

RETROFIT OF SEISMICALLY ISOLATED STRUCTURES FOR NEAR-FIELD GROUND MOTION USING ADDITIONAL VISCOUS DAMPING

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SUMMARY

Recent earthquakes have shown that a large magnitude, long period pulse is often prevalent in ground motion records at sites within a few kilometres of the active fault during an earthquake. Near-field earthquake ground motion containing forward directivity effects can result in a larger response in flexible structures, such as seismically isolated structures, compared to that predicted for conventional ground shaking. Hence, a study was performed on a number of generic seismically isolated structures designed to the 1997 Uniform Building Code, as well as a case study on the William Clayton building in Wellington, to determine the impact of near-field ground motion. In optimising the performance of the buildings for both near-field and original "design level" earthquakes, it is concluded that linear viscous dampers added to the existing isolation systems are effective in controlling the response during large magnitude near-field earthquakes with minimal impact on the design response. Additional viscous damping is more effective than hysteretic damping in limiting isolator displacements while also preventing an increase in base shear and floor accelerations for far-field "design level" earthquakes.

INTRODUCTION

The basic principle behind seismic isolation is to isolate a structure from the ground using a flexible layer, which allows the ground to move with minimal effect on the structure itself. In terms of structural dynamics, the effective natural period of a structure is increased, reducing the seismic forces compared to a fixed base structure. This is only possible if the isolation system has a large displacement capacity, with excessive displacements prevented in part through high levels of damping compared to those typically observed in structures. Lead rubber bearings are one form of isolation device, which allow large displacements using flexible rubber layers and provide high levels of effective damping through the hysteretic behaviour of lead.

Recently it has been observed that structures, particularly flexible structures with long fundamental periods, have suffered considerable damage during large magnitude

earthquakes where the site was located within a few kilometres of the fault rupture [1, 2, 3]. In these regions the earthquakes are referred to as near-field (or near-source) earthquakes. Studies of the earthquake records in near-field regions have shown that many of these records exhibit a large magnitude, long period pulse. This is attributed to forward rupture directivity effects [4] caused by propagation of the rupture along a fault during an earthquake.

Seismically isolated buildings have a long fundamental period. Therefore, if not specifically designed to allow for the large displacement demand they are likely to be vulnerable to forward directivity effects. As an example, the seismically isolated William Clayton building in Wellington, New Zealand (Fig. 1) was designed in the late 1970's before "pulse type" ground motions were well understood. As the building is located near an active fault, the displacement capacity of the isolation system for this building is likely to be inadequate during a potential rupture of this fault. The

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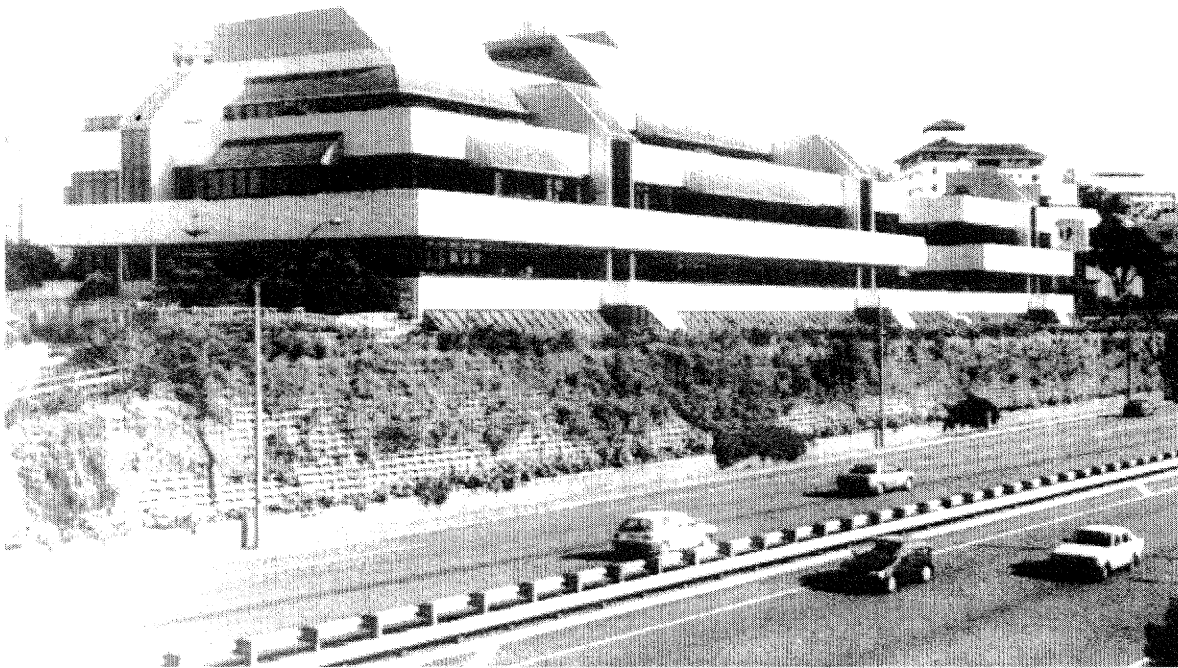


Figure 1. Photo of the William Clayton building in Wellington, NZ.

maximum displacements at the base of the William Clayton building are constrained to 150 mm before impact occurs due to the proximity of nearby retaining walls.

Consequently there is a need to retrofit such structures where the displacement capacity of the isolation system is likely to be inadequate during near-field earthquakes. As a first approximation, near-field earthquakes may be considered to behave like a pulse applied at ground level. Standard vibration theory, as outlined in Chopra [5], shows that for a linear single degree of freedom system the maximum displacement in response to a half sinusoidal pulse can be reduced by up to 40% using 40% additional viscous damping. It was considered likely that additional damping would be effective in reducing the displacement demand during large magnitude earthquakes. However, additional damping may provide a feasible retrofit if there are minimal adverse effects during a far-field earthquake or one where the fault rupture is not in the direct vicinity of the building. Furthermore, as large levels of damping may be required to retrofit an isolated structure in this manner, it was considered

likely that the type of damping will be important in optimising the response of a building.

