SEISMIC RESPONSE OF SHEAR WALL-FRAME SYSTEMS

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SYNOPSIS

The inelastic seismic response of shear walls coupled to frames is investigated by means of time history analyses. The effects of various wall to frame stiffness ratios, wall yield moments and earthquake intensities are considered. For the configurations analysed, yielding takes place in the lower half of the wall but the steel frame remains elastic throughout. The primary effect of yielding of the wall is to transfer loads to the frame increasing the corresponding shear ratio above that predicted by linear static analysis. On the other hand the change in wall moment distribution is less sensitive to inelastic action. The distribution predicted by an isolated wall static analysis is conservative except for the case of a very flexible wall where it is thought that higher mode contributions lead to greater moments close to the top of the wall. The values of base shear computed by inelastic dynamic analyses are in all cases considerably greater than the elastic dynamic results reduced by the appropriate system ductility factor.

RESUME

La réponse sismique inélastique des murs de cisaillement accouplés aux portiques est étudiée au moyen d'analyses incluant le paramètre temps. Les effets de différents rapports de rigidité mur/portique, des moments plastiques du mur et des intensités du tremblement de terre sont considérés. Pour les configurations étudiées, l'écoulement apparait en bas du mur alors que le portique en acier demeure complètement élastique. L'effet principal de cet écoulement est de transférer les charges au portique augmentant ainsi le rapport de cisaillement correspondant au-dessus de la valeur prédite par l'analyse statique linéaire. D'autre part un changement dans la distribution des moments dans le mur est moins sensible à l'inélasticité. La distribution prédite par une analyse statique d'un mur isolé est conservatrice excepté dans le cas d'un mur très flexible où l'on pense que les contributions des modes supérieurs augmentent les moments en haut du mur. Les valeurs du cisaillement à la base calculées au moyen d'analyses dynamiques inélastiques sont dans tous les cas beaucoup plus grandes que les résultats dynamiques élastiques réduits par le facteur approprié de la ductilité du système.

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INTRODUCTION

The use of shear walls as the primary bracing system of medium to high rise moment resistant frames has proved to be very cost effective in recent years. In zones of low seismic risk where drift due to wind loads governs the design, the lateral stiffness of shear walls is of critical importance. On the other hand experience gained in the Managua earthquake (1972) has shown conclusively that not only do shear wall structures possess sufficient strength and ductility to survive major earthquakes, they are also capable of doing so with a minimum amount of structural and nonstructural damage[1].

The superior seismic performance of shear wall structures is due to a variety of reasons of which their considerable stiffness is probably the most important. It is also noted that the section ductility demands on shear walls are often not as severe as for unbraced frames particularly if hinging occurs in columns. This is almost inevitable at the frame base. Furthermore, the achievement of the required local ductility is more reliable for wide shear walls than for other concrete members where the congestion of steel reinforcement leads to situations in the field that differ significantly from that visualized by the designer.

The interaction between shear walls and moment resistant frames under static loads has been the topic of considerable research over the past 20 years or so. This has lead to an understanding of the distribution of lateral forces between walls and frames, which frequently forms the basis for design. Analysis to account for interaction was facilitated initially by simplified procedures suitable for hand calculations (e.g. reference 2) and more recently by the availability of general computer programs. These elastic methods predict reduced bending moments in the lower part of the shear wall, leading to more economic designs. However, if the structural behaviour extends into the inelastic range the beneficial effect of interaction is reduced[3]. As inelastic response is implicit in most analytical procedures that account for seismic effects, the question arises as to the extent to which the beneficial aspects of interaction should be utilized in a design where seismic loads are significant. This is the main question addressed in this paper.

It should be noted that most building codes in North America prescribe a procedure by which 100% of the lateral load is assigned to the shear walls and 25% to the frames acting independently in an attempt to provide an alternative load path if failure of the wall(s) is reached. This procedure can, to a limited extent, safeguard against the consequences of a reduction in interaction effects caused by inelastic action. However, this procedure may not be general enough to cover all possibilities especially for a flexible wall-stiff frame system where because of interaction the frame could be taking more than 25% of the lateral load. Furthermore the distribution of frame shears is described rather poorly by the '100-25%' approach. The suitability of this procedure is investigated in this study.

Clough and Benuska[4] used nonlinear time history analyses to study the seismic response of shear wall-frame steel structures. The effect of period of vibration, stiffness ratio and building height were investigated especially with respect to the ductility demands on the various members. The implication of various design assumptions on the distribution of lateral shears was also studied. The main emphasis of the study is placed on the inelastic action in the frame with almost no yielding in the wall. This is in contrast to the work reported herein where the inelastic action is restricted entirely to the shear walls for the designs considered. The primary reasons for this are the higher wall stiffnesses, and lower percentage of live vertical load used in this study.

The main objective of the present study is to evaluate the change in wall-frame interaction as a result of cyclic inelastic action in the concrete shear wall by means of nonlinear time history analyses. Emphasis is placed on how the changes in interaction affect the most significant internal forces - the wall bending moments, and the frame storey shears. The ductility demands placed on the structure as a whole and on individual components are also studied. Variables considered in the investigation include the wall to frame stiffness ratio, the wall yield moment, and the intensity of the earthquake.

STRUCTURE UNDER INVESTIGATION

The basic design used in this study consists of a 10 storey frame coupled to a shear wall. The building under consideration consists of seven frames and two shear walls and corresponds approximately with that used by Goldberg[5] in a static interaction study. The frames and shear walls are located symmetrically so that there are no torsional effects. Coupling is achieved by the diaphragm action of the rigid floor slabs. The width of the shear wall is varied from 18'6" to 31'8" to correspond with a range in stiffness ratios from 20 to 100. (Stiffness ratio, SR, is defined as $\Sigma(EI)_{wall}/\Sigma(EI)_{col}$).

Goldberg's three bay frame is approximated by a one bay frame in the present study as shown in Figure 1. Implicit in this approach of halving the interior column stiffnesses is that the exterior columns in the prototype structure are half as stiff as the interior ones. It is in this respect that the present model deviates from Goldberg's and probably corresponds closer to realistic designs. Because of the use of seven frames in conjunction with two shear walls the column stiffnesses herein are $3\frac{1}{2}$ times Goldberg's values and the shear wall stiffness is reduced by 2/3. The column stiffnesses are varied linearly between the top and bottom storeys whereas the girder values are constant throughout. The values of area and plastic moment follow corresponding distributions.

