SEISMIC RESPONSE OF REINFORCED CONCRETE SHEAR WALLS YIELDING IN A COMBINED SHEAR AND FLEXURAL MODE

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SUMMARY

Reinforced concrete shear wall buildings designed prior to the introduction of capacity design procedures can be expected to fail, at least partially, in a shear displacement mode. When assessing the likely performance of these existing buildings, the shear displacements expected to occur is an important consideration.

A computer model of a shear wall, able to deform inelastically in either a shear and/or flexural mode, was developed and the model was analysed using inelastic dynamic analysis procedures. A number of variables were considered including the shear strength of the plastic hinge region relative to it's flexural strength, the nature of the earthquake motion, the initial stiffness of the wall and the flexural yield strength of the plastic hinge zone.

If an existing shear wall was assessed on the basis of a code type inverted triangular seismic load distribution to have just sufficient shear strength to permit flexural yielding, it might be concluded that most of the inelastic wall response would be in the flexural mode. However, this study indicates that higher mode and other dynamic effects would ensure that most of the inelastic seismic deformations of the wall would occur in the shear rather than the expected flexural mode. This study concludes that relatively large reserves of shear strength are required in a shear wall to ensure that inelastic shear deformations are kept to a small proportion of the inelastic demand.

INTRODUCTION

The New Zealand concrete design code (NZS 3101, 1995) requires structural walls to be capacity designed to ensure that they do not fail in a shear mode if advantage is to be taken of ductile flexural yielding to substantially reduce the seismic design loads.

To comply with the capacity design requirements, the wall shear force calculated assuming the inverted triangular distribution of loads must be factored up to allow for both dynamic magnification and probable overstrength of the plastic hinge moment capacity. The overstrength factor allows for such parameters as the actual detailed reinforcement content and the probable yield strength of the reinforcement. The dynamic modification factor principally allows for lowering of the effective height of the dynamic load centroid due to higher mode effects.

The effect of higher modes on the moment/shear ratio at the base of an elastically responding wall is illustrated in Figure 1.

This shows two alternative ways of combining the first and second modes of a building's response to dynamic loads. It can be seen that the level of the centroid, h, of the dynamic load for the wall corresponding to its first mode only, is increased or reduced to h_1 or h_2 by the 2^{nd} mode loads.

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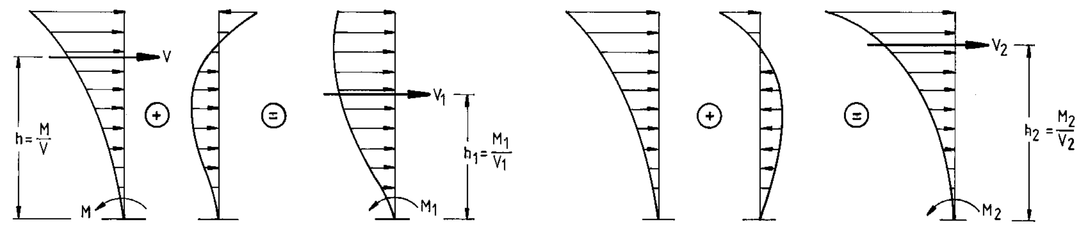


(a) 1st mode (b) 2nd mode (c) 1st + 2nd mode (d) 1st mode (e) 2nd mode (f) 1st + 2nd mode

Figure 1 Effect of higher modes on moment/shear ratio at base of wall

The dynamic magnification factor used in current capacity design allows for the amplification of the wall shear (calculated using essentially a triangular distribution of load) that can occur during a building's seismic response due to the influence of higher modes. This higher shear/moment ratio effect corresponds to the low moment/shear ratio case indicated by Figure 1c. By allowing for flexural overstrength at the base of the wall, ϕ_o, and dynamic magnification of shears, the capacity design procedure given in the NZ design code aims to ensure failure in a flexural rather than shear mode.

Most buildings have traditionally been designed for a triangular distribution of equivalent static loads which have a similar moment/shear ratio at the base of the wall as that given by the walls first mode response. Therefore, most shear wall buildings built in New Zealand prior to 1976 do not meet the capacity design requirements of the current code and are therefore expected to fail, at least partially, in a shear mode.

