechanism of plastic deformation and ensuring, through capacity design, that the mechanism can develop and be maintained.

Experience gained from earthquakes abroad indicates that the cost of repair of buildings designed for ductile response has not been insignificant. This prompts the need to develop structural systems that have large displacement capacity and perform essentially damage-free, even when subjected to large earthquakes. This paper covers design aspects of a new generation of structural systems aimed at minimizing damage. The paper briefly discusses the results of a test programme on precast/post-tensioned structural wall systems being conducted at the University of Canterbury.

1 INTRODUCTION

The impact and cost of the consequences of damage caused by earthquakes worldwide during the past twelve years has raised the question of whether the current building seismic design philosophy is satisfying the needs of modern society. The advance in technology has meant that very often the cost of equipment and stock kept within the building are generally more expensive than that of the structure itself. As a result, risk to property tends to increase with modernization. Most seismic design standards are based on a life-prevention philosophy where building structural and non-structural damage is accepted providing that collapse is avoided. No other economic parameters, such as the cost of damage to equipment and stored goods and the cost associated with the loss of operation following a moderate/strong earthquake, are currently accounted for in the design process.

In New Zealand, ductile design has been common practice since 1976 when capacity design principles were formally codified and adopted. In capacity design a suitable mechanism of plastic deformation is chosen and the critical regions are detailed for ductility. Other regions in the structure are made with sufficient strength to ensure the mechanism can develop and be maintained. Whilst the concept of designing a structure to ensure the development of a suitable mechanism of plastic deformation is a very effective mean of avoiding collapse, it has an important shortcoming. The primary structural system is built with regions that will be sacrificed in moderate and strong earthquakes and may require from minor to expensive repair work or even demolition.

This paper covers design aspects of a new generation of structural systems whose response is aimed at minimizing damage. The paper briefly discusses the results of a test programme on precast/post-tensioned structural wall systems being conducted at the University of Canterbury.

2 WHY DO NEW SYSTEMS ARE NEEDED?

The cost associated with the loss of business operation, damage to equipment and structural damage following a moderately strong earthquake can be significant to modern society, particularly in those centers of advanced technology. Such cost is often comparable, if not greater, than the cost of the structure. There are many ways in which seismic design can be performed to control damage and to minimize the loss of business operation. One of them is through seismic isolation (Skinner et al., 1993). In seismic isolation, special devices are placed in specific locations in the structure with the main aim to dissipate energy and to reduce the dynamic response. Such devices can be usually replaced with minor disturbance in case their limiting performance is reached.   An alternative structural system, which with future development could be designed with replaceable energy dissipation devices, is described in detail in this paper.

In 1993, Priestley and Tao proposed the use of lateral force resisting systems built incorporating unbonded prestressing for use in seismically prone areas. This proposal was supported by a series of non-linear dynamic time-history analyses that showed the viability of such systems.   Priestley and Tao pointed out that a main advantage of these systems is the lack of residual drift following a strong earthquake. This advantage could easily offset the greater lateral displacement demand obtained for such systems when compared with traditional systems.   Since then, several systems have been proposed and tested as part of the co-ordinated four-phase PRESSS research programme in United States. This programme recently finished with the testing of a 60% scale five-storey building (Nakaki et al., 1999; Priestley et al., 1999). Detailed research work has also been carried out in United States to evaluate the response of systems incorporating unbonded prestressing in bridge piers (Mander and Ching-Tung, 1997).