In the present study the values of column and beam plastic moments are fixed for all analyses. The reason for this is that the actual members used are controlled by vertical loading which is not varied. Thus changes in wall to frame stiffness ratio are achieved by varying the wall width. The wall yield moment can be varied independently of the wall dimensions and herein is computed on the basis of a rational analysis. The approach adopted is to analyse the coupled system statically for seismic loads according to SEAOC provisions[11]. The base moment arrived at in this way is referred to as M_{st} which will of course vary for different stiffness ratios. The value of base yield moment generally used thereafter is 1.4 M_{st} to be consistent with ACI procedures[7]. However, this factor is varied between 0.67 and 2.0 in one parametric study. The value of moment capacity is kept constant over the height of the wall in all analyses. The main reason for this is to keep the number of variables within reason and to facilitate the interpretation of the results. It so happens that the minimum reinforcement requirement of ACI comes into play a short distance up the wall so that there is not much flexibility for varying the yield moment realistically.

ANALYTICAL PROCEDURE

The analyses reported herein are nonlinear time history analyses. The nonlinearities stem from yielding of the concrete shear walls*. Changes in geometry are not considered and therefore P - Δ effects are neglected. Even though this latter phenomenon leads to a reduction in the effective stiffness of the frame, it is not very significant for the present study because of the relatively low values of column axial forces.

The computer program used for the analyses is DRAIN-2D[8] which utilizes a constant acceleration algorithm for the integration of the equilibrium equations. The accuracy of this algorithm is enhanced by the use of a corrector which applies the out of balance loads from the last time step to the present one. Some numerical difficulties were encountered with the program but were overcome by reducing the number of freedoms at a floor level to a minimum-four: one lateral translation, and the rotations of the wall and the two columns respectively. A time step of 0.01 seconds was used for all analyses. This was

* Yielding of the steel frame members was included in the analytical model but never occurred during the analyses.

checked for one analysis by using 0.005 secs which gave results differing by only a few percent from that obtained with the larger increments.

The ground motion used is that for El Centro (1940), North-South component. The fact that this accelerogram is reasonably symmetric renders it particularly useful for nonlinear studies. The use of artificially generated accelerograms was also attempted but proved to be difficult because the stability problems mentioned earlier were aggravated. This is probably due to the increased rates of change of acceleration as compared to El Centro. This problem dictated the use of El Centro ground motion throughout the study. Analysis was conducted on the first 8 seconds of that accelerogram. Although cumulative ductility factors are affected by restricting the duration of the ground motion, other results are not affected significantly. This was verified for one particular geometry where considerable inelastic action was experienced.

The steel members in the system are modelled as elasto-plastic with a strain-hardening ratio of 0.00001. This behaviour is included in the computer program by means of elastic hinges at the ends of the members, and it is the stiffness of these hinges that is modified to reflect the inelastic action in the members.

The concrete shear wall is modelled in a similar way except for the presence of strain hardening and some complexity in behaviour on load reversal. The well known Takeda model[9] is used in modified form which allows for a reduction in unloading stiffness based on the extent of inelastic action as shown in Figure 2. A value of 10% is used for the strain hardening ratio on the basis of the experimental results reported by Oesterle et al[6]. This value of 10% refers to the moment rotation plot for a cantilever beam (see Figure 3) subjected to loads corresponding approximately to that which would be experienced by the shear wall in the first storey. The variation of the hinge stiffness k_s is computed to give the required strain hardening.

Rayleigh type damping is used in the structural model, with proportionality constants α and β selected to give approximately 5% of critical damping over a 0.5 to 1.5 sec period range. The tangent stiffness is used throughout for computing the stiffness proportional contribution to the damping matrix.

NUMERICAL RESULTS AND DISCUSSION

This discussion is arranged in sections, each relating to the effect of a particular parameter on the response of the system. The variables considered are stiffness ratio, wall yield moment and earthquake intensity.

In presenting the results of time history analyses there is a dilemma with regard to which quantities to display because of the variations in time. In this study root mean square* (rms) values are

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* In this context, rms was evaluated by integrating the parameter with respect to time.

generally presented unless stated otherwise. The use of rms quantities facilitates the interpretation of the results because their variations are smoother and therefore trends can be identified more easily. In discussions relating to ductility ratios peak quantities are more important and it is these values that are displayed in the corresponding figures.

Stiffness Ratio

To study the effect of stiffness ratio on the change in inelastic response of the wall-frame system three different walls are considered corresponding to stiffness ratios of 20, 60 and 100. The yield moment of each of these walls is 1.4 M_{st} where M_{st} is the wall base moment computed by static analysis of the coupled system. The ground motion corresponds to the El Centro accelerogram without any scaling.

The results of the three analyses for different wall to column stiffness ratios (SR = 20, 60 and 100) indicate yielding close to the base of the wall with the frames remaining elastic throughout. Figure 4(a) shows the variation of the frame shear to base shear ratio over the height of the building. Figures 4(b) and (c) are the corresponding plots for wall shear to storey shear and wall moment to wall base moment respectively. These plots show the general trend of the fraction of the loads carried by the frame decreasing with increasing stiffness ratio. This is consistent with the static response of the system as shown in Figures 5(a) and (b). The frame shear ratios as predicted by inelastic dynamic analysis are greater than the static values for 60 and 100 stiffness ratios. It has been found that there is very good agreement between static and dynamic frame shears for the stiff walls when both analyses are elastic and therefore the increase is due to the inelastic action at the base of the wall. This redistribution of forces cannot be predicted by a linear static analysis. The irregularities in the dynamic results in the bottom two storeys are in contrast with the smooth static distribution. This phenomenon is also present when a soft rotational spring is incorporated at the base of the wall in a static analysis. This would indicate that wall hinging is the cause of the irregularity.

Figure 4(c) shows the variation of the dynamic normalized wall moments over height for the three different stiffness ratios which can be compared with static values shown in Figure 5(c). The dynamic response of the most flexible wall differs markedly from the static prediction especially with respect to wall moments (Figures 4c and 5c). This discrepancy is due not only to inelastic action but also to dynamic phenomena as indicated in Figure 6 where the static, elastic dynamic and inelastic dynamic wall moments are compared for a stiffness ratio of 20. It would seem that the difference between static and elastic dynamic results is due primarily to higher mode contributions. The introduction of hinging in the inelastic analysis reduces the stiffness of the system and thus leads to an even greater contribution of the higher modes.

Shear Wall Yield Moment

Inelastic action is restricted to the bottom of the shear wall in the present study and therefore it is apparent that the value of wall yield moment is a critical parameter in the response of the coupled system. Values of this variable in the range 2/3 M_{st} to 2 M_{st} are studied for a design corresponding to a stiffness ratio of 100. The ratio of frame shear to base shear from the dynamic analyses is shown in Figure 7(a) for the different yield moments. In addition two sets of static results are shown, one corresponding to the structure 'as is' and the second corresponding to a modified structure having a rotational spring at the base of the wall with a stiffness equal to the strain hardening plastic hinge used in the dynamic model.

