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Displacement Focussed Seismic Design - Presentations

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FORCE BASED SEISMIC DESIGN:

A DISPLACEMENT FOCUSSED APPROACH

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ABSTRACT

The paper explains how the currently used seismic design method, "Force Based Design", can be modified to "focus" on displacements. The proposed method, "Displacement Focused Force based design", and a modified form of "Direct displacement based Design", are both used to design a set of single degree of freedom structures. The maximum displacement of each of these structures was found using non-linear time history analyses for a number of ground motions that had elastic response spectra, which are compatible with the design spectrum. It was found that the two methods of design had comparable accuracy. A major advantage of the proposed method over the "Direct Displacement based Design " approach is that it is very similar to the currently widely practised "Forced Based Design" method.

KEYWORDS

Design, Force Based Design, Displacement Based Design, Performance Based Design

INTRODUCTION

An important aspect of performance based design requires the designer to control the amount of damage that may result from a design level earthquake. It is widely accepted that material strains and inter-storey drift may be used as measures of damage. Current codes attempt to limit damage by limiting inter-storey drift. The force based design (FBD) method is a well established code procedure used to assign the required strength to structural members in seismic design. The method requires the engineer to ensure that the expected structural displacements do not exceed nominal code limits. The method is not focussed on obtaining specific displacement values to ensure the optimal seismic performance of the structure and consequently it is not focussed on a performance based approach. The direct displacement based design (DDBD) method, Priestley [8], which has recently been proposed as an alternative procedure, focuses on the displacement profile of the structure. One of the difficulties of its introduction is that it requires a retraining of current designers. This paper offers an alternative compromise, where a small modification is introduced into the existing well known force based method, to focus the design on to structural displacements.

THE FORCE BASED DESIGN METHOD

Before introducing the proposed modification to the force based design method it is worthwhile to review this approach as it is currently implemented, commenting on some of its weaknesses.

The following are the basic steps in the force based design method.

(i) The material and form of the lateral resisting system of the structure is selected, for example a reinforced concrete shear wall.

(ii) An elastic analytical model of the structure is developed. This is used to obtain its dynamic properties, member actions and the expected maximum deflections for the design level earthquake. The model requires estimates of member stiffness based on forces and displacements sustained by members in the pre-yield condition. For steel structures this is straight forward. However, for reinforced concrete members an effective stiffness is required and codes are used for guidance. This is a topic for debate as different codes provide different advice. For example, the New Zealand Concrete Standard [12] recommends that the flexural stiffness of a rectangular reinforced concrete beam should be taken as 0.4 I_{gross} where the Eurocode [1] suggests that the full section stiffness should be used in the analysis. Recently Priestley [9,10] has called for the results from the latest research to be used, where it is claimed that for concrete members, "stiffness is proportional to strength" and not to gross section properties.

(iii) The dynamic characteristics of the structure are determined. In the simplest form these are represented by the period of the first mode of vibration which is used in an "equivalent static" analysis. For a "modal response spectrum" analysis the mode shapes and corresponding periods of vibration are required.

(iv) Calculate the base shear for the structure. For an "equivalent static" analysis, the base shear is obtained from an equation of the form,

$$V = C\,W_t \tag{1}$$

where W_t is the seismic weight of the structure and C is the lateral force coefficient. This coefficient is essentially a suitable elastic seismic hazard spectrum scaled to allow for the expected nonlinear behaviour of the structure. The scaling differs between codes but is essentially a division of the elastic spectrum by a "force reduction" factor (F_R). For example, the International Building Code [2] and Eurocode [1] divide the spectrum by the factors "R" and "q" respectively. These depend upon the lateral resisting system of the structure. In the New Zealand Loadings Standard [11] the force reduction factor depends on the expected ductility "μ" of the lateral resisting system, a structural performance factor S_p and the period of the first mode of the structure. Thus F_R takes the form

$$\begin{array}{ll} \mu/S_p & \text{for } T > 0.7\text{ s} \\ [(\mu - 1)\,T/0.7 + 1]/S_p & \text{for } 0.45\text{s} < T < 0.7\text{s} \\ [(\mu - 1)\,0.45/0.7 + 1]/S_p & \text{for } T < 0.45\text{s}\,. \end{array} \tag{2}$$

The magnitude of the force reduction factor governs the amount of nonlinear behaviour that is acceptable for the structure. Unfortunately these factors are provided for general structural forms and do not take into account the specific details of the structure. As a consequence they only allow for, at best, only an approximate estimate of the expected material strains and inter-storey deflection that may occur during a design level earthquake.