Draft guidelines for the "Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings" have been published by the New Zealand Society for Earthquake Engineering (NZNSEE, 1996). These guidelines state that there is no need to allow for dynamic magnification of shear forces because of the "comparatively short duration of dynamically amplified shear action". This recommendation seems reasonable when only the elastic response of a shear wall is considered.

Table 1

Number of storeys of wall	Roof Level Acceleration (%g)				Roof Level Displacements (mm)		
	1st mode period (sec)	1st mode	2nd mode	1st mode / 2nd mode	1st mode	2nd mode	1st mode / 2nd mode
12	0.9	0.78	0.644	1.21	156	4.1	38
24	3.41	0.23	0.547	0.42	658	42.6	15.4
30	5.28	0.145	0.436	0.33	1000	80	12.5

Note: Sample walls obtained from Fenwick *et al*, 1989; modal analysis used the response spectrum given in NZS 4203, 1992 for μ = R = Z = 1.0 and normal soils)

Table 1 indicates that, although the dynamic loads (accelerations) generated by the 2nd mode of an elastically responding wall are large relative to those generated by the 1st mode, the displacements generated by the 2nd mode are relatively small. Therefore, if a wall had just sufficient shear and flexural strength to respond elastically in its first mode, the inelastic displacements generated by higher modes could be expected to be quite modest.

This is still thought to be true for walls with sufficient strength to respond elastically in their first mode. However, the research presented in this paper established that this is not true when walls have a strength level that results in significant inelastic response. In this case, earthquake motions are capable of generating a large shear displacement demand in the wall if higher mode amplification of shear forces has not been allowed for in the design of the wall.

This paper describes an investigation into the seismic response of reinforced concrete shear walls yielding in a combined shear and flexural mode. The investigation was part of an original study funded by the EQC Research Foundation and is described in more detail in the report for that study (Blaikie *et al*, 1990).

DETAILS OF INELASTIC DYNAMIC ANALYSIS

Computer Model Developed for Analysis

In order to evaluate the inelastic shear displacement demand that could be imposed on existing shear wall buildings by earthquakes, two shear wall buildings designed and built in the late 50s and early 60s were selected for study by computer modelling and inelastic dynamic analysis. The first of these buildings was a 14 storey shear wall building with 2 basement levels. A lumped mass computer model was developed with a plastic hinge zone near the base of the wall where any flexural or shear plastic "hinging" is assumed to occur.

Initially the diagonal members of the shear "hinge" were modelled as elastoplastic "truss" elements and the displacement at ground flood level at the initiation of yield, Δ_y, was varied with the yield strength of the shear hinge (i.e. Δ_y = 1.25 x $V_p h_c/M_p$ mm). However, most of the analyses were carried out using a model for the shear plastic hinge that had only 30% of the shear strength provided by elastoplastic elements and the remaining 70% provided by diagonal elements that yielded elastoplastically in tension but buckled in compression. Therefore, the buckling elements behaved like yielding cross bracing rods in a frame and were intended to model the behaviour of yielding horizontal reinforcement in the wall and produce the expected "pinched" hysteric behaviour. In this model, the initial yield displacement of the shear plastic "hinge" was set to 4mm.

The model was analysed using the inelastic dynamic analysis program DRAIN-2DX Version 1.01. The program uses time step-by-step numeric integration to perform inelastic dynamic analysis.

Details of Variables Investigated

The principal variables selected for study were the earthquake ground motion, flexural plastic hinge strength, M_p, and the ratio of shearing plastic hinge strength, V_p, to flexural plastic hinge strength, M_p.

The first 10 seconds of three recorded earthquake motions were selected as the first principal variable to be used for the inelastic dynamic analysis of the wall. The El Centro N-S 1940 motion was selected because it has a similar spectral intensity to the elastic design spectra for Wellington in the NZ Loadings Code (NZS 4203, 1992). The other two earthquake motions, Pacoima Dam S16E 1971 and Imperial Valley (I.V.) College N230, 1979 were selected because they both have a long acceleration pulse. This type of damaging motion is characteristic of the ground shaking recorded close to earthquake faults when the rupture of the fault propagates along the fault towards the observation site.