The frame shears converge towards the elastic static response with increasing yield moment because of the associated trend of decreasing inelastic action. The 'hinged' static solution bounds most of the dynamic results as this model represents the wall at its most flexible. However, the '2/3 M_{st}' results give even larger frame shears because this design experiences yielding over a considerable height of the shear wall, shedding an even greater percentage of the wall shear than the wall with a single hinge. The irregularity in the frame shear ratio at the bottom of the structure is particularly pronounced for the hinged system indicating that this phenomenon, as mentioned previously, is due directly to hinging at the base. These irregularities are exhibited by the inelastic dynamic results also, but tend to dampen out for the '2/3 M_{st}' structure because of the spreading of the inelastic action up the wall.

It would appear that the inelastic dynamic shear ratios of Figure 7(a) can be interpolated from the results of the two linear static models for all the designs except the rather extreme '2/3 M_{st}' case. When linear interpolation is attempted on the basis of system ductility (μ_s) excellent correlation is achieved (even for '2/3 M_{st}' case) if μ_s of 1 is assigned to the static solution for the fixed wall configuration and μ_s of 4 to the hinged wall. General conclusions cannot be drawn until more data is available especially with regard to different stiffness ratios.

The distribution of the ratio of the wall shear to the total storey shear is shown in Figure 7(b) for various values of yield moment. The ratio diminishes with the yield moment at any height because of the load shedding from wall to frame with inelastic action. However, it is interesting to note that there is an associated redistribution of the wall shears giving rise to an increase in wall shears close to the top of the structure. This trend is not present in the elastic dynamic results which show a continuous reduction in wall shear ratio right to the very top of the building. The irregularities noted for the frame shear ratio distributions close to the base of the wall are also exhibited by the wall shears.

Figure 7(c) shows the variation of wall moment to base moment over the height of the structure. The moments at the top of the wall increase with inelastic action and are consistently higher than the static prediction. This trend is reflected in the elastic dynamic results for the hinged case also, but surprisingly not for the corresponding elastic static results. The phenomenon may therefore be due to higher mode contributions to the dynamic response which become more significant for the increased flexibility associated with increased inelastic action. It should be noted however that the differences between the wall moment ratios for various levels of yield moment are quite small. Thus wall moment ratios are considerably less sensitive to inelastic action than wall or frame shear ratios for the stiff walls. Although the inelastic moments are always higher than the static values this should not cause a severe problem because of the fact that there is generally a reserve of moment capacity close to the top of the wall resulting from the minimum reinforcement design provisions.

Because of the importance of ductility in the seismic performance of structures the results relating to this phenomenon are discussed next. Figure 8 shows the variation of member ductility with height for the different wall yield moments. The definition of member ductility used in the present study is

$$\mu_m = \frac{(M_{max} - M_y)}{0.1\ M_y} + 1$$

where M_{max} is the absolute peak dynamic moment and

M_y is the design yield moment of the wall.

The 0.1 factor stems from the 10% strain hardening ratio used for the concrete wall throughout this study.

A quantity of considerable interest in simplified dynamic analysis for earthquake effects is the system ductility which is defined herein as

$$\mu_s = \frac{\text{Maximum displacement at top of structure}}{\text{Static displacement at top at first yield}}$$

Figure 9 shows that the increase of system ductility with member ductility is almost linear. It is interesting to note that to achieve a value of system ductility of 3 as suggested in the National Building Code of Canada[10] a member ductility of 6 is required at the base of the wall.

System ductility factors are commonly used to reduce seismic design forces and the justification of this procedure for the present structure is examined in Figure 10 where the peak base shear normalized by the peak value corresponding to $\mu_s = 1$ is plotted against $1/\mu_s$. Points on or below the broken line shown in the figure would reflect a conservative design. The fact that the analytical data is well above this line reflects the inadequacy of the simplified procedure. This is not surprising as the ductility factor method is best suited to a uniform distribution of inelastic action through the structure, in contrast to the present system which exhibits concentrated hinging close to the bottom of the wall.

Earthquake Intensity

Of the parameters relevant to the earthquake response of structures, the earthquake intensity is the most uncertain. It is therefore important to investigate the effect of various intensities in the present study. El Centro ground motion is used throughout, but is scaled linearly by the following factors: 0.5, 1.0, 1.5 and 2.1.

It is expected that an increase in earthquake intensity is equivalent to a corresponding decrease in the wall yield moment. This assumption was tested numerically and found to be exact. The procedure consisted of comparing the '1.4 M_{st}' design subjected to El Centro factored by 2.1 with the '2/3 M_{st}' design loaded by an unfactored El Centro. The only difference numerically in the results is that the forces and displacements from the first analysis are 2.1 times larger than those obtained by the latter.

The above conclusion is also valid for system and member ductilities. In addition it is found that there is an almost linear variation in these ductilities with respect to earthquake intensity. System ductilities of 1.3, 2.3, 3.6 and 5.1 are recorded for intensity factors of 0.5, 1.0, 1.5 and 2.1 respectively. The relation between member and system ductilities for various intensities is plotted in Figure 9 where a correspondance with varying yield moments is observed.

The results of the various inelastic analyses can be used to check the design provision[10,11] by which at least 25% of the lateral load is assigned to the frame neglecting interaction effects. The corresponding variation of frame shear to base shear would start at 0.25 at the base and decrease parabolically over the height of the building. From Figure 7(a) it is apparent that the above provision would increase the governing design shears for the first two storeys only. However, even in this zone the inelastic shears are generally higher than the 25% value.

The same design provisions as referred to above also recommend the application of 100% of the lateral load to the shear wall acting alone. This produces a moment envelope which exceeds the static results incorporating interaction. Furthermore, this envelope exceeds the inelastic results from the lower half of the wall. The extent to which the inelastic results exceed the envelope for the upper half is significant only in the case of the most flexible wall (Figure 6). Thus for the stiffer walls, the simplified procedure leads to a distribution of wall moments that are generally conservative. This differs from the frame shear distributions which are grossly underestimated, especially in the mid-height region of the structure.

SUMMARY AND CONCLUSIONS

Time history analyses have been conducted on coupled wall-frame structures with various values of stiffness ratio, wall yield moment and earthquake intensity. The structure included discrete hinges in the shear wall at the ends of elements to reflect the behaviour of a modified Takeda model. No yielding was experienced in the steel frame. The earthquake input corresponded to the N-S component of El Centro. The following conclusions can be drawn:

1. Inelastic distributions of frame shears for various stiffness ratios (20, 60 and 100) display a similar behaviour to the results of static analyses.

2. Inelastic distributions of wall moments correspond closely with the static results near the base but deviate in the upper sections. This is quite pronounced for the most flexible wall where higher mode contributions are increasingly evident.

3. The primary effect of a reduction in the wall base yield moment is to shed more load to the frames resulting in significant increases in frame shears compared to the static results.

4. Irregularities in frame and wall shear distributions close to the base arise because of hinging in the wall. This phenomenon dampens out with the spread of the hinging region.