The basic behaviour of a system incorporating unbonded prestressing is illustrated in Fig. 1. The structural element, in this case a precast concrete wall jointed at the base, is prestressed with partially unbonded tendons. The tendons are generally prestressed to stress levels lower than those used in conventional prestressed systems. The unbonded length of the tendons is proportioned to ensure that the limit of proportionality is not reached as a result of the elongation caused by the opening of the gap at the wall base during the largest expected lateral displacement demand. The opening of a large gap at the base of the wall implies that large compressive strains are expected to arise at the corner of the wall. This usually requires the use of confinement to enable the concrete to develop such strains without crushing. The opening of the gap at the base of the wall shown in Fig. 1 plays a fundamental role in the lateral force-displacement response of the system, see Fig. 2 (a). During small amplitude displacements, the joint at the wall-foundation beam remains closed and most deformations take place within the wall panel and the foundation structure. A gap opens as soon as decompression is reached at one end.   A marked change in the tangential stiffness occurs when the neutral axis depth, measured from the extreme compressive fibre, migrates to within 50-25% of the length of the wall.   This change of stiffness results in the apparent "yield" point P shown in Fig. 2 (a). Imposed displacements beyond point P result in some increase in the restoring force. This is because the stiffness of the wall is significantly reduced as a result of the development of the large gap at the base of the wall. Unloading takes place through essentially the same loading path. This implies that the response of the wall is non-linear elastic. The main two advantages of this response are (i) the lack of structural damage, and, (ii) the lack of residual displacements. A disadvantage of the system shown in Fig. 1 is the lack of energy dissipation capacity. The lack of energy dissipation capacity can significantly increase the demand on the system, usually by increasing the lateral displacements, the shear forces and the floor accelerations.

Energy dissipation capacity can be incorporated into the system by several means. For example, in the case of the cantilever precast wall shown in Fig. 1, energy dissipation can take place if mild steel bars, with a milled segment in the form of a "dog-bone", are cast in the foundation and then grouted into the wall, see Fig. 3. These bars look just like the commonly used starter bars. Energy dissipation takes place through extensive yielding in tension and compression in the tensile strain domain within the milled portion of the bar only. Note that buckling cannot occur as the milled portion of the bar is surrounded by concrete in the elastic foundation beam. The diameter of the milled segment is selected such that closing of the gap at the horizontal connection is ensured upon unloading. Thus, the prestressing force after losses, in

addition to the gravity loads, must be sufficient to ram the "dog-bones" to nominally zero strain upon unloading. As consequence of this action is that the hysteretic response of the overall system is characterised by loops showing energy dissipation and no residual lateral displacements, see Fig. 2 (b).

3 DESIGN ASPECTS FOR JOINTED CANTILEVER WALLS

Rahman and Restrepo (2000) developed a series of guidelines for the design of cantilever walls prestressed with partially unbonded tendons incorporating the energy dissipation devices described in previous section. A summary of these guidelines is presented below.

To ensure the development of rocking in walls that are seated on the foundation beam but are not slotted into it, the aspect ratio of a wall, defined as the ratio between the wall height, H, and its length, L_w, should be such that

$$\frac{H}{L_w} = \frac{\omega_v}{2\mu_f}$$

(1)

where ω_v is a shear force dynamic magnification factor that accounts for the increase shear resulting from the higher modes of response and μ_f is the coefficient of friction between the contacting wall and foundation surfaces. As it will be indicated in Section 4, the shear force magnification factor ω_v can be made equal to the value recommended by the New Zealand Concrete Structures Standard for monolithic walls (NZS 3101, 1995).

As a wall is subjected to cyclic reversals under sesimic loading conditions, the force in the tendons will increase with the amplitude of the lateral displacement. In order to delay the onset of yielding the optimum location for the tendons at midlength of the wall. The maximum stress after losses in the post-tensioning tendons, f_{psi}, is determined from the expected stress increase due to kinematics of the rocking wall. The critical tendons is that furthest from the neutral axis depth at the drift θ_u at the ultimate limit state, see Fig. 1. Stress f_{psi} is given by,

$$f_{psi} = f_{lp} - \frac{d_{ps} - c}{L_{ps}} E_{ps} \theta_u$$

(2)

where c is the position of the neutral axis depth measured from the extreme fibre in compression, E_{ps} is the elastic modulus of the tendon, f_{lp} is the stress in the tendon at the limit of proportionaly, d_{ps} is the distance of the tendon furthest from the extreme compressive fibre to this fibre and L_{ps} is the tendon's unbonded length.

The lengths of the non-milled segments at either end of the milled section of the bar should be such that the full tensile strength of the bar may be developed while the non-milled segments remain elastic. The diameter of the milled segment is selected such that closing of the gap at the horizontal connection is ensured before and after an earthquake. Consequently, the prestressing force after losses, in addition to the gravity loads, must be sufficient to push the milled section of the bar to nominally zero strain upon unloading. To meet this design requirement, the total area A_{sd} of the milled segments acting as energy dissiaptors should satisfy,

$$A_{sd} \leq \frac{A_{sp} f_{psi} + N^*}{1.5 f_y}$$

(3)

where N^* is the axial compression force acting at the base of the wall, A_{sp} is the total area of prestressing steel reinforcement and f_y is the lower 5% characteristic yield strength of the energy dissipation devices.