The second major variable examined was the strength of the flexural plastic hinge, M_p, that was assumed to be located in the wall at ground floor level. A "triangular" code distribution of load was used to calculated M_p for a given seismic design coefficient, $C_{d,}$ where C_d is defined by the relationship, $V_{code} = C_d$ x W_t, and where V_{code} is the shear at the base of the wall and W_t is the total seismic weight.

In order to evaluate the influence of the ratio of the shear and moment plastic "hinge" strengths, the expression shown in Equation 1 was selected as the third major variable where V_p is the shear plastic hinge strength and h_c is the height to the centre of the seismic load assumed to have a code triangular distribution.

$$\frac{V_p h_c}{M_p} \tag{1}$$

For the code distribution of load, simultaneous flexural and shear yielding will occur when $V_p = V_{code}$ and therefore $V_p h_c/M_p = 1.0$. If M_p is held constant and the shear plastic hinge strength, V_p, is increased (i.e. $V_p h_c/M_p$ >1.0), yielding in shear will only occur when the centroid of the dynamic load is lowered by higher modes as indicated in Figure 1c. Similarly, if the shear plastic hinge strength, V_p, is reduced (i.e. $V_p h_c/M_p < 1.0$) yielding in flexure will only occur when the centroid of the dynamic loads is raised by higher modes as indicated in Figure 1f.

This behaviour may be compared with the behaviour of a Single Degree of Freedom (SDOF) structure. In this case, all the yielding would be in the shear mode when the shear/moment strength ratio $V_p h_c/M_p$ is less than 1.0 and all the yielding would be in the flexural mode when the ratio is more than 1.0.

RESULTS OF ANALYSIS

Displacement Characteristics of Wall

Figure 2 shows the results of the analysis using the model that had 70% of the shear plastic hinge strength provided by buckling elements. In this case, the El Centro N-S 1940 earthquake motion was used for the analysis. As a means of comparison, the shear displacement plot at ground floor level from the initial analyses using only elastoplastic shear yielding elements in the shear plastic "hinge" is also shown dotted in Figure 2. The plot for the flexural plastic hinge component of the displacement was virtually unchanged from the initial analysis and is therefore, not repeated.

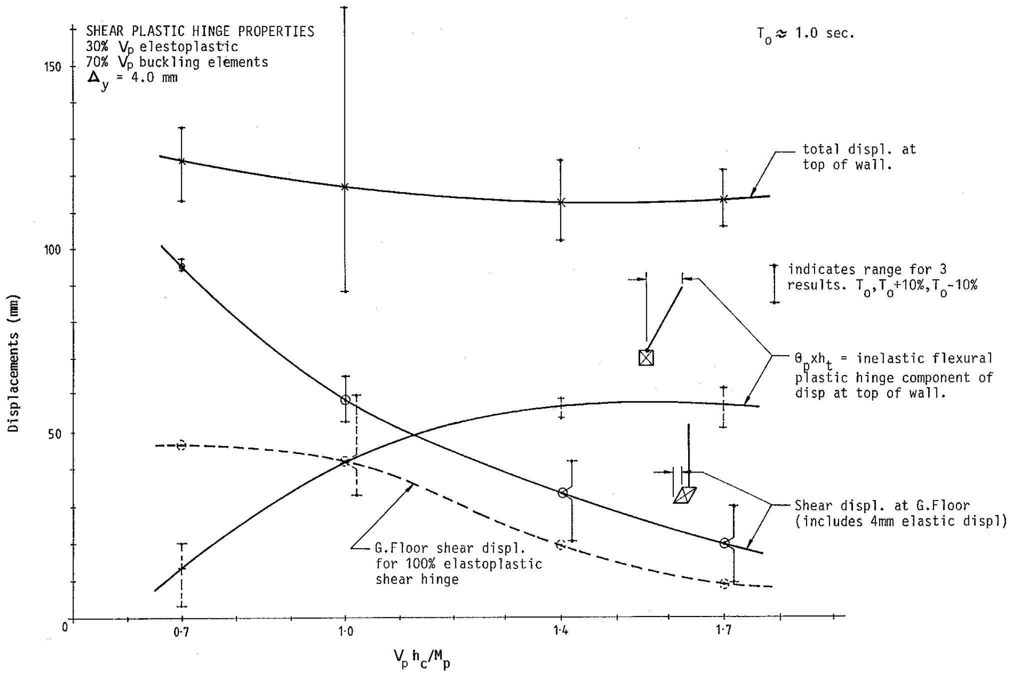


Figure 2 Peak displacements of wall with a Constant Flexural Plastic Hinge Strength, M_p computed for C_d = 0.1 and Variable Shear Hinge Strength, V_p, responding to El Centro N.S. 1940 EQ motion with 5% damping

