Design, construction and dynamic testing of a post-tensioned precast reinforced concrete frame building with rocking beam-column connections and ADAS elements

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ABSTRACT: Reinforced concrete structures permitted to rock on their foundations and provide recoverable rotations at the beam-column interfaces offer significant advantages over conventional ductile detailing. A jointed construction philosophy can be applied whereby structural elements are connected with unbonded steel tendons. Supplemental damping is provided by replaceable flexural steel components designed to deform inelastically. A multi-storey test building of one quarter scale has been constructed and tested on a uniaxial earthquake simulator at the University of Canterbury. A computer model has been developed and a set of preliminary design procedures proposed.

1 INTRODUCTION

Post-tensioned reinforced concrete structures permitted to rock on their foundations and provide recoverable rotations at the beam-column interfaces offer several advantages over conventional monolithic and emulative reinforced concrete detailing in terms of seismic performance. These benefits include reduced displacements, diminished base shear demands and negligible residual deformations. Seismic energy dissipation can be achieved during the earthquake event through replaceable dissipation devices without the aftermath of damage to structural elements.

Much of the research surrounding structures able to rock on their foundations stems from early work by Housner (1956, 1963) and his assessment of the performance of seemingly unstable structures in the Arvin-Tehachapi earthquake of 1952 and the Chilean earthquakes of 1960. Housner found that by permitting a structure to rock on its foundations a degree of protection from an earthquake. He went on to develop an expression for the decay of motion of a simple rocking block model which was later amended by McManus (1980).

The viability of post-tensioned precast beam-column connections with a capacity to rock at the joint interface as an alternative to traditional emulative construction practices has been examined by a number of researchers (Priestley and Tao, 1993; Cheok and Lew, 1993; Priestley and McRae, 1996; Stanton *et al.*,. 1997; El-Sheikh, *et al.*, 1999; Davies, 2003; Arnold, 2004). Recently the Joint US-Japan PRESSS program undertaken at the University of California at San Diego has sought to develop design criteria for buildings incorporating a number of these types of connections based on large scale pseudo-dynamic experiments (Priestley, et al., 1999).

This paper seeks to summarise the results of an investigation into the dynamic response of a post-tensioned reinforced concrete frame building with rocking connections. A ¼ scale model was designed, constructed and tested at the University of Canterbury. This structure consisted of a one bay

by one bay configuration over two levels as shown in Figure 1. Symmetrical arrangement of the columns about the central axis of the structure was done to facilitate testing in both the transverse and longitudinal directions without the need for any modification to the foundations. In accordance with the requirements of constant acceleration similitude, additional mass was provided at each floor level.

Supplementary damping and lateral resistance was afforded to the structure through added damping and stiffness (ADAS) elements installed across the beam-column joints. A series of tests were undertaken to appraise the dynamic properties of the building, both with and without ADAS elements. Using synthetic base motions as well as four real earthquake acceleration records the dynamic performance of the building was examined and the design process appraised. At this time, testing in one direction only has been completed.