5. Wall shear ratios decrease with increasing inelastic action over most of their height. At the top this trend is reversed because of a redistribution of interaction forces.

6. For the stiffer walls the distributions of moments are insensitive to inelastic action and can be predicted satisfactorily by accepted design procedures wherein the total lateral load is assigned to the wall.

7. Reduction of the elastic base shear by the system ductility factor gives a poor estimate of the corresponding inelastic force.

8. An increase in the earthquake intensity is exactly equivalent to a corresponding decrease in wall yield moment if the forces are appropriately scaled.

ACKNOWLEDGEMENTS

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REFERENCES

1. Fintel, M., "Ductile Shear Walls in Earthquake Resistant Multi-Storey Buildings", ACI Journal, Proceedings V.71, No. 6, June 1974.

2. Khan, F.R., and Sbarounis, J.A., "Interaction of Shear Walls and Frames", Journal of the Structural Division, ASCE, V.90, No. ST3, June 1964.

3. Clark, W.J., MacGregor, J.G., and Adams, P.F., "Inelastic Behaviour of Reinforced Concrete Shear Wall-Frame", Proceedings Eighth Congress, IABSE, New York, 1968.

4. Clough, R.W., and Benuska, K.L., "FHA Study of Seismic Design Criteria for High-Rise Buildings", Report HUD TS-3, Federal Housing Administration, Wash., D.C., August 1966.

5. Goldberg, J.E., "Analysis of Multistorey Buildings Considering Shear Wall and Floor Deformations", in Tall Buildings, edited by A. Coull and B. Stafford Smith, Pergamon Press, 1967.

6. Oesterle, R.G., Fiorato, A.E., Johal, L.S., Carpenter, J.E., Russell, H.G., and Corley, W.G., "Earthquake-Resistant Structural Walls - Tests of Isolated Walls", Report to NSF, Portland Cement Association, Skokie, Illinois, 1976.

7. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)", American Concrete Institute, Detroit, 1977.

8. Kanaan, A.E., and Powell, G.H., "General Purpose Computer Program for Dynamic Analysis of Plane Inelastic Structures - DRAIN 2D", Report No. EERC 73-6, Earthquake Engineering Research Center, University of California, Berkeley, April 1973.

9. Takeda, T., Sozen, M.A., and Nielsen, N.N., "Reinforced Concrete Response to Simulated Earthquakes", Journal of the Structural Division, ASCE, V.96, No. ST12, December 1970.

10. National Building Code of Canada, Supplement No. 4, Commentary K, 1977.

11. Seismology Committee, Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary", 171 Second Street, San Francisco, California 94105, 1975.