To obtain these results, the flexural plastic hinge strength, M_p, was held constant (i.e. C_d = 0.1) and the shear plastic hinge strength, V_p, was varied so that the ratio $V_p h_c/M_p$ had a value of 0.7, 1.0, 1.4 or 1.7. Preliminary analysis established that the results were not sensitive to a small change in the ratio $V_p h_c/M_p$ but they were found to be sensitive to small changes in the initial elastic stiffness assumed for the wall.

Therefore, for each value of $V_p h_c/M_p$ considered, the analysis was repeated with the wall stiffness varied so that it's initial elastic period of vibration, T_o = 1.0 sec varied by ± 10%. The results plotted are the average values obtained from the three analyses. The three results averaged, were the peak displacement values and these did not necessarily occur at the same time during the earthquake record and, in some cases, did not even have the same sign. The range of the three results is also indicated in Figure 2.

The results from these analyses show that the distribution of inelastic deformation between the shear and flexural modes changes relatively "slowly" with changes in the ratio $V_p h_c/M_p$. When this ratio is less than about 0.7, shear yielding provides almost all of the inelastic deformation. The plotted results also indicate that significant inelastic shear displacements (e.g. 20 mm or more) could still be expected for ratios of $V_p h_c /M_p$ up to 1.7.

The two sets of curves in Figure 2 for the shear displacement at ground floor have a similar form for $V_p h_c/M_p$ > 1.0 but have a quite different form for $V_p h_c/M_p$ < 1.0. The difference in form is not thought to be due to the change in shear hinge modelling alone as all the sets of shear displacement curves plotted during the study had one of these two characteristic forms.

As can be seen from Figure 2, the inclusion of pinching into the shear displacement model, but not in the flexural model, has preferentially increased the inelastic shear displacements. As a result, the inelastic shear displacements exceed the inelastic flexural displacements up to a value of V_ph_c/M_p of approximately 1.1. This bias towards inelastic shear displacements was noted in all subsequent analyses in which the "pinched" shear model was used.

The effect of doubling the flexural and shear plastic hinge strength of the wall (i.e. increasing C_d to 0.2) was also investigated. As expected, doubling the wall strength significantly reduced the inelastic displacement demand for both shear and flexural yielding but did not change the general form of the results.

Similar analyses were also carried out using the Pacoima S16E earthquake record. The form of the results is similar to that shown in Figure 2. Generally, the increased intensity of this earthquake motion increased the total inelastic demand without changing the general form of the results. The relatively high displacement ductility demand induced by the Pacoima record was also evident by comparing the combined flexural and shear yield displacement with the total displacement at the top of the wall. The effect of doubling the wall strength was also similar to that previously described for the analysis using the El Centro earthquake motion.

The results obtained when using the I.V. College earthquake record are also similar to those obtained from the analysis using the Pacoima S16E record for the same wall shear and flexural plastic hinge strength. However, the fall off in shear yield displacement with increasing shear plastic hinge strength was found to be more rapid for the I.V. College earthquake motion.

A second eight storey building was also analysed with a shorter initial elastic period of 0.5 secs. The increased stiffness was found to reduce the "elastic" component of the walls displacement. However it did not reduce the inelastic displacement demand as expected. This could be explained by considering the detailed shape of the elastic displacement response spectra corresponding to the earthquake motions used for the inelastic analysis. Generally, the form of the shear and flexural displacement plots were still similar to those shown in Figure 2.

Shear Yielding Response of Wall

To examine the shear yielding behaviour of the wall in detail, the wall's response to the Pacoima S16E earthquake motion was selected for further study. In particular, the wall analysis with a flexural plastic hinge strength, M_p, computed using $C_d = 0.2$ and with a shear to moment plastic hinge strength ratio, V_ph_c/M_p, of 0.7 was chosen for detailed examination.