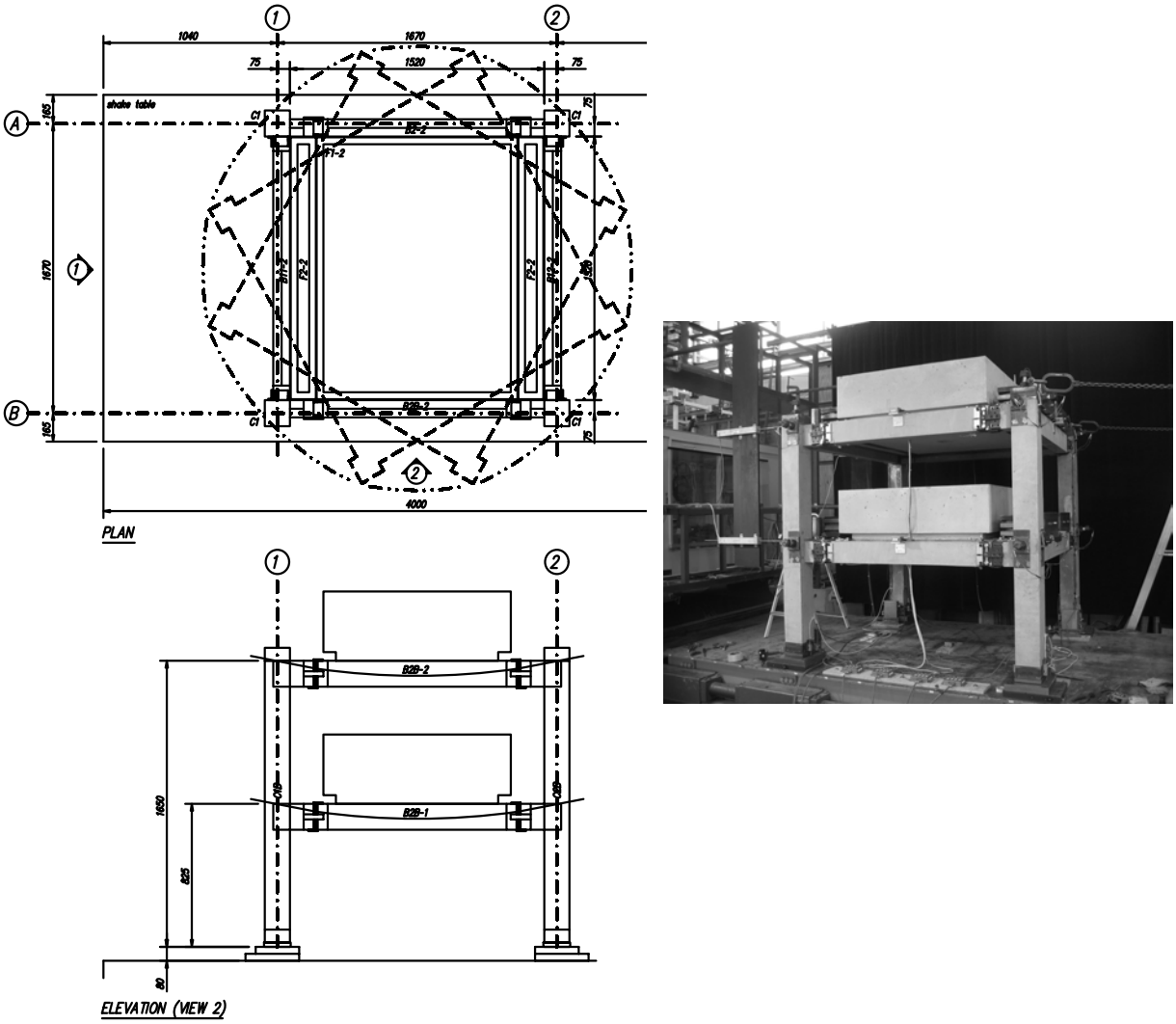


Figure 1 Model test structure

2 DESIGN & CONSTRUCTION

2.1 Design Approach

The design philosophy adopted for the test frame structure was one of damage control. This holistic approach is encapsulated by its definition as design and distribution of "a combination of several structural systems and energy transformation devices that are integrated in such a way as to restrict damage to a specific set of structural elements that can be readily repaired." (Conner *et al.*, 1997).

Conventional ductile reinforced concrete design amalgamates both the stiffness and energy absorption

characteristics of the constituent parts into a single structural system. Consequently, structural components are obliged to undergo marked inelastic behaviour in response to severe ground motions with such severe damage that repair is often uneconomical. In contrast, a damage control designed structure uncouples the resistance mechanisms into two independent systems. The first system offers resistance to vertical and lateral loads as well as providing a measure of damping. Detailing of members consistent with the paradigm of Damage Avoidance Design (Mander and Cheng, 1997) ensures that inelastic deformations between rocking connections are prevented and material distress in regions of concentrated force is mitigated. The second system provides supplemental lateral resistance in the form of devices with added stiffness partnered with energy dissipation characteristics.