Two past studies have suggested a possible retrofit for the William Clayton building using different forms of additional damping [6, 7]. In these studies the focus was on hysteretic buffers and hysteretic dampers, which were able to reduce the effects of near-field ground motion. However, this paper contrasts the effectiveness of hysteretic damping with that of viscous damping and other systems to investigate a wider range of retrofit options. A number of generic hypothetical buildings, designed to the 1997 Uniform Building Code (UBC) [8] were investigated in this study. However, the response of the William Clayton Building is used to illustrate findings from these investigations as its response is representative of the response of each of the different generic buildings. Moreover, it can be directly compared with the previous studies.

MODEL OF THE WILLIAM CLAYTON BUILDING

The William Clayton building is a four storey reinforced concrete frame structure as described by Megget [9] (Fig. 1). To determine the lateral performance of the isolation system, a typical transverse frame of the building (Fig. 2) was modelled using SAP2000 [10]. It was assumed that the frame was elastic except for the isolation system, which was modeled with bi-linear properties identical to those used in the original design.

To investigate the lateral performance of the building frame to near-field and far-field earthquakes, a series of non-linear time history analyses were performed. A number of acceleration records from past earthquakes, previously identified as containing forward directivity near-field ground motion [3, 4], were selected to represent near-field earthquakes. Two artificial earthquakes were included amongst the near-field earthquake records, including one from a simulated fault rupture during the 1994 Northridge earthquake, and a second from a predicted fault rupture of the Elysian Park fault in southern California. Other earthquake records were selected to represent far-field earthquakes. These included the 1940 El Centro north-south record which was used in the original design of the William Clayton building, hence these earthquakes are considered “design” level earthquakes. The Joshua Tree record from the 1992

Landers earthquake was selected as one that exhibited backward directivity effects and is also included amongst the design level earthquakes. A summary of the near-field and design level earthquake records is given in Table 1.

Suitable scale factors for the different earthquake ground motions were calculated to provide a consistent response for the unretrofitted building. The design level earthquakes were scaled such that the maximum response, from a time history analysis of an idealized single degree of freedom model of the William Clayton building frame, resulted in a maximum displacement of 122 mm. This was the maximum displacement calculated in response to 1.5 times El Centro which was used in the original design of the building. The idealized single degree of freedom model assumed a bi-linear isolation system, with the same properties as used in the original design [9], but instead of a flexible building frame, a rigid superstructure was assumed, for numerical efficiency during scaling. The near-field earthquakes were scaled using the assumption in the 1997 Uniform Building Code (UBC) that, the spectral ordinates for accelerations at a near-field site are twice those for a far-field site in the same seismic zone [8]. Had the structure been designed using the UBC for a zone outside a near-field region, a seismic coefficient equal to 0.42 would result in the same isolation system

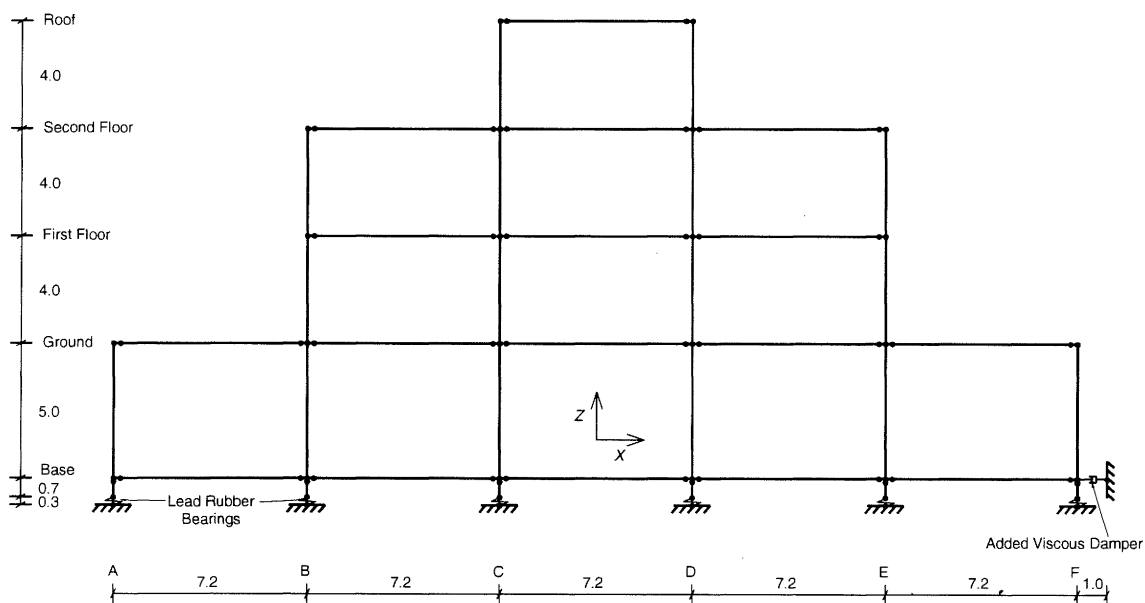


Figure 2. Two dimensional multi-storey model of a transverse frame of the William Clayton building.

Table 1. Properties of selected design level and near-field earthquakes.