The design procedure proposed by Rahman and Restrepo (2000) for determining f_{psi} and A_{sd} is iterative and requires the determination of the neutral axis depth c. An approximate expression for c at the ultimate limit state is given by,

$$c = \frac{N^* + 0.9f_{lp}A_{sp} + 1.5f_y A_{sd} + 1.4f'_c b_e c_c}{1.4f'_c b_e} \qquad (4)$$

where f'_c is the concrete cylinder compressive strength, b_e is the wall width and c_c is the concrete cover to the longitudinal reinforcing bars confining the wall edges.

For determining the length L_e of the milled segment of the energy dissipator see Fig. 3, Rahman and Restrepo (2000) proposed the following expression,

$$L_e \geq 1.5(d_{ed} - c)\frac{\theta_u}{\varepsilon_{su}} \qquad (5)$$

where d_{ed} is the distance from the dissipator furthest from the extreme compressive fibre to this fibre and ε_{su} is the uniform strain of the reinforcing steel used for manufacturing the dissipator. Equation 5 is based on the assumption that the axial strain in the milled section of the energy dissipator would not exceed $2\varepsilon_{su}/3$. They pointed out that the energy dissipator needs to be properly anchored in the foundation beam and in the wall panel to ensure the development of tensile and compressive stresses in the milled section equal in magnitude the ultimate tensile strength.

The nominal flexural strength M_n at the base of a cantilever wall at the level at which the limit of proportionality of the tendons is attained can be approximated by the following equation,

$$M_n = (N^* + f_{lp} A_{sp} + 1.5f_y A_{sd}) (L_w - c - c_c)/2 \qquad (6)$$

where L_w is the wall length. Equation 6 assumes that, when the limit of proportionality of the tendons is reached, the axial strain in the energy dissipators is greater than $\varepsilon_{su}/2$.

4 DYNAMIC RESPONSE

Figure 4 shows the bending moment and shear force envelopes obtained from non-linear time history analyses for a twelve-storey cantilever wall building subjected to a synthetic record matching the design response spectra for intermediate soil conditions in Wellington. Analyses were conducted on models representing conventional monolithic construction and jointed construction incorporating energy dissipators as described in Section 3. To enable a comparison of the dynamic response, the backbone moment-curvature response employed in the analyses was identical for the monolithic and jointed wall models. The response of the monolithic wall was modeled using a Takeda hysteresis rule whereas an Origin-centered rule was used to represent the response of the jointed wall. The monolithic wall was designed for ductile response following the recommendations of the Loadings and Concrete Structures Standards (NZS 4203, 1992; NZS 3101, 1995). The jointed wall was designed to match the capacity of the monolithic wall. The axial for the walls was small and was ignored in the analyses. Constant 5% damping ratio was assigned to all modes of response. Figure 4 (a) shows that, for the particular case studied, the maximum bending moment demand in the jointed wall develops above the base where rocking occurs. In the case of the monolithic wall, the maximum bending moment develops at the base. Yielding in this wall spreads upwards up to about fifty percent of the wall's height, a value greater than currently assumed in design. In the upper half of the walls the bending moment envelopes are nearly identical for both systems. Figure 4 (b) compares the shear force envelopes obtained from the analysis for both systems with the design

envelope derived from the recommendations given by the Concrete Structures Standard (NZS 3101, 1995). The shear force envelopes are very similar for both systems and are, for most of the height of the walls enveloped by the design envelope obtained from the standard. This finding suggests that the shear force magnification factor, ω_v, recommended for the design of monolithic walls in the Concrete Structures Standards is equally applicable to jointed walls incorporating energy dissipators.