Much of the previous research into precast frame buildings with rocking connections has been devoted to configurations with post-tensioning positioned symmetrically across the beam-column rocking interface. Although this arrangement ensures that the moment resistance and hence capacity is identical at both ends of the beam, it essentially overlooks the enhanced serviceability performance that may be gleaned from the presence of prestressing. By draping the tendon in a parabolic profile, gravity loads are able to be balanced resulting in larger spans and more slender members with a consequential reduction in cost. For the test building the tendon profile was established between the geometrically defined kern points of the beam sections thereby eliminating the prospect of tensile stresses in the beam independent of prestressing forces.

A floating floor system was used to isolate the floors from the adjacent perimeter frame. Each floor comprised of three sections linked by shear connections that overlapped the adjacent units and perimeter frame. This configuration provided single direction fixity perpendicular to the edge of the unit whilst retaining freedom of translation and rotation in the remaining directions.

2.2 Design criteria

The prototype building was conceived as being representative of a structure situated within the central business district of Wellington, New Zealand. For design purposes, a high importance occupancy such as a hospital or civil defence facility was allocated. A basic live load of 2kPa was adopted in accordance with the recommendations of the current New Zealand Loadings Standard. (NZS4203, 1992) and a distributed dead load of 5kPa was calculated comprising the self weight of the floor structure and superimposed loads. Allowance was made for the self weight of the remainder of the structure. For convenience, the same load rationale was applied to all levels.

For the purposes of this study, five earthquake demand levels as presented in Table 1 were defined in to provide an assessment of the design approach and structural response. For each earthquake demand level, a 5% damped uniform hazard design spectra was developed based on the parametric description given in the draft standard DR AS/NZS1170 (NZS, 2003) for Category C soils. This is shown in Figure 5(a).

Table 1 Earthquake demand design level

earthquake demand level	description	annual probability of exceedance	damage index	structural performance level
EQ-I	infrequent	1/25	negligible	fully operational (SLS I)
EQ-II	frequent	1/100	negligible	fully operational
EQ-III	occasionally	1/500	light	operational (SLS II)
EQ-IV	rare	1/1000	moderate	life safety
EQ-V	very rare	1/2500	severe	structural stability (ULS)

2.3 Evaluation of Seismic Resistance for Design

Traditional forced based design could not be used for the building considered due to its jointed precast

configuration. Instead a direct displacement design methodology was used. Although this technique is currently recognised as an acceptable design practice for evaluation of non-linear response it is acknowledged that this approach neglects dynamic effects which for a rocking system are highly complex.

In the application of a non-linear static procedure to evaluate the seismic resistance of a complicated structure, it is convenient to uncouple the various components of the structural system and appraise each one individually. Through cautious use of superposition the global monotonic behaviour can then be appreciated.

To evaluate the capacity of the structure due to the force of the prestressing alone, the principle of virtual work was applied. Each connection was modelled as a discrete bi-linear rotational spring. The work done at each spring was then determined by evaluating the integral of moment due to the applied prestressing with respect to local connection rotation, θ. By assuming an inverted triangular distribution of lateral forces and displacements over the building height the base shear was able to be determined.

The resistance to lateral loading afforded to the building due to its self weight may be quantified by considering a slender rocking block substitute structure with a base dimension equivalent to the effective rocking width provided by the columns. In contrast to the rotational stiffness provided by the beam-column connection which increases with top storey displacement, the restoring force here decreases with displacement. By once again assuming a linear distribution of inertial forces and displacements over the height of the building this characteristic may be expressed in terms of base shear force. Finally, conversion of the pushover curve to the acceleration/displacement domain was performed and is shown in Figure 6.

In the absence of a rigorous deterministic model for establishing the damping of the building as a function of displacement, a constant value of equivalent viscous damping was adopted. 2% intrinsic damping has been suggested as an appropriate value for conventional prestressed concrete frame structures (Chopra, 2001). However, it was anticipated that this would be complemented by other energy dissipating mechanisms in the structure including: (i) friction between the prestress medium and its ducting; (ii) material interaction at the beam-column connections and across floor joints; (iii) radiation damping due to the impact of the column bases on the foundation; and (iv) hysteretic damping due to the inelastic deformation of the ADAS elements. For convenience, total damping was assumed to be no more than 5%.