For response spectrum analyses the approaches are a little more varied, but are consistent in that they use a elastic spectrum, reduced to allow for the anticipated nonlinear response of the structure.

(v) For the equivalent static method, a linear elastic analysis is performed where "equivalent lateral inertia forces" are applied to the structure. The sum of these forces is equal to the base shear. For both the equivalent static and response spectrum methods of analysis, the member actions that result are the required strengths of the critical members.

(vi) The expected deflection profile of the structure for a design level earthquake is obtained by multiplying deflections from these analyses by a "deflection amplification" factor (D_A). The value of D_A varies such that the ratio D_A/F_R ranges between 2/3 and unity. This ratio generally decreases with an increase in ductility level and increases with the more pinched forms of hysteric response [5]. The decrease in the ratio with increase in ductility recognises that for ductile structures the controlling deflection is more likely to be a value reached several times during the ground motion rather than the maximum deflection which is sustained only once. The increase in the ratios for structures, which develop a more pinched hysteric response allows for the increased deflection that these structures tend to sustain compared with those that have fuller hysteric forms. Where necessary, the deflections and structural actions are increased to allow for P-delta effects. Note, the use of displacement amplification factor, D_A, gives rise to the design displacement and not the peak displacement as outlined previously, see also commentary to reference [11].

(vii) The scaled displacements found in (vi) are compared with code allowable drift values. Typically, if the code values are exceeded, members are stiffened and the analysis is repeated. If the displacements are satisfactory the designer proceeds with the detailed design of the members.

Discussion

A number of criticisms of the force based design method have been expressed, Priestley [9], and a new design method, the "Direct Displacement Based Design (DDBD)" method has been proposed in an attempt to resolve them Priestley [8]. A short description of the method, emphasising the key points is given below.

THE DIRECT DISPLACEMENT BASED DESIGN METHOD

To address the required focus on displacements as a design parameter the "direct displacement based design (DDBD)" method devises a new approach whereby the acceptable displacement of an equivalent single degree of freedom (SDF) structure, Δ_d, is selected. The basic steps of this method are outlined below.

(i) and (ii) are the same as for the FBD procedure except that "equivalent" elastic stiffness values are used for members in the analytical model. These equivalent values are assessed from consideration of the forces and displacements in each member when the structure is sustaining its design displacement, Δ_d.

(iii) The magnitude of the design displacement, Δ_d, is determined from the displaced shape found from an equivalent static analysis together with consideration of material strains and the specified limiting inter-storey deflection. In many cases the later will govern the value of Δ_d.

(iv) Calculate the base shear for the structure. As with the force based design procedure the proposal of an equivalent single degree of freedom structure allows the use of design response spectra. In contrast with the FBD method, where the lateral force coefficient (C) of a ductile single degree of freedom system is used to obtain the base shear of the structure, the DDBD method applies the design displacement, Δ_d, to the elastically responding analytical model. The damping of the oscillator is defined as "equivalent" and its value depends upon the estimate of the ductility of the structure and its dominant hysteric form. A typical graph providing this "translation" is shown in Figure 1. With the magnitude of "equivalent" damping estimated, a secant period of the equivalent (SDF) oscillator (T_s) can be found from a "displacement" design spectrum, as shown in Figure 2. The corresponding stiffness, K_s, associated with that period is found from-

$$K_s = W_t/g\ (2\pi / T_s)^2, \tag{3}$$

and the base shear is found by multiplying this stiffness by the design displacement, Δ_d. In most designs, an iterative process is required to ensure that the base shear is consistent with the estimate of the structural ductility.

(v) as for the FBD procedure.

(vi) and (vii). The deflection profile of the structure has already been ascertained in the previous steps.

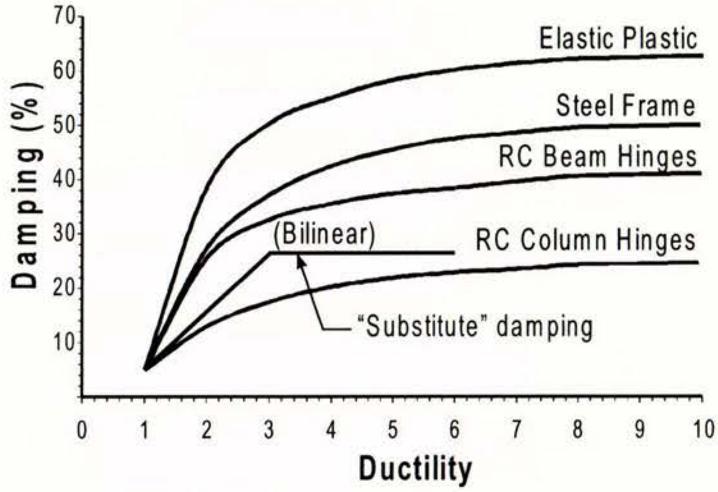


Figure 1: Damping vs ductility (Priestley [8])