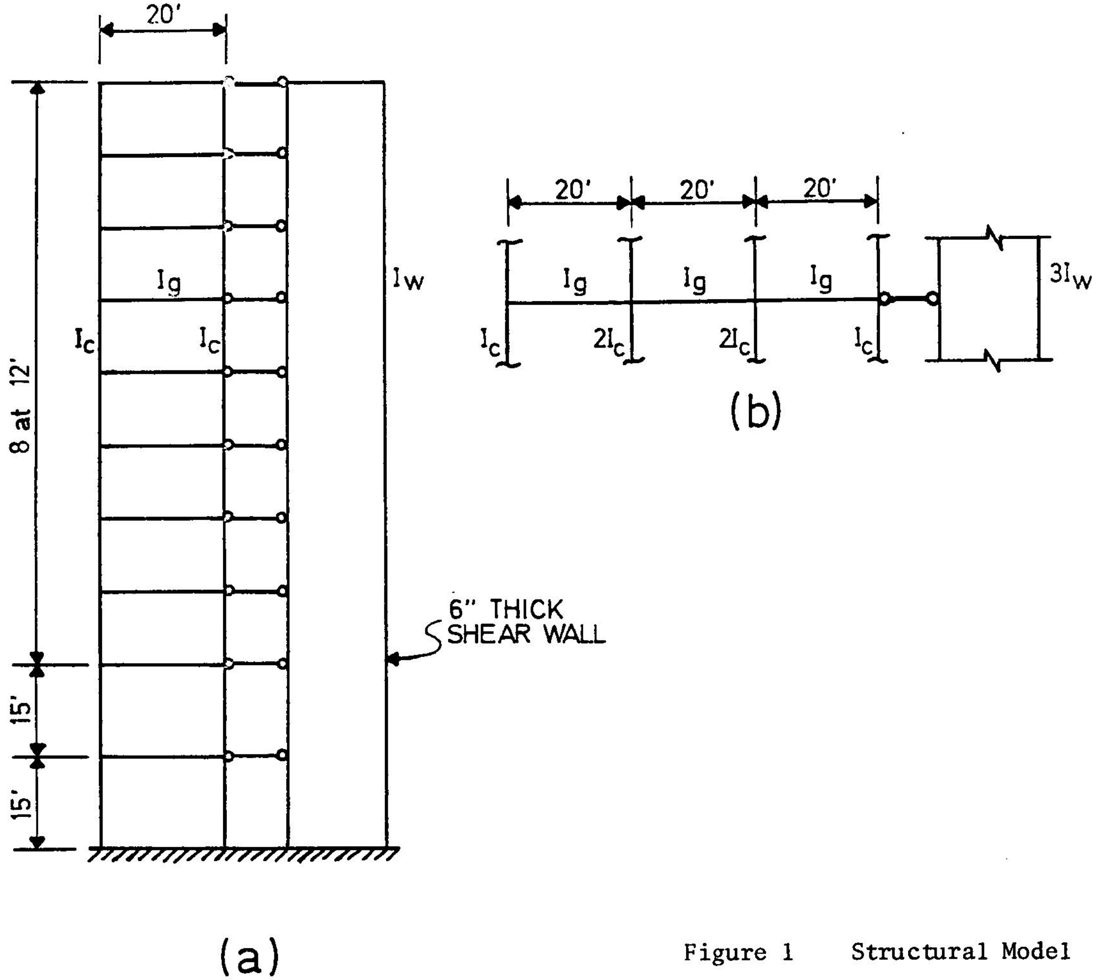


Figure 1 Structural Model

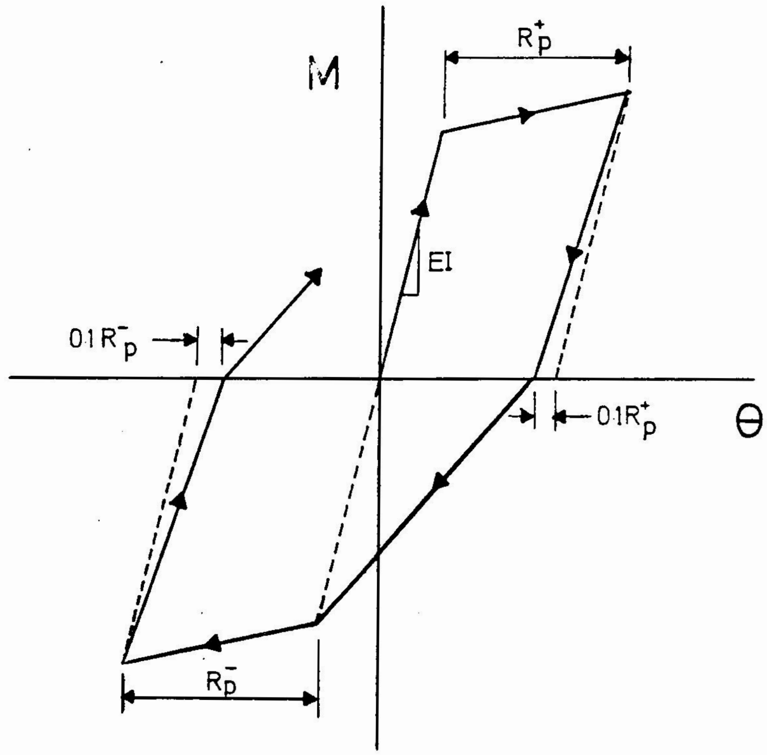


Figure 2 Hysteretic Behaviour of Wall Hinges

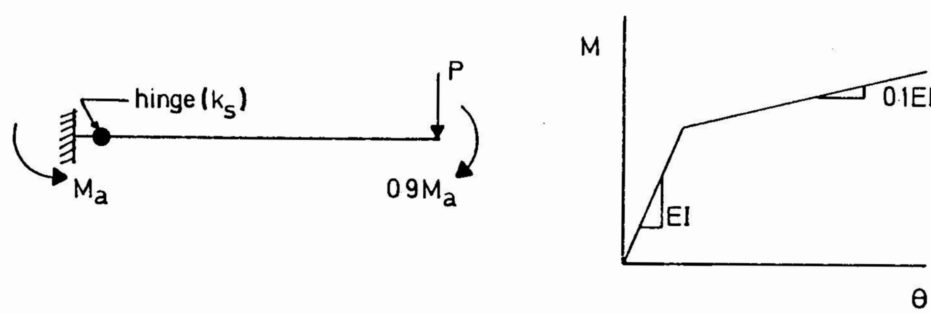


Figure 3 Strain Hardening Characteristics of Hinge

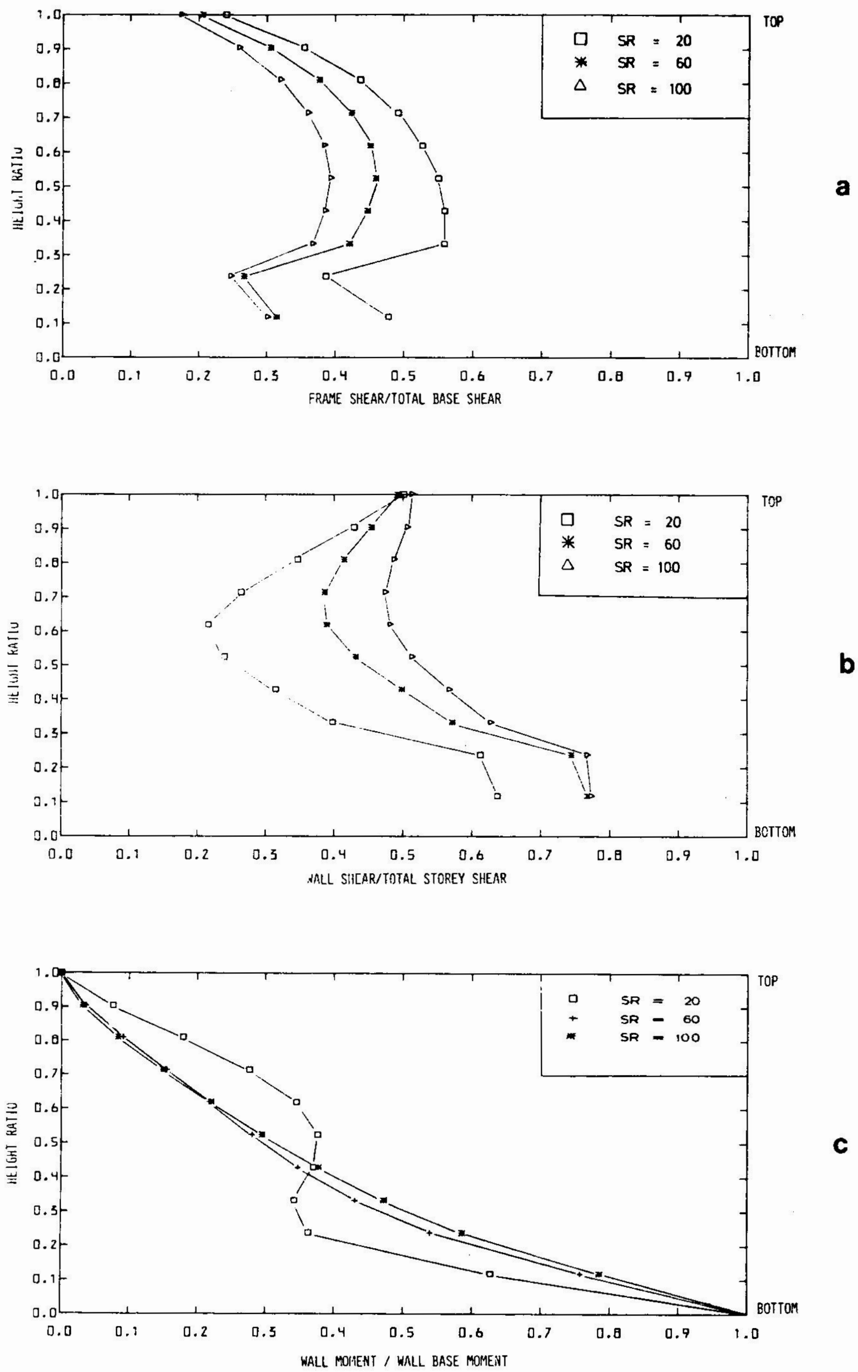