2.4 **Member Design and Fabrication**

The precast beam and column elements consisting of a series of composite sections were fabricated in the laboratory using commonly available steel sections and 40MPa concrete with a 6mm maximum aggregate size. For non-prestressed reinforcement, 7.5mm cold formed wire with a yield stress of approximately 500MPa was used representative of XD25 rod in the prototype. Tie steel was provided in the form of 2.5mm wire with a yield strength of around 300MPa. For the prestressing medium, a 7mm high strength rod was used with an innovative termination assembly which provided a stressing mechanism as well as anchorage.

The columns were terminated at both ends by steel caps which provided a fixing surface via threaded inserts for the base plates upon which the structure would rock. They also provided armament to regions where concentrated stresses due to rocking were expected to occur.

Two configurations of beam connection were developed for the test structure. In the longitudinal direction, short beam stubs were used to displace the localised rotations due to rocking away from the column face in a similar manner to that used in conventional reinforced concrete design. In the transverse direction, the rocking connection was located as close to the column face as practical. Gravity and seismically induced shear forces across the joints were accommodated by ledges installed at the top of the beam end caps and at the bottom of the receiving fixtures. To prevent rotation of the beams along their longitudinal axis due to eccentrically applied gravity loads from the floor system a vertical keyway was specified along the rocking interface.

2.5 ADAS Device Design and Testing

A wealth of research has been conducted at the University of Canterbury for variety of dampers with energy dissipation characteristics derived from flexural behaviour particularly in rocking walls (Toranzo, 2002; Sudarno, 2003) and more recently in post-tensioned beam-column sub-assemblies (Davies, 2004; Arnold, 2004).

At the expense of the reasonable efficiency offered by a fully fixed flexural dissipater arrangement, a simply supported beam concept as illustrated in Figure 2 was used. The objective was to offer additional stiffness to the structure while providing energy absorption during moderate to severe earthquake ground motions. Quasi-static unidirectional studies of the laser cut elements were performed using an Instron displacement controlled axial testing machine. Cyclic testing was undertaken with increasing displacement amplitude as shown in Figure 3(a).