5 EXPERIMENTAL WORK

Five walls have been built and tested as part of an ongoing research programme at the University of Canterbury. Full details of the experimental work can be found elsewhere (Holden, 2001; Rahman and Restrepo, 2000). Figure 5 shows the main features of each test unit. Unit 1R tested by Rahman and Restrepo was post-tensioned only. Units 2R and 3R incorporated energy dissipation devices in the way of dog-bone bars. Unit 3R was tested under constant axial load. Unit 1H tested by Holden was precast and was designed to emulate a ductile cast-in-place concrete wall following the requirements of the Concrete Structures Standard, (NZS 3101, 1995). The wall was seated inside a slot built into the foundation beam and was grouted afterwards. This wall was designed for the same capacity as Units 2H and 3R. Problems with scaling down the reinforcing bars while satisfying the minimum spacing requirements meant that the wall was about 30% stronger than the post-tensioned units. Units 2H and 3R were similar, except that the wall in Unit 2H was cast using steel-fibre reinforced concrete and was post-tensioned using carbon fibre tendons. The reinforcing detailing was eased in all the jointed walls as a plastic hinge, resulting in structural damage at the base of the walls was not expected to occur.

Figure 6 shows the general geometry and loading arrangement. The test units represented a one-half scale of a 250 mm thick wall used in a prototype four-storey building. The units were subjected to quasi-static reversed cyclic loading to increasing drift levels. The lateral force was applied by a single double acting hydraulic actuator at 3.75 m from the base of the walls. Figure 7 shows the lateral force – lateral displacement response for Units 1H and Unit 3R. Both units showed satisfactory behaviour as far as the hysteretic response is concerned. Unit 1H failed by fracturing the longitudinal reinforcing bars during a cycle towards 2.5% drift. In this unit several residual cracks of 1 mm and 2.2 mm in width were observed after the unloading from cycles to 1% drift. In contrast, Unit 3R reached 4% drift without failure and with very limited damage. The gap at the base of the wall always closed upon unloading. Only hairline cracks were observed to develop in this unit but cracks closed upon unloading. Figure 8 shows the extent of damage in each unit at a drift of 3%.

A comparison of the hysteretic response of the two units shows that Unit 1H has the ability to dissipate more energy than Unit 3R. This is due to the "fatness" of the hysteresis loops. For example during the first cycle to 2% drift the equivalent viscous damping obtained for Unit 1H 24 % whereas Unit 3R attained a ratio of 11.5%. While a higher damping ratio may be a desirable feature to control the dynamic response of a system, the lack of structural damage and lack of residual drifts observed in jointed walls will easily overshadow the smaller inherited damping.

6 CONCLUSIONS

This paper covered the design aspects of a new generation of structural systems whose response is aimed at minimizing damage. The system comprises jointed precast concrete walls that are prestressed with partially unbonded tendons. The paper briefly discussed the results of a test programme on precast/post-tensioned structural wall systems being conducted at the University of Canterbury.

The following conclusions can be drawn:

1. The impact and cost of the consequences of damage caused by earthquakes worldwide during the past twelve years has raised the question of whether the current building seismic design

philosophy is satisfying the needs of modern society. The advance in technology has meant that very often the cost of equipment and stock kept within the building are generally more expensive than that of the structure itself. A consequence of this is that the level of structural damage observed to occur in buildings in earthquake affected regions may no longer satisfy the needs of modern society. The level of damage prompts the need for the development of systems that could be considered to be inert to input ground motion. One such system comprises jointed cantilever precast concrete walls that are prestressed with partially unbonded tendons. Such system can be built with energy dissipation devices in the way of starter bars with a milled section placed at the wall-foundation structure connection.

2. Jointed walls have the main advantage over conventional ductile reinforced concrete wall systems in that no residual drifts or structural damage is expected to occur after a major earthquake. These two features easily offset the fact that the energy dissipation capacity of the jointed system is less than of that expected from a well detailed ductile reinforced concrete system.

3. The basic mechanics and design parameters for the jointed walls described in the paper were outlined in the paper.

4. Dynamic non-linear time history analyses on multi-storey wall systems have shown that the shear force envelope is very similar to that found for conventional monolithic wall systems. This allows the shear force magnification factor given in the Concrete Structures Standard (NZS 3101:1995) to be used for the design of jointed walls.