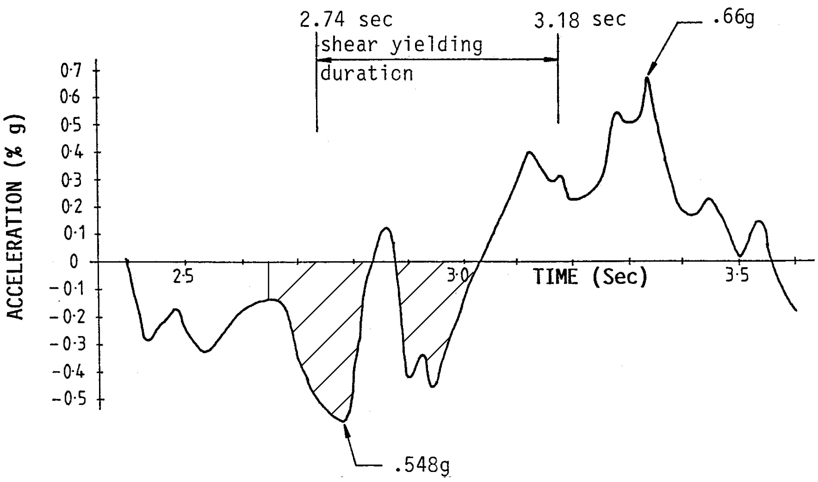
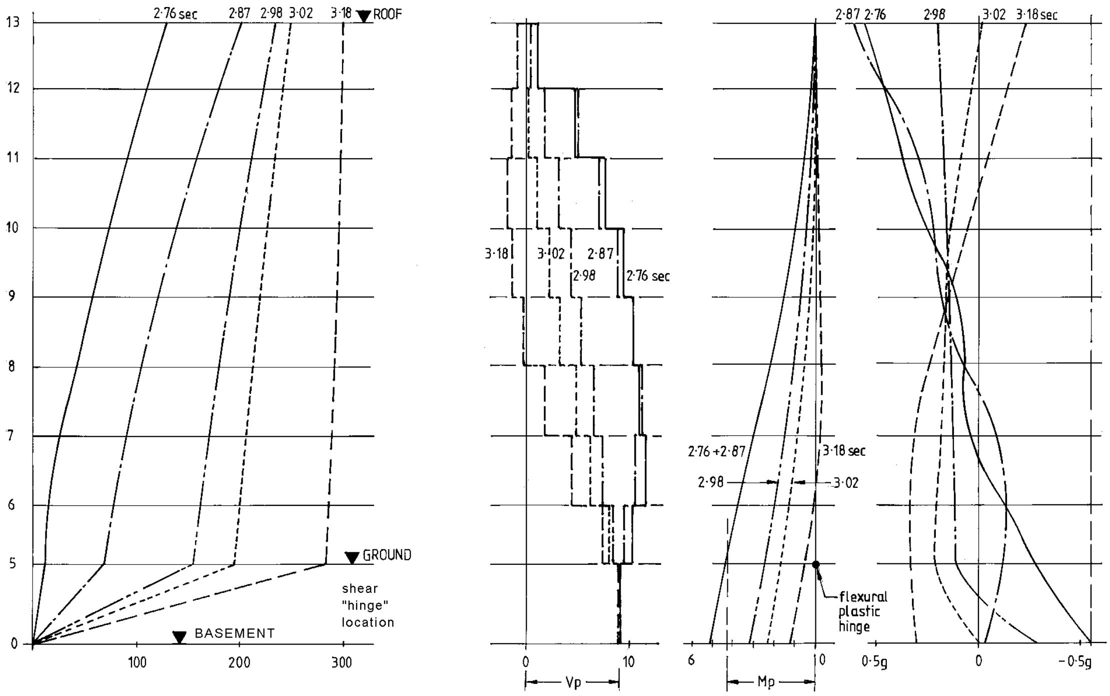


Figure 3 Long acceleration ground motion pulse of Pacoima S16E record

Most of the walls shear displacement in the positive direction was found to occur between 2.74 and 3.18 seconds after the start of the earthquake record. This period during the motion corresponds to the first part of the long acceleration pulse (i.e. "near fault fling") shown plotted in Figure 3. The shear yielding displacement was generated by the first part of the pulse shown shaded. It is interesting to note that the peak ground acceleration during this part of the motion is only 0.548g. This is less than half the peak ground acceleration of 1.17g that occurs at 7.72 seconds from the start of the motion. However, this high acceleration only lasts for a very short period (i.e. is a spike) so that its effect on the building response is hardly discernible when time history plots of the displacement, bending moment and shear force in the walls plastic hinge zones are examined.