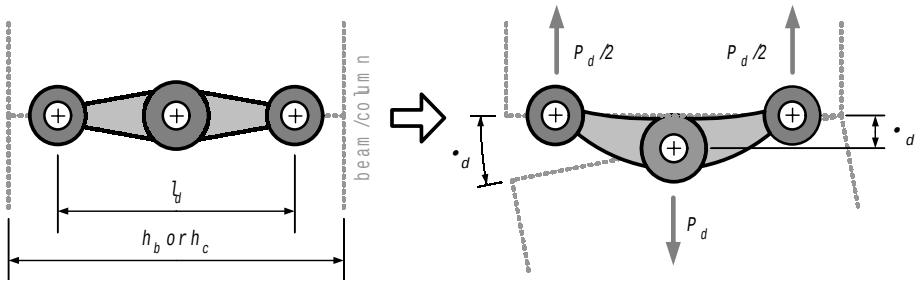
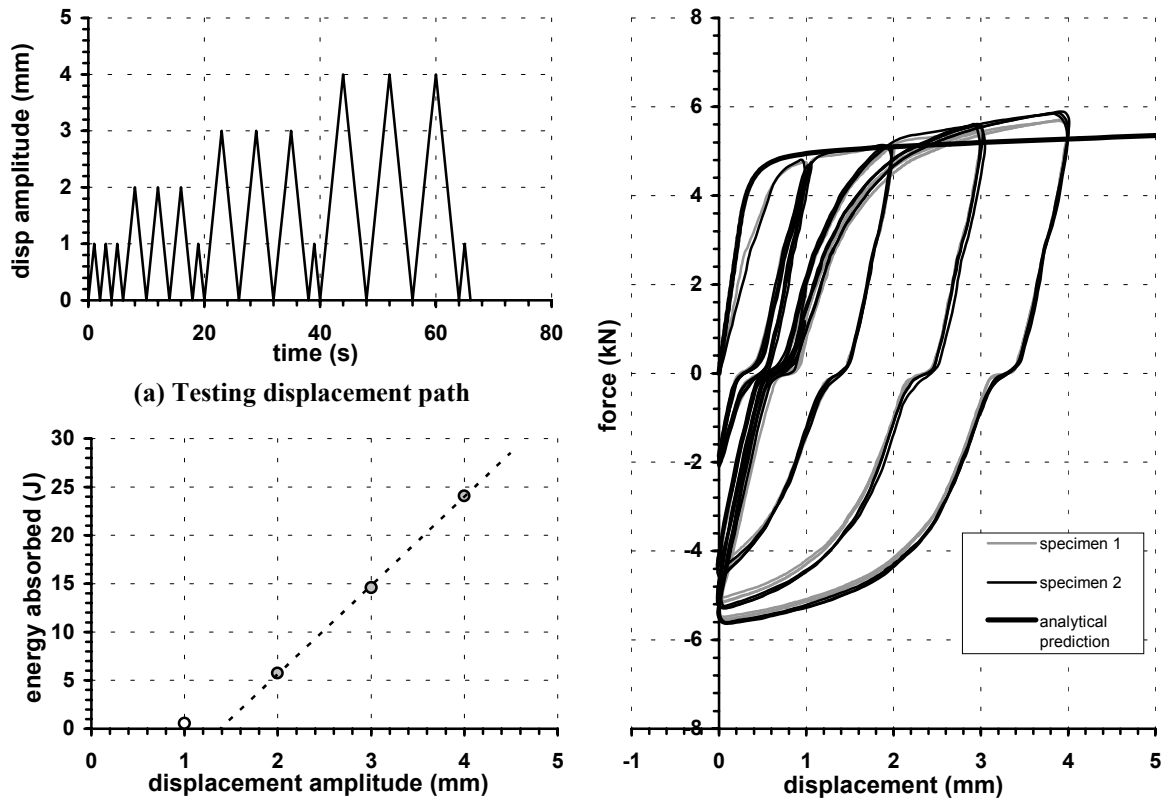


Figure 2 ADAS element concept

Reasonably good agreement was found between the predicted inelastic response, shown in Figure 2(c) and that observed from cyclic testing. Plastic work was evident in the first excursion to 1mm of displacement. However, the remaining cycles to the same amplitude contribute minimally to energy absorption as the now preloaded component was essentially cycled back and forth elastically.



(a) Testing displacement path

(b) Energy dissipated

(c) Force – displacement plot

Figure 3 ADAS element testing

Subsequent deformation to higher displacement amplitudes showed an almost linear increase in the amount of energy dissipation with hysteretic energy efficiency ratios of approximately 72% and 79% attained at displacements of 3 and 4mm respectively. At these amplitudes there is noticeable softening on the return portion of the hysteresis indicative of the Bauchinger effect. Also evident is the marginal slackness of the system due to tolerances in the device's fixities demonstrated by the pinching of the force-displacement plot at zero load.

3 DYNAMIC TESTING

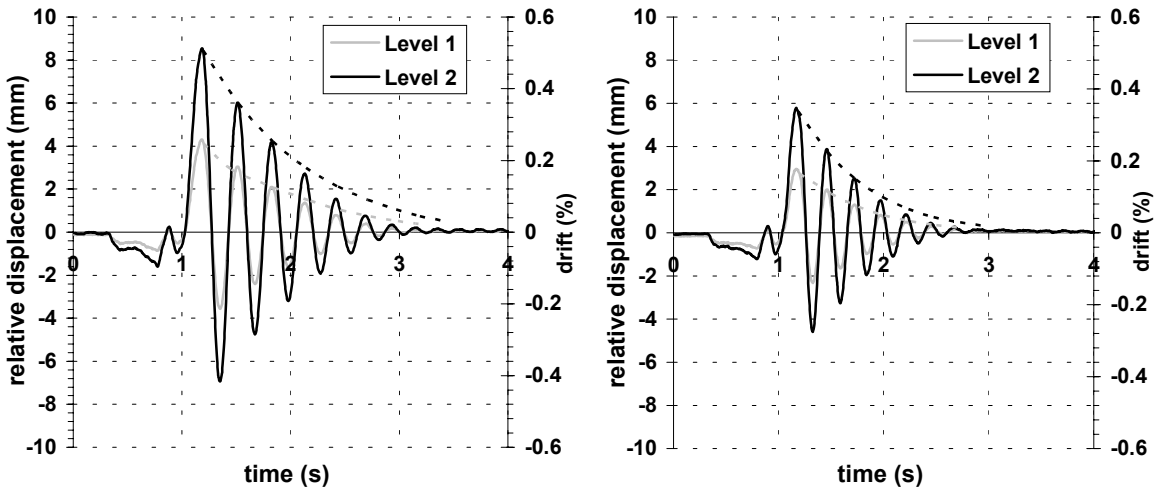
3.1 Instrumentation and Data Acquisition