Year	Reference	Earthquake	Station	Comp.	Mag.	Epicentral Distance (km)	Focal Depth (km)	Soil Type	Peak Accn (g)	Peak Vel. (m/s)	v/a (s)	Scale Factor
Design Level Earthquakes												
1940	El Centro ¹	Imperial Valley	El Centro Array #9	NS	6.9	8	9	Stiff	0.35	0.32	0.09	1.50
1966	Parkfield ¹	Parkfield	California Array #2	N65E	6.1	36	7	Stiff	0.50	0.78	0.16	0.55
1977	Bucharest ²	Bucharest	Building Res. Inst.	NS	7.2	n/a	94	Soft	0.20	0.72	0.37	0.55
1992	Joshua Tree ¹	Landers	Joshua Tree	EW	7.3	14	5	Stiff	0.28	0.43	0.15	0.95
Average					6.9	19	29		0.33	0.56	0.19	0.89
Near Field Earthquakes												
1994	Northridge (Sim.) ³	Northridge	Artificial E04		6.7	25	19	Stiff	0.77	1.76	0.23	0.50
	Rinaldi ¹		Rinaldi	\$49W	6.7	10	19	Stiff	0.84	1.70	0.21	0.60
	Sylmar ⁴		Sylmar Hospital	360	6.7	16	19	Stiff	0.84	1.29	0.16	0.65
Pred.	Elysian Park ²	Elysian Park	Artificial E05		7.0	9	9	Rock	0.93	1.76	0.19	0.90
1992	Lucerne ¹	Landers	Lucerne	N90E	7.3	42	5	n/a	0.73	1.46	0.20	1.20
1979	Imperial Valley ¹	Imperial Valley	Array #7	230	6.4	29	12	Stiff	0.46	1.13	0.25	1.25
Average					6.8	22	14		0.76	1.52	0.21	0.85

Notes: 1. Acceleration and velocity taken from source data

2. Acceleration taken from source data. Velocity calculated by SAP

3. Acceleration taken from source data. Velocity as published by Somerville [4]

4. Acceleration taken from source data. Velocity as published by McVerry [3]

displacement as that calculated in response to 1.5 times El Centro. Consequently, using the UBC philosophy a seismic coefficient of $2 \times 0.42 = 0.84$ was chosen as suitable for the structure when known to be located within two kilometres of an active fault. Using this near-field seismic coefficient, a UBC analysis of the idealised structure resulted in a maximum near-field displacement in the isolation system of 314 mm. Thus each of the near-field earthquakes were scaled such that the time history response to the idealized single degree of freedom model resulted in a maximum displacement of approximately 314 mm. The scale factors for each near-field and far-field earthquake are listed in Table 1.

The frame of the William Clayton building was then modelled in SAP2000 as a full two dimensional frame incorporating the properties of the isolation system and beams and columns of the superstructure (Fig. 2). The dimensions and properties of the model were chosen to represent properties of the real structure as in the initial time history analysis performed during design of the building in Drain2D by Megget [11]. Each beam and column was modelled with rigid end zones of 0.25 m at either end of the

element to allow for the joint regions. Plastic hinges were modelled at the end of the rigid end zones and were given a high initial stiffness, so that the effect of each hinge was negligible while the element remained elastic. They were assigned appropriate yield moments and post-yield stiffness in order to be consistent with the original Megget model after the elements yielded. The isolation system was modelled with six bi-linear shear elements as per the original model. Similarly, the mass of the building was distributed at each floor as per the original analysis with rigid diaphragms assumed by slaving the nodes at each floor. Damping was modelled in Drain2D as Rayleigh damping using mass and stiffness proportional damping coefficients. For modelling in SAP2000 these were converted to a fraction of critical damping for each mode. To verify that the model of the building in SAP2000 was comparable to the original Drain2D model, the 1.5 x El Centro response of the building was compared using time history analyses in Drain2DX, a current equivalent version of Drain2D, and SAP2000. The base and top floor displacements of the building are compared in Figure 3 with less than 3% difference illustrated between the maximum displacements from SAP2000 and Drain2D at the base and roof of the building.

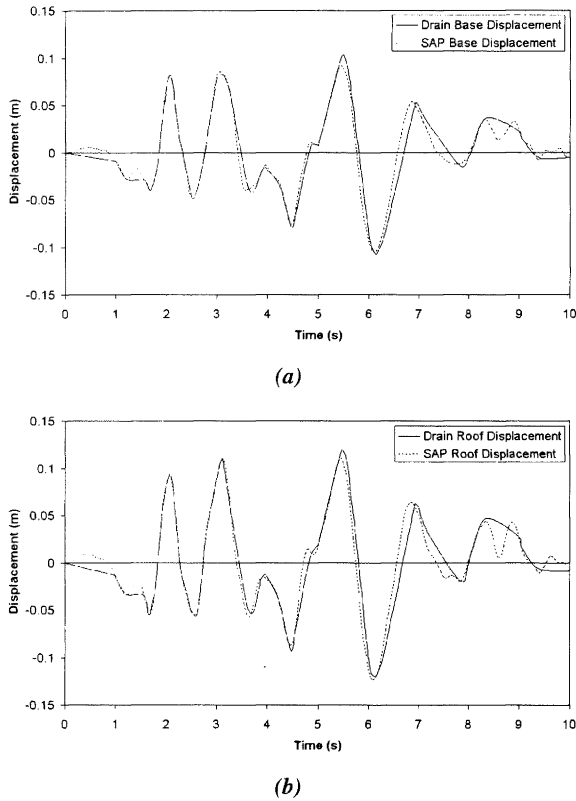


Figure 3. Comparison of 1.5 times El Centro time histories using SAP2000 and Drain 2DX at a) base and b) roof.