5. A comparison of the test results of a precast concrete cantilever wall designed to emulate monolithic behaviour and a jointed wall conclusively showed the main advantages of the system proposed. The monolithic wall presented residual cracks between 1 mm and 2.2 mm in width after unloading from a cycle to 1%. Buckling of the longitudinal reinforcement preceded fracture in a cycle to 2.5% drift. In contrast, the jointed wall developed a large gap at the wall base. This gap always closed upon unloading. Hairline cracks also closed upon unloading. The wall reached 2.5% drift with only cosmetic damage. The jointed unit attained cycles to 4% drift with no strength degradation.

REFERENCES:

Holden, T.J., *A Comparison of The Seismic Performance of Precast Wall Construction: Emulation And Hybrid Approaches*, ME Report, Department of Civil Engineering, University of Canterbury, Christchurch, 2001. (in preparation).
Mander, J. and Chin-Tung, *Seismic Resistance of Bridge Piers Based on Damage Avoidance Design*, NCEER Report 97-14, State University of New York, Buffalo, , 1997.
NZS 3101:1995, *Concrete Structures Standard*, Standards New Zealand, 1995.
NZS 4203:1992, *Code of Practice for General Structural Design and Design Loadings for Buildings*, Standards New Zealand, 1992.
Otani, S., 1997, Development of Performance-Based Design Methodology in Japan, *Seismic*

Design Methodologies for the Next Generation of Codes (eds: Fajfar and Krawinkler), AA. Balkema, pp. 59-67.

Priestley, M.J.N. and Tao, J.R.T., 1993, Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons, *PCI Journal*, 38 (1). 58-69.

Priestley, M.J.N., Sritharan, S., Conley, J.R. and Pampanin, S., 1999, Preliminary Results and Conclusions From the PRESSS Five-Story Precast Concrete Test Building, *PCI Journal*, 44 (6). 42-67.

Rahman, A. and Restrepo, J.I., Earthquake Resistant Precast Concrete Buildings: Seismic Performance of Cantilever Walls Prestressed using Unbonded Tendons, Research Report 2000- 5, Department of Civil Engineering, University of Canterbury, Christchurch, 2000.

SEAOC, "Vision 2000 – A Framework for Performance-based Design", Vols. I, II and II, Structural Engineers of California, Sacramento, 1995.

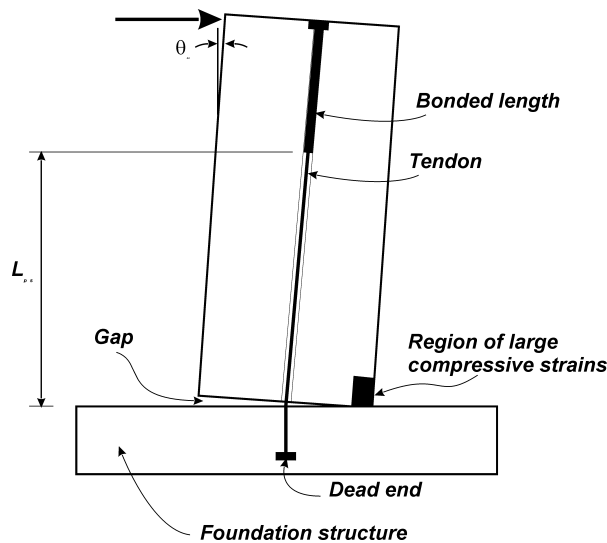Skinner, R.I., Robinson, W.H. and McVerry, G.H., *An Introduction to Seismic Isolation*, John Wiley & Sons, 1993.

Figure 1. Main features of walls prestressed with partially unbonded tendons.