The tension force of the prestressing rods in the structure was monitored by evaluating the signal from three strain gauge pairs mounted along the length of each rod. Accelerometers were installed horizontally at the mid span of all of the beams at the floor level. Base excitation was recorded by semi-conductive accelerometer and a strain gauge accelerometer linked to the control system. Potentiometers were located at each level at one end of the building and small potentiometers were installed centrally across the beam and column connections to monitor gap opening.

3.2 Free and Forced Vibration Tests

A series of snap-back, white noise, impulse and sine-sweep experiments were conducted to characterise the mode shapes frequencies and damping factors of the model structure. The following results were obtained: (i) First and second in-plane translational frequencies: 4.7 and 17Hz; (ii) Out-of-plane translational and first torsional frequencies: 5.0 and 9.5Hz; (iii) Damping 5% to 7% as measure by both the half-power and logarithmic decrement methods.

To gain insight into post rocking behaviour of the building, a series of rectangular impulse records with peak accelerations of between 0.025 to 0.3g were used. Comparison of the response with and without supplemental damping devices installed is presented in Figure 4.



(a) Without ADAS elements installed **(b) With ADAS elements installed**

Figure 4 Relative Displacement at Levels 1 and 2 under 0.125g impulse

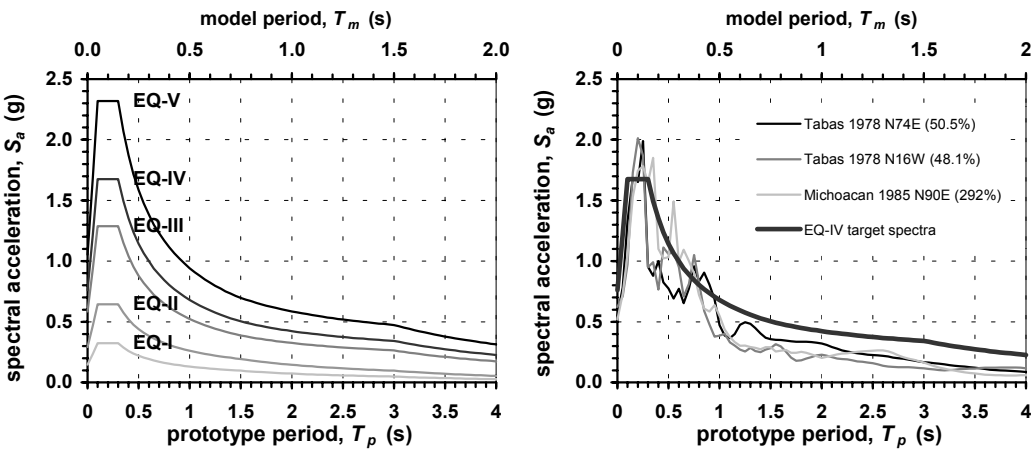
As anticipated, the presence of the ADAS elements provided supplemental stiffness and resulted in a reduction in lateral displacement of more than 30% during the initial stages of testing. The decay of motion was observed to depart from the expected response due to viscous damping alone as shown by the dotted lines in the above figure. The almost linear decay at large displacement amplitudes is indicative of Coulomb damping. This test also demonstrated the dependence of fundamental frequency on uplift amplitude.