Figure 4 Dynamic Structure Force Relationships for Variation of Stiffness Ratio

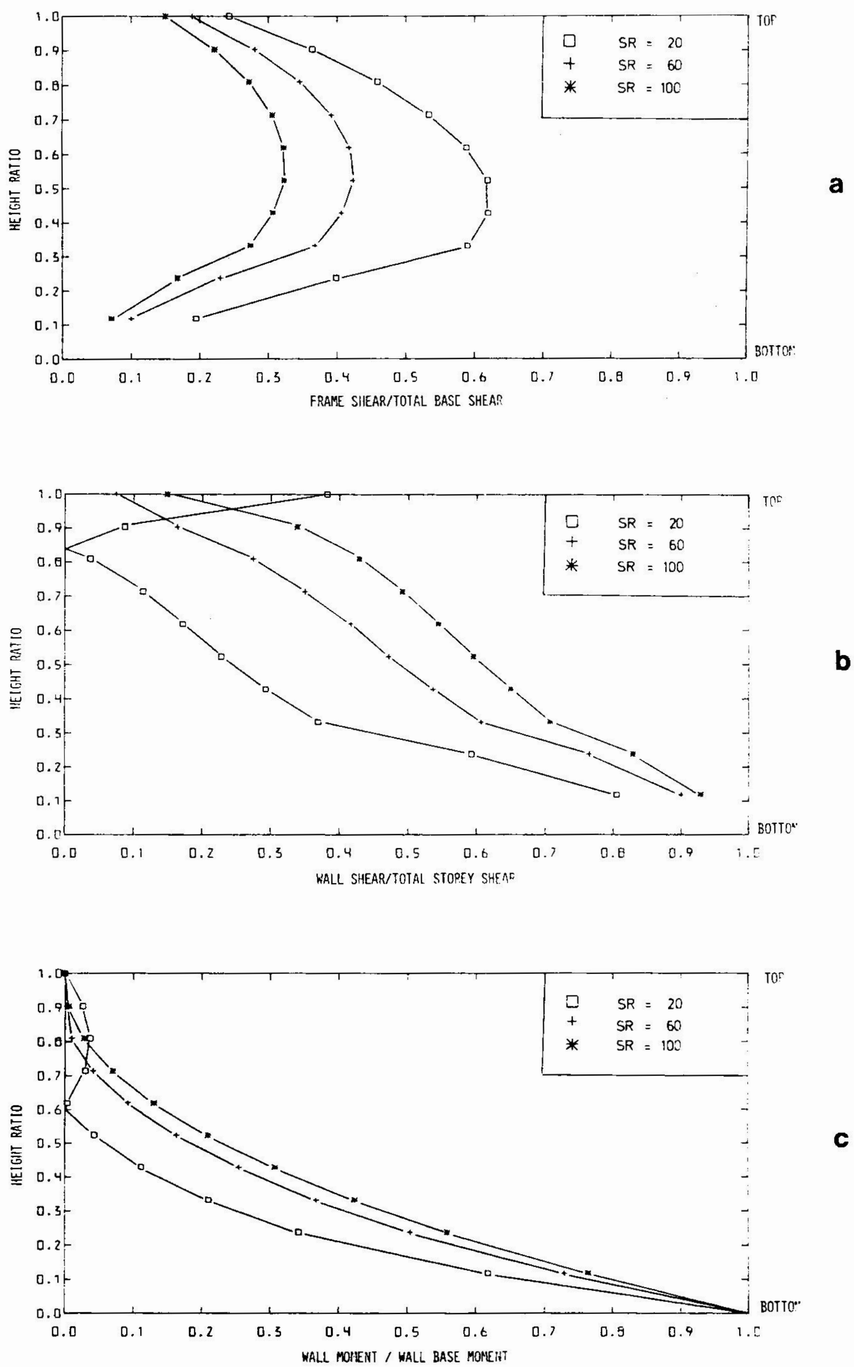


Figure 5 Static Structure Force Relationships for Variation of Stiffness Ratio

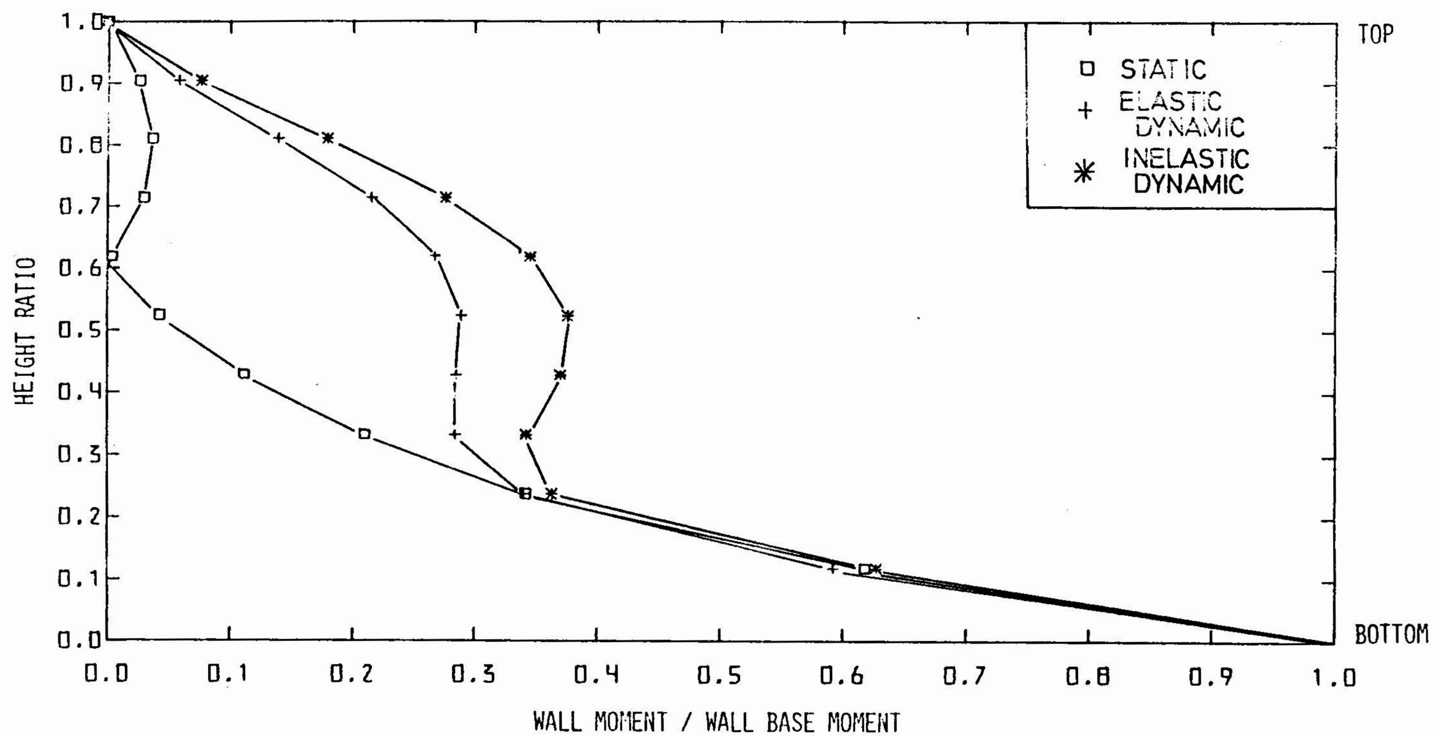
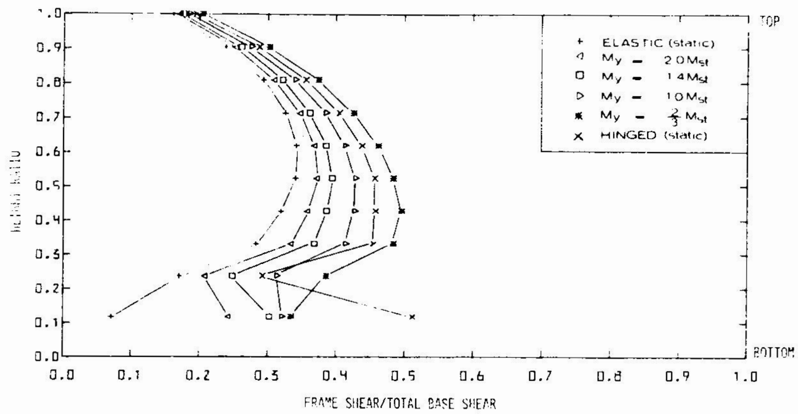