(a) System prestressed with unbonded tendons

(b) System prestressed with unbonded tendons and incorporating energy dissipation devices
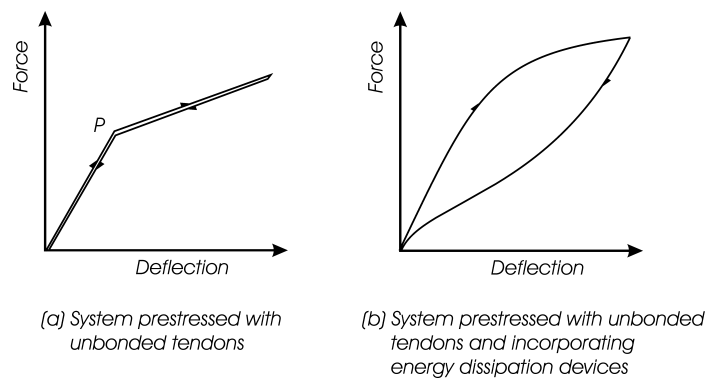
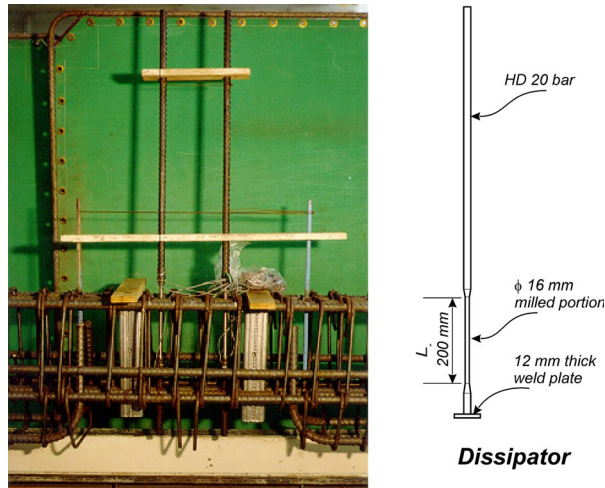Figure 2. Lateral force-lateral displacement response of walls prestressed with partially unbonded tendons.

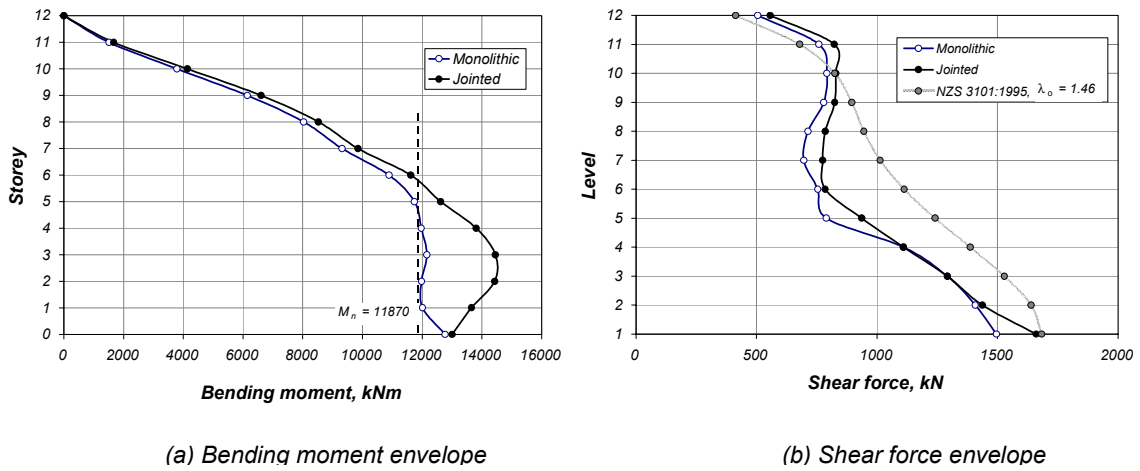Figure 3. Energy dissipators installed in Unit 3R tested by Rahman and Restrepo.

(a) Bending moment envelope

(b) Shear force envelope

Figure 4. Comparison between the dynamic response of monolithic and jointed walls in a twelve-storey building.

Researcher	Unit	System Type		Axial Load (kN)	Energy Dissipators	Other
		"Monolithic"	Jointed			
Rahman & Restrepo	1R		●	16		
	2R		●	16	●	
	3R		●	216	●	
Holden	1H	●		216		
	2H		●	216	●	Pre-stressed with carbon tendons Steel-fibre reinforced concrete