3.3 **Simulated Earthquake Testing**

3.3.1 *Testing Program*

Three different configurations of the building were tested. The initial incremental testing was undertaken with the supplemental damping devices installed across the beam-column joints. A second series of tests were carried out with the ADAS elements removed from the beams in the longitudinal direction alone which included the commonly referred to El Centro 1940 North-South component record. Finally, a virgin set of ADAS elements were installed to the longitudinal beams and a single earthquake test was undertaken using the El Centro record.

For the incremental testing three earthquakes considered representative of a shallow soil site in Wellington, New Zealand, were chosen. The selection was based on analytical studies presented to the working committee for the DR AS/NZS1170 (NZS, 2003) and included records from the Tabas, Iran earthquake of 1978 and the East-West component of the Michoacan, Mexico earthquake of 1985. A 5% damped elastic spectrum for each earthquake was developed using the computer program Spectra (Carr, 2002). These spectra were then used to obtain scaling factors for each level of earthquake demand using the method proposed in DR AS/NZS1170. Matching of the spectra for earthquake level EQ-IV is shown in Figure 5(b). According to the laws of constant acceleration similitude for the scale of the building, records were run at twice their original time scale.



(a) DR AS/NZS1170 5% damped spectra **(b) spectral matching**

Figure 5 Earthquake excitation and spectrum matching

3.3.2 *Results*

The fundamental frequency of the structure both with the ADAS elements installed and with them removed was found to be 4.7Hz corresponding to a prototype frequency of 2.4Hz. The original design had been based on the assumption that the structural response prior to the onset of rocking could be likened to a building constructed using monolithic emulation techniques. However, the dynamic properties of the building departed from what had initially been predicted and this assumption was found to be invalid. Although this diminished stiffness was unexpected, it is a result consistent with the idea that localised stiffness reductions, be they the result of inaccuracies in fabrication or misalignment in the placement of structural elements, has the potential to dramatically influence the global dynamic properties of the structure.

Although the fundamental period of the building was unexpectedly low it remained constant throughout the testing program which is consistent with the observed absence of damage to the primary structural system. Residual displacements were negligible at the completion of each test.

Table 1 provides a summary of the maximum top storey displacements observed for each earthquake record and structural configuration. During the incremental earthquake testing it was found that in some instances marginally larger maximum displacements were recorded with ADAS elements

installed than without. This slightly softer response can be attributed to the preloading of the elements from the preceding tests. For the El Centro record a 35% reduction in maximum top storey displacement was observed with the damping devices installed when compared to the response when they were removed. Testing with the preloaded dampers resulted in a reduction of only 15%.

Table 1 Summary of maximum displacement response form incremental earthquake testing (Series 1 & 2)

	EQ-I		EQ-II		EQ-III		EQ-IV	
	no ADAS	with ADAS	no ADAS	with ADAS	no ADAS	with ADAS	no ADAS	with ADAS
Tabas 1978 N74E	*	2.3mm (0.14%)	6.3mm (0.38%)	5.4mm (3.3%)	27.5mm (1.7%)	12.1mm (0.73%)	*	50.4mm (3.1%)
Tabas 1978 N16W †	*	3.8mm (0.23%)	33.7mm (2.0%)	9.9mm (0.60%)	27.3mm (1.7%)	30.9mm (1.9%)	*	33.7mm (2.0%)
Michoacan 1985 N90E	*	4.1mm (0.24%)	11.6mm (0.70%)	13.5mm (0.82%)	25.6mm (1.6%)	27.3mm (1.7%)	*	27.5mm (1.7%)

* structural configuration not tested at earthquake level
† record with forward directivity
Total building drift are shown in parenthesis.

4 COMPUTER MODELLING

Computer modelling of the structure was undertaken using SAP2000 (CSI, 2000) and Ruaumoko 3D (Carr, 2003). This was limited to investigations of a two dimensional representation of the model and non-linear static pushover analyses were performed using both programs. Ruaumoko 3D was used to assess the feasibility of performing dynamic time history analyses for the type of structure examined here.