RESPONSE OF THE AS-BUILT BUILDING

The average maximum displacement in the isolation system, in response to the near-field earthquakes scaled according to the scale factors in Table 1, was 273 mm. This was 15% less than the isolator displacement from the idealized single degree of freedom system due to superstructure flexibility. The average maximum isolation system displacement was almost two times the maximum allowable displacement of 150 mm, dictated by the proximity of retaining walls adjacent

to the William Clayton building. Thus, based on this analysis a near-field earthquake would cause pounding of the structure on the adjacent retaining wall which was shown in other studies to cause inelastic behavior in the building [6, 7] along with the damage to the retaining walls. The design displacement was 108 mm averaged over the four design level earthquakes, which is equal to the design displacement from the original El Centro time history analysis in Drain2D [11].

In order to investigate the as-built performance of the superstructure, a non-linear pushover analysis of the frame was performed assuming a fixed base structure with no isolation system. Forces were applied to each floor of the building, proportional to the weight and height of each floor using an equivalent static analysis (Table 2). The resulting pushover curve for the roof of the building is shown in Figure 4. Fitting a bi-linear model to the pushover curve resulted in a calculated yield displacement of 42 mm. The inter-storey yield displacements were assumed to be distributed linearly up the height of the building, resulting in those listed in Table 2. These correspond to 0.25% inter-storey drifts.

The inter-storey displacement ductility of each floor was calculated in response to each earthquake from time history analyses of the frame. The average inter-storey displacement ductility for the design level earthquakes was between 0.4 for the first floor and 0.2 for the top floor. The near-field response of the upper floors had ductilities of approximately two times these values, at between 0.8 and 0.4. Therefore, each floor remained essentially elastic although some localized inelastic rotations in the plastic hinges of the beams were observed.

Table 2. Equivalent static forces and calculated yield displacements in superstructure of William Clayton Building.

Level	Weight (kN)	Height (m)	Relative Equivalent Static Force	Yield Displacement (mm)	Inter-storey Yield Displacement (mm)
Base	2550	0	0	0	
Ground	2820	5	0.208	12.4	12.4
2	1580	9	0.210	22.2	9.9
3	1580	13	0.303	32.1	9.9
Roof	1110	17	0.279	42.0	9.9
Total	9640		1.00		42.0

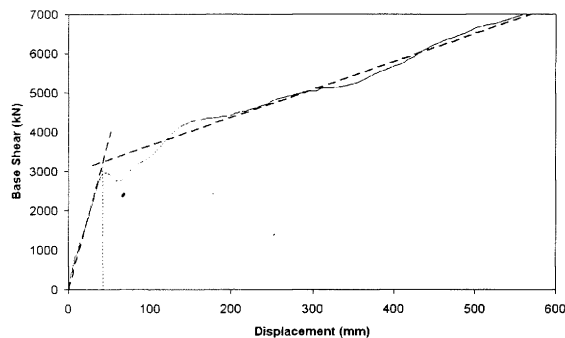


Figure 4. Force-displacement curve at the roof of the building frame from pushover analysis.

From these analyses it was apparent that there is a need to reduce the maximum displacement of the isolation system for near field earthquakes. Although, it was recognised that if the isolator displacements could be reduced with minimal increase in base shear then the performance of the superstructure of the building would be expected to be essentially elastic in response to near-field earthquakes. Thus, no retrofit of the superstructure would be necessary.

OPTIONS FOR RETROFIT USING ADDITIONAL DAMPING

During studies on generic seismically isolated structures various levels of different forms of additional damping were attempted as a means to retrofit the isolation system of the buildings. These forms of damping included linear viscous damping, non-linear viscous damping, hysteretic damping, and a dual level hysteretic buffer as suggested by Skinner and McVerry [12]. In addition mass tuned damping was also investigated in the current study [13]. It was found that a dual level hysteretic buffer was unable to adequately reduce the maximum displacement of the isolation system in response to near-field earthquakes and the mass tuned damper was also ineffective. Consequently just three forms of retrofit were considered for the William Clayton building.

Mathematically, the force in a viscous damper can be described by:

$$F_{Damp} = c\dot{u}^\gamma \quad (1)$$

where c = damping constant; \dot{u} = damper velocity; γ = velocity exponent. Forms of viscous damping are differentiated by the velocity exponent which typically ranges between one, for linear viscous damping, and zero, for properties equivalent to elasto-plastic hysteretic or friction damping. In the earlier studies additional dampers with three velocity exponents equal 1.0, 0.5 and 0.3, which are inside the typical range for commercially available dampers, were attempted for retrofitting the generic buildings. A fourth form of damping, with a velocity exponent of 1.5, was also modelled, although a damper with such properties may not be currently available. From these previous studies two velocity exponents, equal to 1.0 and 0.5 were considered most effective. These were consequently used in attempts to retrofit the William Clayton building. Different levels of damping were defined by varying the damping constant. Damping constants were calculated based on 5, 10, 20, 30 and 40% of additional damping using:

$$c = 2\xi \sqrt{\frac{Wk}{g}} \quad (2)$$

where ξ = fraction of critical damping; W = total weight of the building; k = secant stiffness of the isolation system at the design displacement; and g = acceleration due to gravity. Examples of commercially available viscous dampers for seismic applications include: hydraulic piston fluid dampers such as those manufactured by Taylor Devices Inc. [14] (Fig. 5), or hydraulic pot dampers such as GERB high viscous dampers (Fig. 6) [15], or viscous damping walls [16].

Additional elasto-plastic hysteretic damping was also considered. Devices which provide hysteretic behavior with minimal post-yield stiffness [7] include Penguin Vibration dampers [17] and lead extrusion dampers [18]. For modeling of the hysteretic dampers a high initial stiffness was assumed, thus the properties of the additional damping were governed by the yield force. Various levels of hysteretic damping were defined by yield forces equal to 0.02W, 0.05W, 0.10W, and 0.15W, where W is the total inertial weight for the frame of the William Clayton building.