“Snap shots” of the walls response during the shear yielding period between 2.74 and 3.18 seconds are shown in Figure 4. Displacement, shear, bending moment and dynamic load profiles over the height of the wall during this period are shown in Figures 4a to 4d respectively. The first snap shot is at 2.76 seconds, which is just after the start of yielding in shear and is at the onset of flexural yielding.



(a) Deflection (mm) **(b) Shear (x10 kN)** **(c) Bending mom (x10^{5}kNm)** **(d) Dynamic loads**

Figure 4 Shear yielding response of wall to Pacoima S16E pulse between 2.76 secs. and 3.18 secs. – 5% damping

Figure 4d shows the dynamic loads acting on the wall. These were derived by dividing the difference between adjacent interstorey shears (i.e. dynamic forces) by the seismic weight assumed to act at each floor. They are therefore expressed in terms of acceleration units. At ground level, the wall acceleration is the same as the ground acceleration. The values plotted in Figure 4d were, therefore, obtained from Figure 3. These dynamic loads can be seen to depart significantly from the “triangular” code distribution.

At 2.87 seconds, the flexural yielding has just finished but the bending moment at the plastic hinge has not fallen significantly below yield as shown in Figure 4c. During the time interval between 2.76 and 2.87 seconds, the shear yield displacement is 59 mm while the displacement at the top of the wall generated by flexural yielding is 21 mm. During this time interval, the dynamic loads shown in Figure 4d do not change significantly except at the base of the wall where the ground acceleration falls to nearly zero. Consequently there is very little change in the bending moment and shear force distribution over the height of the wall as indicated by Figures 4b and 4c. It is interesting to note that the shear force remains above the base shear yield force, V_p, over most of the wall height during this time interval. In the real wall, where shear yielding is not confined to the base of the wall and shear strength typically declines over the height of the wall due to a fall off in axial load and shear reinforcement, shear yielding would be expected in the upper parts of the wall. This would probably reduce the shear yielding displacement demand at the base of the wall.

It is also interesting to note that the peak acceleration reached at the top of the wall is 0.6g. This is three times the lateral load coefficient, C_d = 0.2, required to cause flexural yielding and over four times the coefficient, C_d = 0.14 required to cause shear yielding for a triangular code distribution of load. In other analyses, ratios of peak acceleration to C_d (x g) up to 9 were noted at the top of the walls for both shear and flexurally yielding walls.

The NZ Loadings Code (NZS 4203, 1992) assumes this ratio is approximately two when computing seismic loads for parts and portions located at the top of buildings.

As noted previously, shear yielding continued for approximately 0.4 seconds. A wall with a 1.0 second first mode period can be expected to have an elastic 2nd mode period of approximately 0.2 seconds. If the postulated mechanism of higher modes allocating inelastic demand between shear and flexural yielding was to hold during the 0.4 second yielding period, the flexural hinge bending moment would be expected to reach two peaks during the 0.4 seconds. However, Figure 4c indicates that the flexural moment at the plastic hinge location declined throughout the shear yielding period.

An explanation for the almost uniform decline in the wall moment evident in Figure 4c, is that the long period of shear yielding partially isolated the wall above ground floor level. As a result, there was no excitation of the higher modes. This behaviour can be seen from the dynamic load profiles shown in Figure 4d. At the start of shear yielding (T = 2.76 sec), the effect of the higher modes can be clearly seen in the wall's dynamic load profile. This is still apparent at T = 2.87 sec, but from 2.98 sec to 3.18 sec, the slope of the dynamic load profile up the wall is almost constant above ground level indicating that the higher mode part of the response is not being excited and has largely been damped out.

Another aspect of the behaviour apparent from Figure 4d is that the ground acceleration itself can significantly influence the shape of the dynamic load profile and hence the height of the shear centroid, particularly during long acceleration pulses such as illustrated in this example of shear yielding behaviour.