In SAP2000, members were simple beam elements with effective sections from New Zealand Concrete Structures Standard NZS3101 (SNZ, 1995). At the rocking connections, rotational springs were defined in terms of moment resistance over both an elastic and post-elastic range of rotations. Allowance for prestressing was made by applying an equivalent uniformly distributed load. A more refined model was developed in Ruaumoko 3D. Prestressing was provided to the model as discretised tendon profiles using bi-linear spring elements. These connections were described by two contact springs in parallel located at the top and bottom of the connection having high compression and zero tensile stiffnesses. Good agreement was obtained between the pushover curve developed using virtual work described previously and the two computer models as shown in Figure 6.

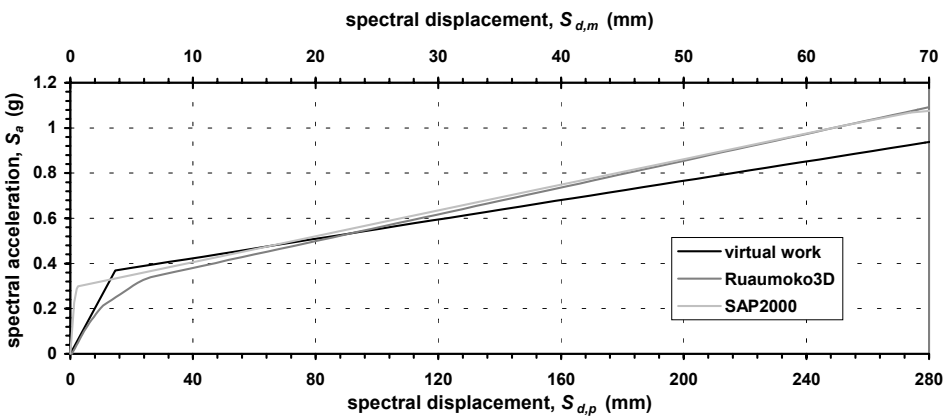


Figure 6 Earthquake excitation and spectrum matching

The SAP 2000 computer model demonstrated a very high pre-rocking stiffness corresponding to a

natural frequency of 10Hz, more than twice as much as was observed in the dynamic testing. The Ruamoko model was considerable softer however this response was found to be highly dependent on the selection of contact spring compression stiffness at the rocking interfaces. The elevated post-rocking stiffness was attributed to the inclusion of beam elongation effects in the model.

A time history sensitivity study was undertaken using the 0.3g impulse and the El Centro acceleration records taken from the dynamic tests. Comparison was made between the top storey displacement of the computer model and results from actual testing. It was determined that the computer model was most sensitive to three parameters: contact spring stiffness, applied prestressing magnitude and effective member depth at the rocking interfaces. Reasonably good agreement between the computer model and test structure response was obtained for the impulse acceleration record. However, the sensitivity of the model to the type of excitation was manifested in the poor agreement, both in terms of frequency and amplitude, from analyses using the El Centro record. Work to improve the computer model remains ongoing included efforts to allow for frictional effects proposed by Spieth et al. (2004).

5 CONCLUSIONS

It has been established from the dynamic tests of the model structure that precast post-tensioned reinforced concrete frame buildings able to rock on their foundations are inherently resistant to severe earthquake ground motions. By adopting a damage control philosophy, primary structural elements are able to be protected from displacement demands that could result in inelastic deformations and inhibit post-earthquake serviceability. Additionally, energy dissipation can be achieved through the use of replaceable ADAS elements. The rocking response of this type of structure has been shown to be sensitive to small changes in structural configuration and properties as well as variations in excitation particulars. The incorporation of flexural ADAS elements has been shown to improve both structural stiffness and displacement response under severe ground motions.

Work is continuing to improve the computer modelling of this type of structure and it is expected that in the fullness of time the test structure will be returned to the shake table to be tested in the transverse direction. It is probable that future research will also look at alternative supplemental damping devices. A set of comprehensive design procedures and criteria for the frame type building presented here is currently under development.

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