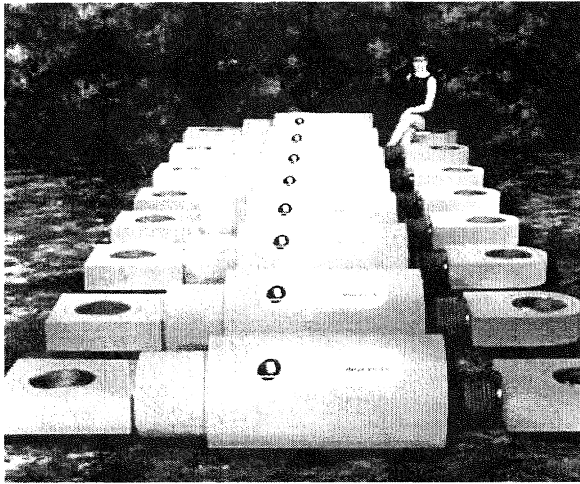


Figure 5. Fluid viscous dampers from Taylor Devices [14].

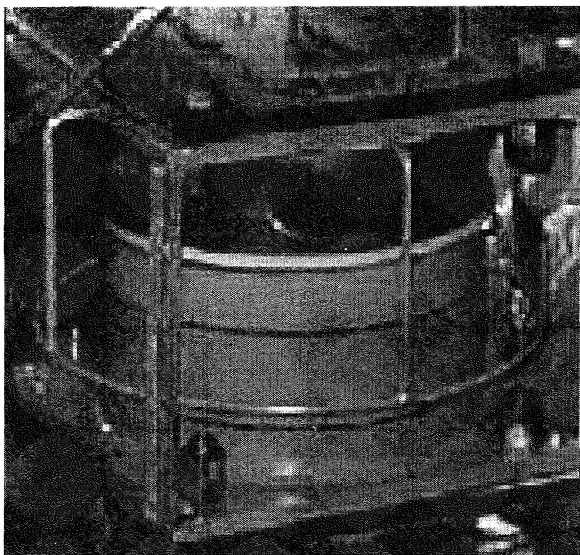


Figure 6. High viscous pot dampers [15].

RESPONSE OF THE BUILDING WHEN RETROFITTED USING DIFFERENT FORMS OF ADDITIONAL DAMPING

The maximum displacement and base shear at the base of the William Clayton building with different levels of each form of addition damping, averaged over the individual design and near-field earthquakes respectively, and shown in Figure 7. The design response is represented by the set of curves on the left hand side while the near-field response is represented by the set of curves on the right. Each curve represents a different form of damping and the points indicate increasing

levels of damping from the as-built response on the right hand side of each set of curves.

Figure 7 shows that all three forms of additional damping are able to reduce the maximum near-field displacement to less than the maximum allowable displacement of the isolation system. The optimum level of damping for each form of additional damping was defined as the level of damping which reduced the average near-field displacement response to the maximum allowable displacement of 150 mm and was interpolated from the figure.

Base shear was used as an indicator of the performance of each form of damping as it is a measure of distress in the superstructure of the building. The average base shear for the near field earthquakes using both forms of additional viscous damping was equal at their corresponding optimum levels, while the base shear using additional hysteretic damping was 5% higher. Thus in terms of near field response there is little differentiation between the different forms of damping.

There was a greater differentiation, however, when considering the design level response averaged between the four considered design level earthquakes. The design level base shear using linear viscous damping was the lowest at the optimum level of damping calculated from the near field response compared to the other forms of damping. Conversely, the base shear was largest using the optimum level of additional hysteretic damping, and while the design level base shear was reduced with use of additional viscous damping, no reduction was observed with additional hysteretic damping. Thus all three forms of damping are able to reduce the near field response to an allowable level without increasing the design level base shear, however, additional linear viscous damping was most effective in terms of minimising base shear in the building during a design level earthquake.

Figure 8 shows the displacement response for the different floors of the building frame using the corresponding optimum levels of additional damping, again averaged over the individual design level and near-field earthquakes respectively. The top horizontal axis gives the inter-storey ductility of each floor excluding the base, while inter-storey drift is given on bottom horizontal axis. As the displacement

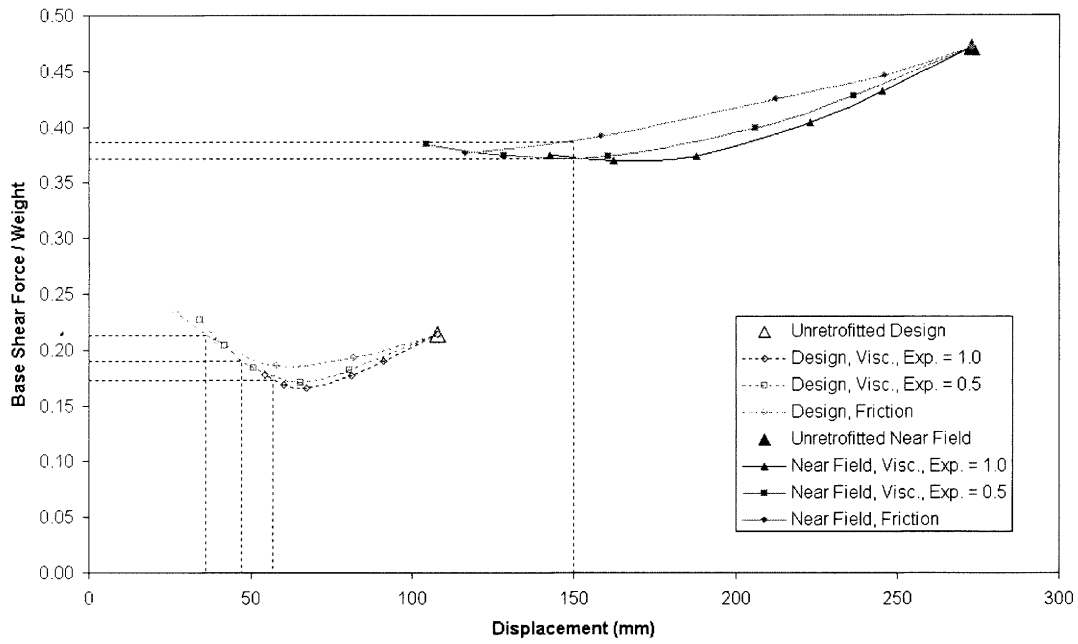


Figure 7. Average design and near-field response of isolation system retrofitted with three forms and increasing levels of damping.