At the end of the shear yielding period (3.18 seconds), the walls elastic deflection between ground and roof level was only 1.0 mm. The visual impression given by Figure 4a, is that the energy stored elastically in the wall at the start of shear yielding (at 2.74 seconds) has been converted into a shear yielding distortion by the end of the shear yielding period. It was noted that for $V_ph_c/M_p = 0.7$ relatively large shear yielding and total displacements at the top of the wall occurred when the bending moment in the flexural plastic hinge fell to near zero at the end of the principal shear yielding period. Conversely, it was observed that when the bending moment rose during the principal shear yielding period, the shear yielding displacement tended to be relatively smaller. This may explain the two distinct forms of shear displacement curves illustrated in Figure 2 for low V_ph_c/M_p ratios.

After the initial long acceleration pulse examined in detail above, the time history response for the wall indicated that the pinched shear displacement historesis loops had the effect of limiting the forces developed in the wall. It was also noticeable that once the "initial slackness" (i.e. pinching) in the shear response had developed, all subsequent inelastic deformations occurred by shear "yielding". Even after the slackness had been taken up, the moments at the base of the wall did not even reach 0.7 times M_p. This means that even if the flexural strength had degraded to be only equivalent to the shear capacity (i.e $M_p = V_ph_c$), there would not have been any further flexural yielding.

It may be concluded that the inelastic shear displacement is not directly caused by higher modes. This was confirmed by examining the period of the motion observed in the time history plots of the moment and shear in the plastic hinge zones. These plots indicated that the inelastic yielding is principally the result of the first mode response. The higher modes appear to act principally as a "gating" mechanism to allocate the inelastic demand between the shear and flexural yielding options. However, this is an over simplified explanation of the walls yielding behaviour during relatively long periods of shear yielding. During these periods the ground acceleration and the wall's own inelastic response modify the dynamic loads.

OTHER WORK

An investigation was carried out on the use of elastic response spectra to predict inelastic displacements in walls of the type described in this paper. It was concluded that an approximate estimate of the inelastic displacement demand in a wall that is yielding in a combined flexural and shear mode could be obtained from a SDOF elastic response spectra. In addition, an investigation was carried out to predict the shear yield displacement capacity and strength of walls that fail in shear. These additional investigations were part of the original study on which this paper is based (Blaikie *et al*, 1990), but are beyond the scope of this paper.

CONCLUSIONS

The results of the inelastic dynamic analysis indicate that the total displacement at the top of the wall and its inelastic component are not particularly sensitive to whether the yielding takes place in the shear or flexural mode. However the proportion of the total inelastic demand that takes place in the shear or flexural mode is sensitive to the shear/moment plastic "hinge" strength ratio, $V_p h_c / M_p$. When this ratio is less than about 0.7 almost all of the inelastic demand is in the shear mode and when the ratio is greater than 1.7 most is in the flexural mode.

The inelastic dynamic analysis results also indicate that the shear or flexural yielding is principally the result of the 1st mode response of the wall. The primary role of the higher modes is to allocate the total inelastic demand between the shear and flexural yielding modes. Although this is a useful conceptual framework within which to view the role of higher modes, a detailed examination of the shear yielding behaviour of one of the walls indicated that it is an over simplification. During relatively long periods of shear yielding, the dynamic lateral loads acting on the wall are generated by a complex interaction of ground accelerations and the modal responses of the wall which are in turn, modified by the wall's inelastic response.

It was observed that when the wall was predominately deforming with shear yielding at the base of the wall, shear forces higher than those developed at the base of the walls were present above the mid height of the wall. There were also relatively high floor accelerations generated at the top of the walls for both shear and flexurally yielding structures.

The "form" of the results expressed as a characteristic shape of the plotted curves of peak wall displacements, was insensitive to a number of variables examined. However, there were local departures. Increasing the walls flexural and shear plastic hinge strength by the same amount, increased the "elastic" component of the wall's peak displacement and reduced its inelastic component as would be expected. Increasing the intensity of the earthquake motion increased the total inelastic demand without changing the general form of the results.

The draft guidelines for the "Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings" (NZNSEE, 1996) state that there is no need to allow for dynamic magnification of shear forces because of the "comparatively short duration of dynamically amplified shear action". Clearly this recommendation needs to be reconsidered in light of the research presented in this paper.

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