Figure 6 Comparison of Wall Moments from Different Analyses for Stiffness Ratio of 20.



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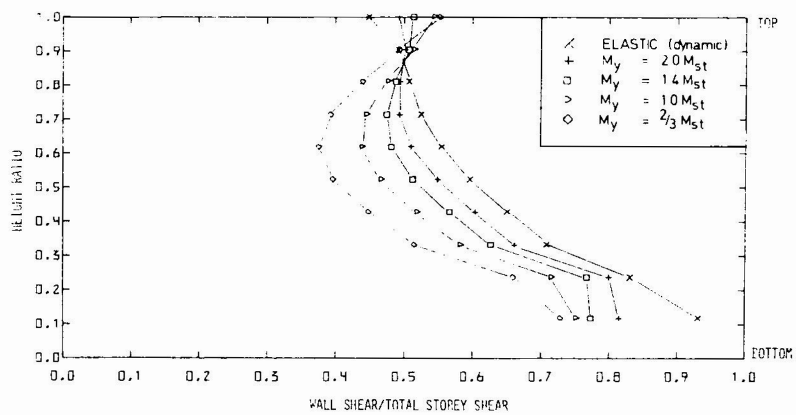

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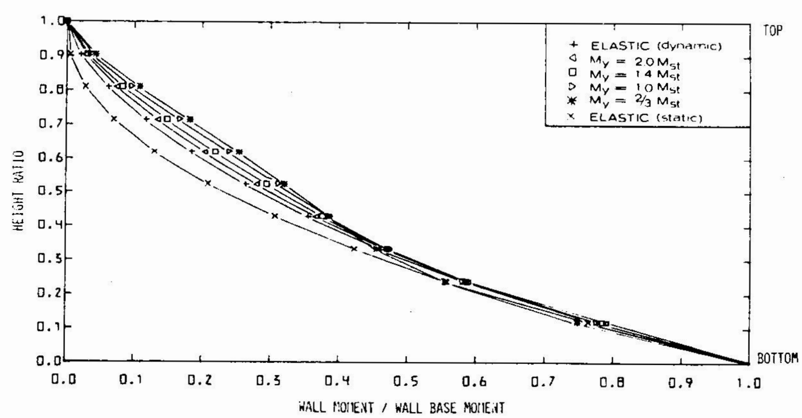

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Figure 7 Dynamic Structure Force Relationships for Variation of Wall Yield Moment