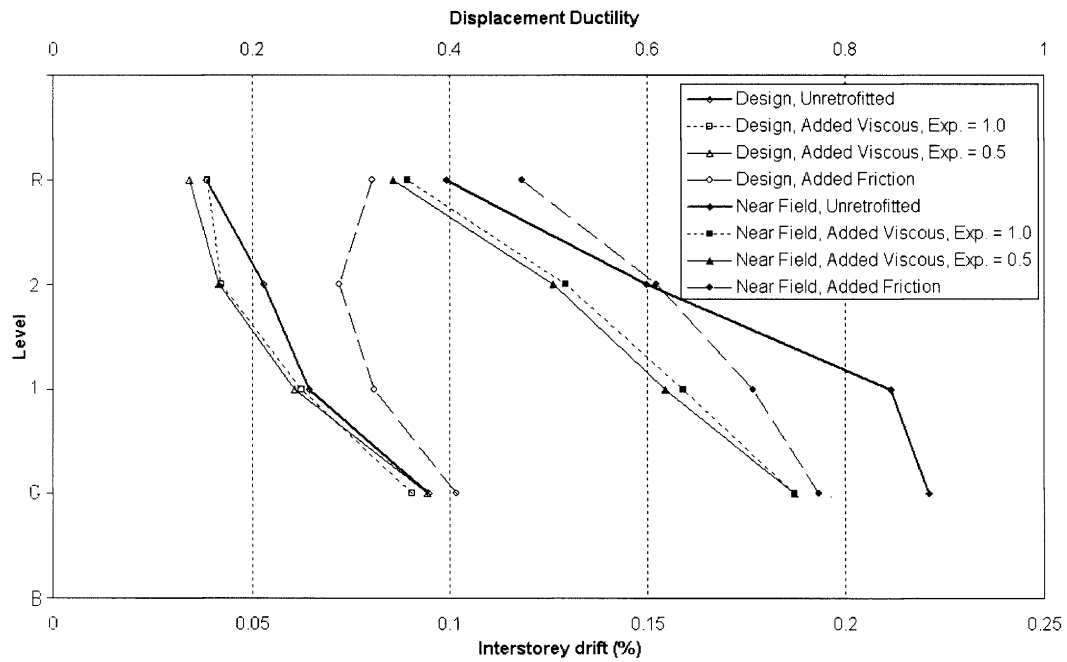


Figure 8. Average inter-storey drift at each floor.

ductilities at each floor are less than 1.0 for the unretrofitted and retrofitted building with each different form of damping, no global inelastic behavior is expected in the superstructure. However, Figure 8 shows that the deformations in the superstructure with the two forms of additional viscous damping are similar and smaller than the unretrofitted response, while additional hysteretic damping induced 10-100% larger displacements. This is consistent with the lower base shear measured using additional viscous damping compared to additional hysteretic damping as the superstructure is essentially elastic.

RESPONSE OF INDIVIDUAL EARTHQUAKES

A study of individual earthquake responses at the base of the building frame showed that the variation between the individual earthquakes was smallest when the building was retrofitted using additional linear viscous damping. Figure 9 shows the maximum displacement and base shear response to the individual earthquakes when increasing levels of linear viscous damping are added to the base of the building. In response to the Bucharest record, which was included to investigate the effects for a site founded on soft soil, the

building showed the most favourable performance out of the design level earthquakes. Joshua Tree which exhibits backward directivity ground motion, less damaging but equally as likely as forward directivity ground motion, resulted in a response close to the average response. The least favourable design level response was obtained from the El Centro ground motion when linear viscous damping was added. However even for this record, at large levels of damping the base shear was less than the base shear in the as-built building. The near-field earthquakes also showed some variation in response. The maximum displacement and base shear were very effectively reduced in response to the Sylmar Hospital record. In contrast the most severe response of the building with additional linear viscous damping, was found when the building was subjected to the artificial Elysian Park record. A similar figure is shown in Figure 10 for the building when additional hysteretic damping is added. The greater variability is apparent in this figure compared to Figure 9, indicated by the spread between the individual responses. These figures indicate that the response of the building with additional linear viscous damping is less sensitive to the type of earthquake than the other forms of damping.