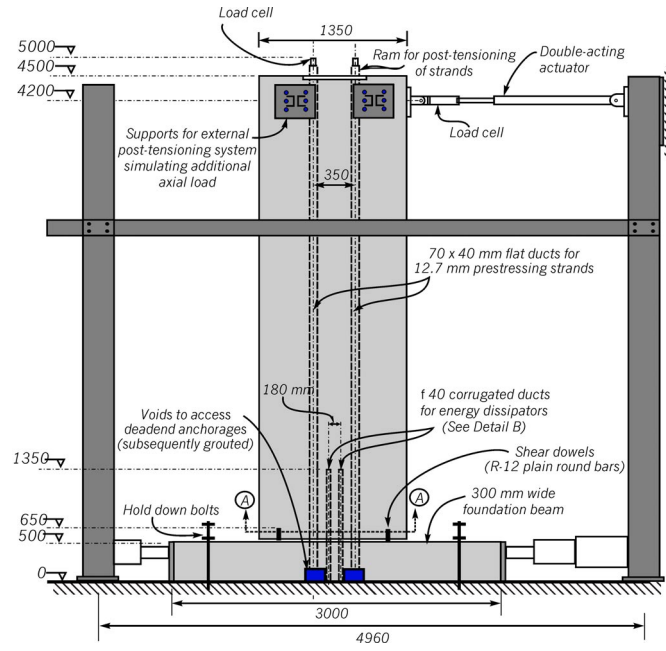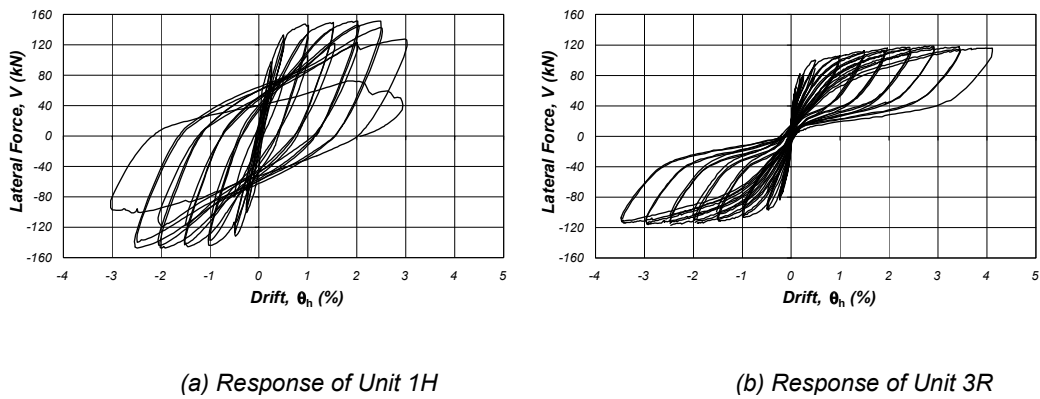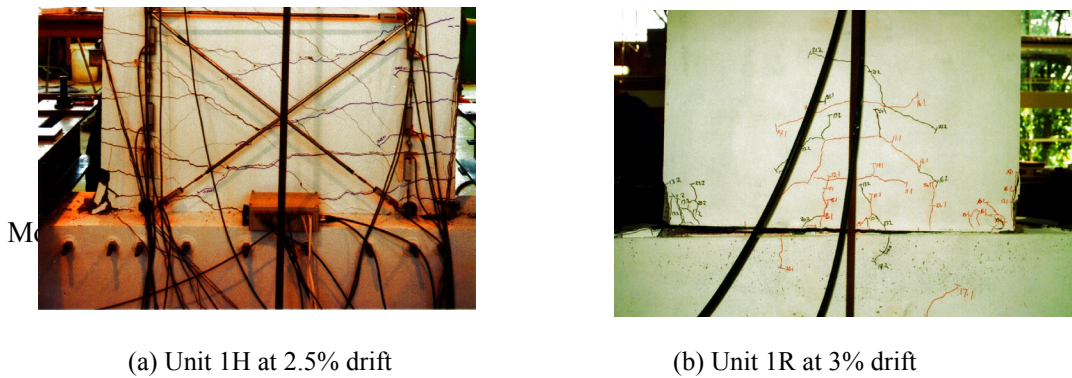Figure 5. Overview of the experimental work conducted at the University of Canterbury.

Figure 6. General geometry and load arrangement.

(a) Response of Unit 1H   *(b) Response of Unit 3R*

Figure 7. Lateral force-lateral displacement response of Units 1H and 3R.

(a) Unit 1H at 2.5% drift   (b) Unit 1R at 3% drift

Figure 8. View of Units 1H and 3R at near end of the tests.

7 RETURN TO INDEX