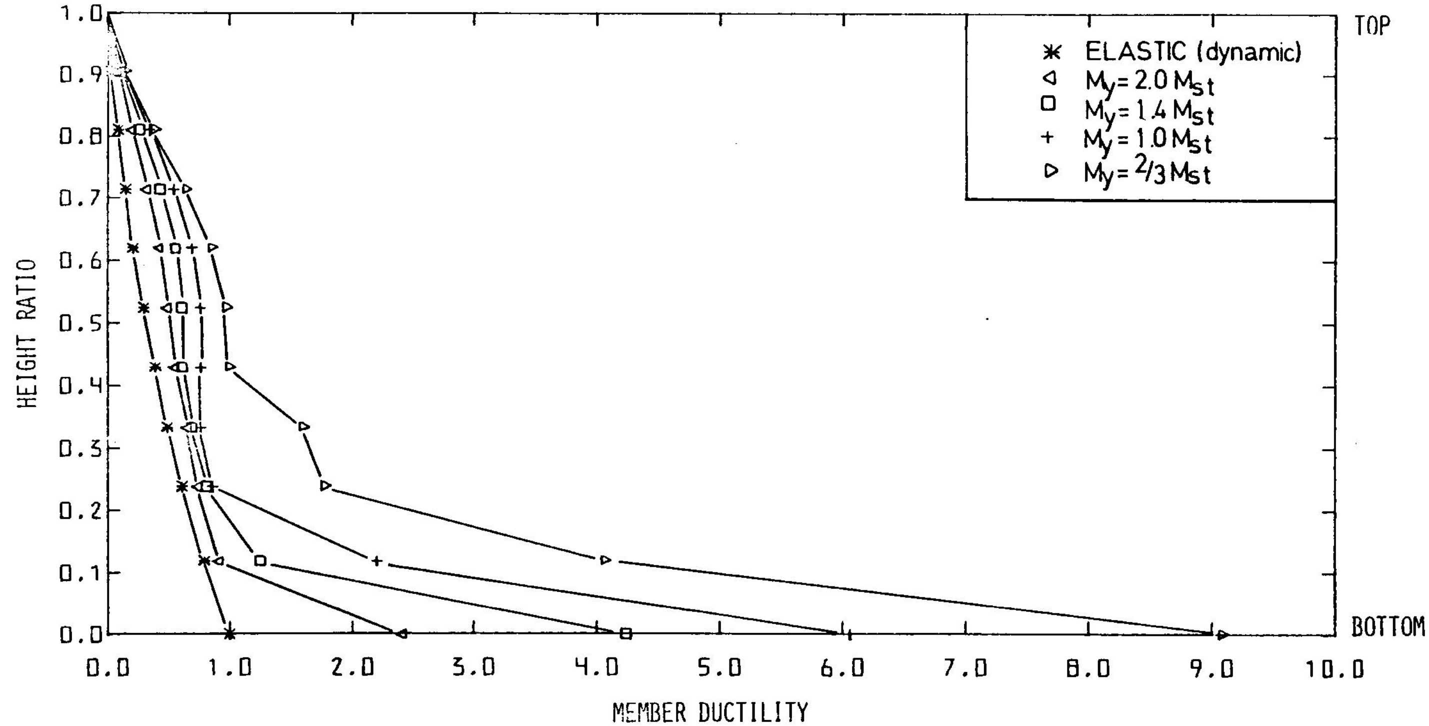

Figure 8 Shear Wall Member Ductilities for Variation of Yield Moment

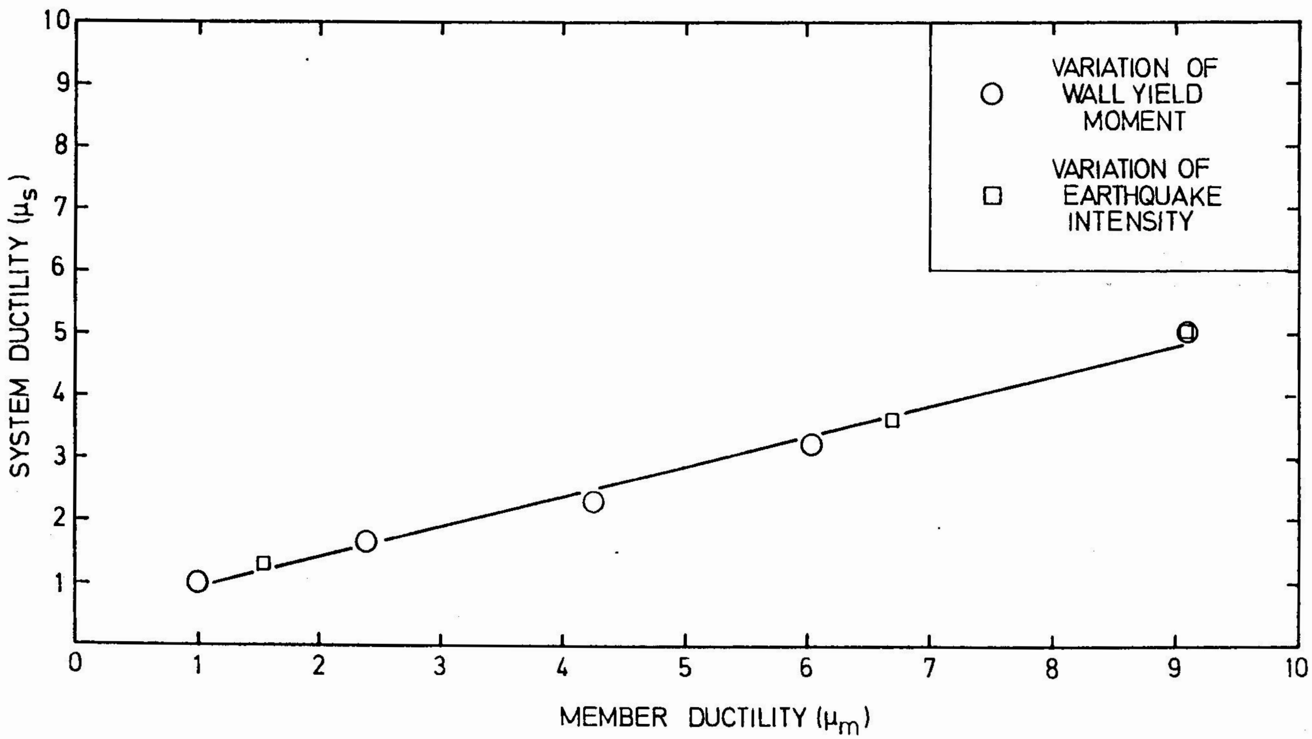

Figure 9 Relationship of System and Member Ductilities

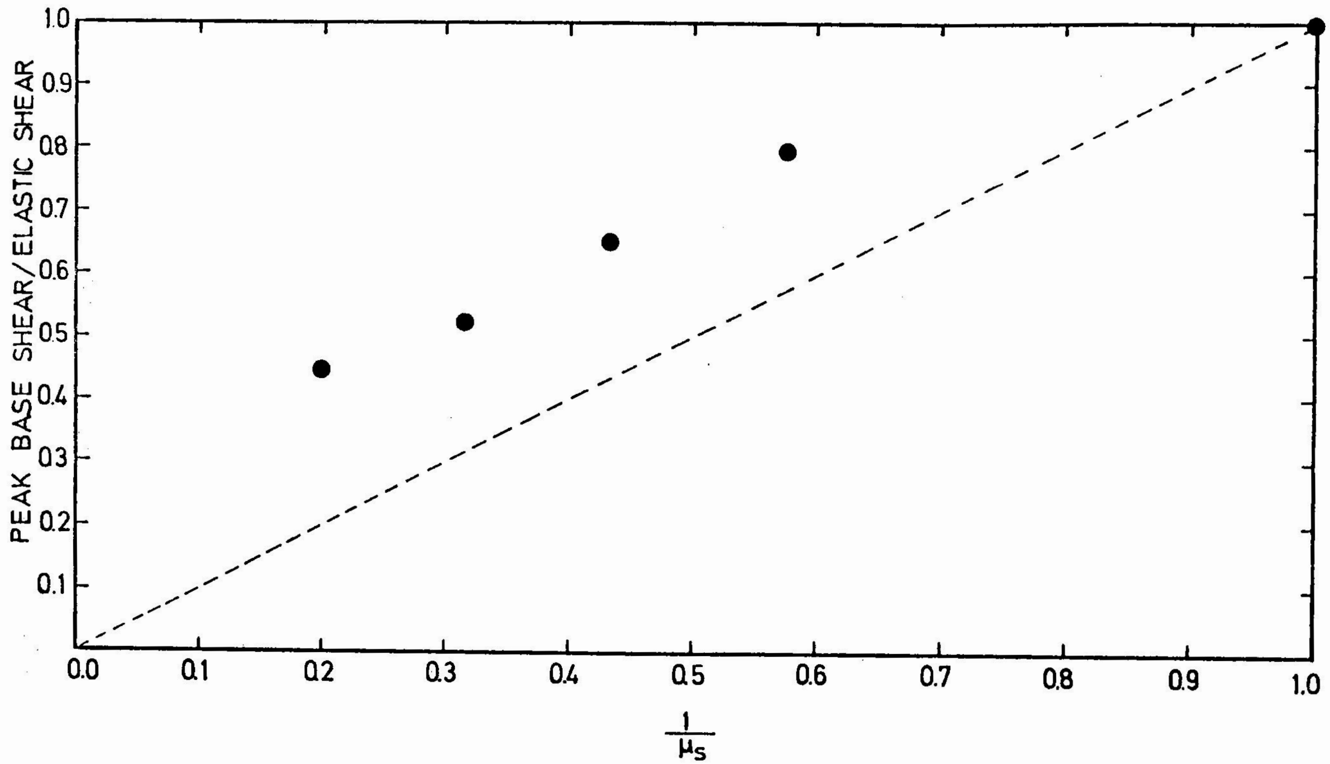

Figure 10 Relationship of Base Shear and System Ductility Ratio