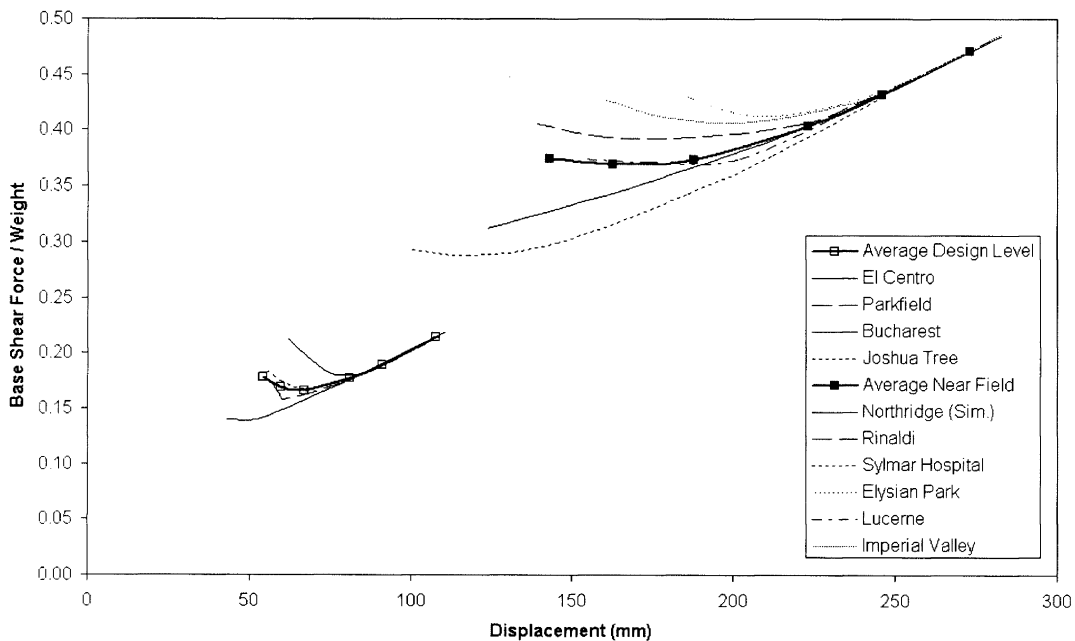


Figure 9. Individual earthquake response at the base for various levels of linear viscous damping.

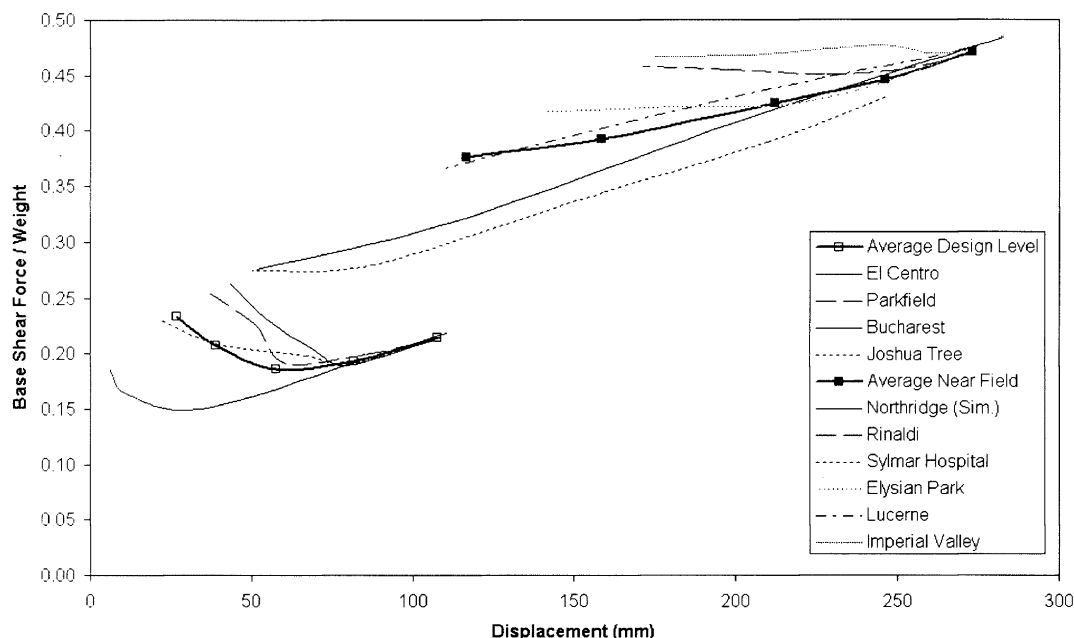


Figure 10. Individual earthquake response at the base for various levels of hysteretic damping.

With the optimum level of additional linear viscous damping as previously calculated the maximum displacement in response to the worst near-field earthquake would have been 190 mm, 25% greater than the maximum allowable displacement. However, even for this worst case, the as-built displacement was reduced by 67% of that necessary to prevent the isolation system exceeding its maximum allowable displacement, therefore while some additional forces and subsequent damage would be expected due to pounding between the building and adjacent walls, a large portion of the damage would be prevented by using the additional damping.

ACCELERATION SPECTRA AT EACH FLOOR

Acceleration response spectra at different floors can be used to identify the amount of damage expected in non-structural components of a building. The acceleration spectra at each floor of the William Clayton building were calculated to further investigate the impacts of possible retrofits using various forms of additional damping on the design level response of the building. Figure 11 shows the acceleration spectra at each floor in response to El Centro, the design level earthquake inducing the most severe response with additional damping. It can be seen that, for periods between

0.3 and 1.1 seconds, the acceleration response in the structure is increased. However, the increase in acceleration is smallest using additional linear viscous damping. This increase was two to three times larger using additional hysteretic damping, particularly at the upper floors with the maximum response observed at the top floor. The acceleration floor spectra for the other design level earthquakes are shown in Figures 12 to 14. These again show that the response with additional friction damping is most severe and additional linear viscous damping has the smallest impact on the response. Thus, floor acceleration spectra further support the use of additional linear viscous damping for retrofit over hysteretic damping.

OPTIMUM DAMPER PROPERTIES

The optimum damper properties for this frame of the William Clayton building have a viscous damping constant equal to $0.320 W$ (s/m), where W is the total weight of the frame. This is equivalent to 36% of critical viscous damping based on the effective period (1.44 s) of the isolation system at the design displacement. This type and level of damping can be achieved using the commercially available dampers mentioned earlier.

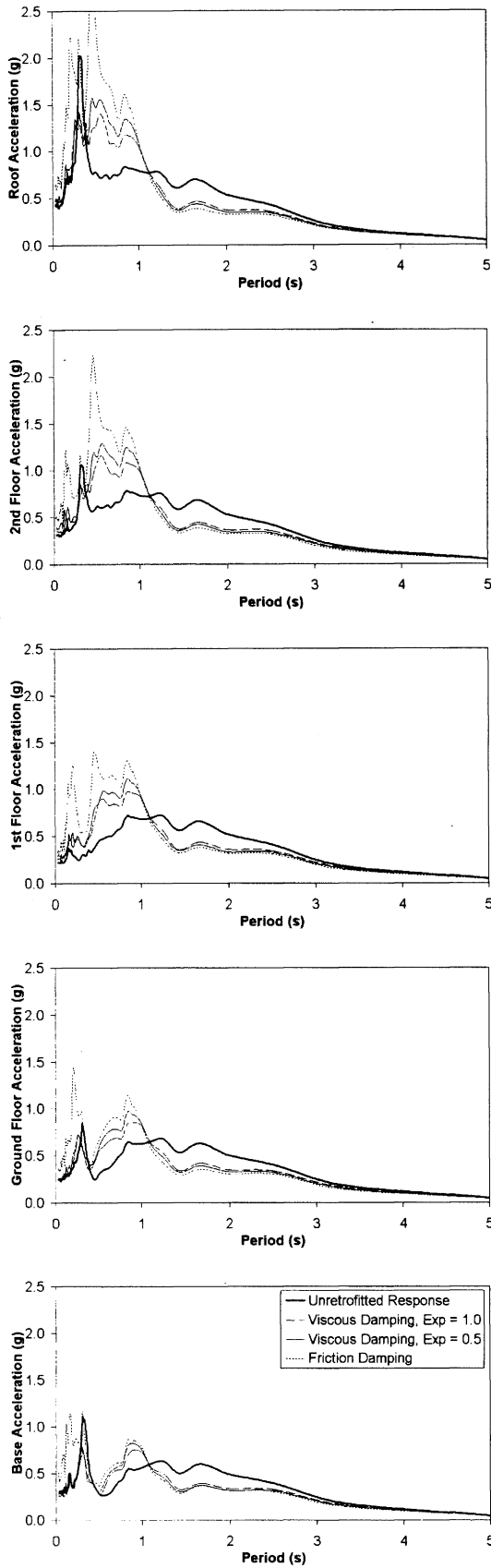


Figure 11. El Centro acceleration spectra at each floor.

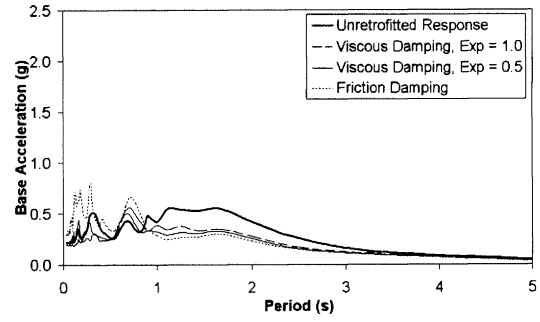


Figure 12. Parkfield acceleration spectra at the base.

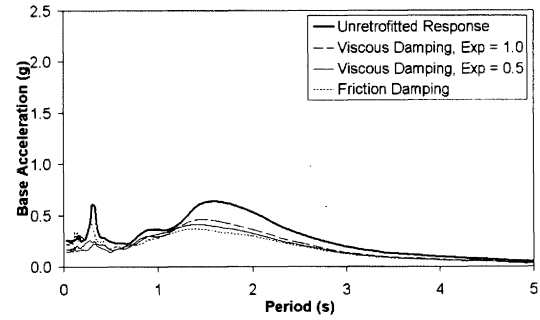


Figure 13. Bucharest acceleration spectra at the base.

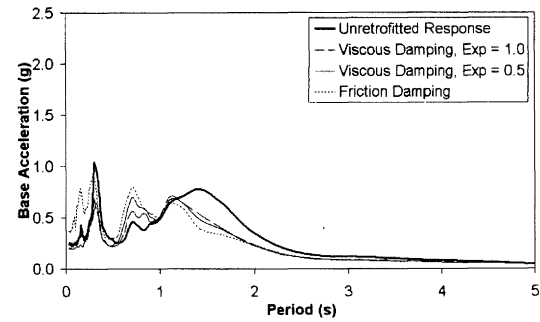


Figure 14. Joshua Tree acceleration spectra at the base.

SUMMARY AND CONCLUSIONS

Near-field earthquakes can result in larger ground motion than previously anticipated in regions close to active faults. Seismically isolated buildings can be particularly vulnerable as the near-field earthquakes may result in larger than designed isolation system displacements. A study was performed on a series of generic isolated buildings as well as a more specific study on the William Clayton building in Wellington, NZ, which was found to be typical of the generic structures. The maximum near-field displacement response

of the isolation system in the William Clayton building, using criteria taken from the 1997 Uniform Building Code [8], was predicted to be almost twice the maximum allowable response for the existing isolators. This would cause damage to structures surrounding the building and induce large forces in the superstructure as a result of a subsequent impact. It was found that additional damping in the isolation system was able to limit isolator displacements to their maximum allowable level.

Different forms of additional damping including hysteretic, linear viscous and non-linear viscous systems were found to be effective in reducing the near-field response of the William Clayton building. However, additional linear viscous damping was found to be more effective than the other systems for the following reasons:

The near-field displacement response of the isolation system is limited to maximum allowable levels with the smallest level of base shear.

The base shear in response to far-field "design level" earthquakes is smallest.

There is less variability between individual earthquake responses.

The floor acceleration spectra are least affected by the additional damping, with minimal additional non-structural component damage expected and some reduction possible.

It is anticipated that this study could be applied to the detailed design of a retrofit for the William Clayton building using commercially available viscous dampers. However, the concepts presented could also help to optimise the design and retrofit of other structures using passive energy dissipation.

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