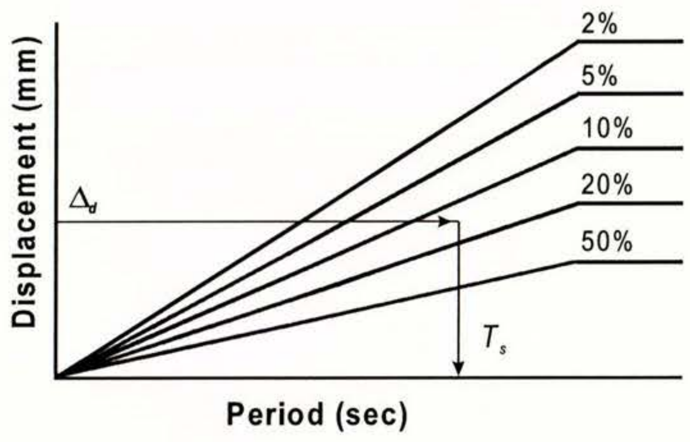


Figure 2: Schematic of Displacement Spectra (Priestley [8])

Discussion

In essence the method starts with a structural displacement and provides the designer with an appropriate base shear and a set of member design strengths. The force based method as currently used, provides the designer with the base shear and member strengths from an estimate, or code advised, force reduction factor. In this method there is no focus on the expected structural displacements and they are typically only checked for code exceedence.

THE DISPLACEMENT FOCUSSED FORCE BASED DESIGN METHOD

The main criticisms of the FBD method are that it fails to focus on displacements, a major measure of damage, and for reinforced concrete structures it over looks the influence of strength in the selection of effective member stiffness. Both these weaknesses are addressed below.

(a) Effective stiffness of reinforced concrete members.
This issue, that is the dependency between stiffness and strength, is not an issue with steel structures. To implement a procedure for reinforced concrete design requires the initial stiffness in step (ii) of the FBD method to be calculated from an estimated steel reinforcement layout. Then the member strengths calculated in step (v) must be compared with the estimated reinforcement. If these are not sufficiently close, the steel content can be modified and the stiffness estimates for step (ii) adjusted, then steps (iii) to (v) repeated.

(b) To focus on displacements, it is proposed that an additional two steps be added to design sequence described in the section on "Force based Design". These, to be inserted after step (iii) are:
(iii a) Determine the magnitude of Δ_d the design displacement of an equivalent single degree of freedom oscillator, from considerations of material strains and permissible inter-storey deflections. This is consistent with the recommendations in the DDBD method.
(iii b) Check whether the stiffness of the structure satisfies the limiting displacement. This is achieved by use of a displacement spectrum consistent with the FBD method. The typical form of these spectra are $C*D_A*g*(T/(2\pi))^2$, where g is the acceleration due to gravity. The displacement spectrum that are consistent with the New Zealand Loadings Standard [11] lateral force coefficient for intermediate soils with an S_p factor, or D_A/F_R, equal to "1", is shown in Figure 3.

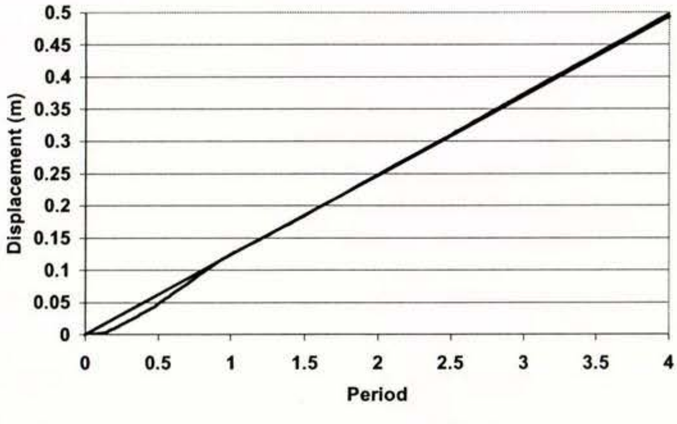


Figure: 3 Displacement Spectrum for NZ Code

To implement this step the period of the structure, T, calculated in step (iii), must be compared with T_e, the period associated with Δ_d. If it is longer then the structure is too flexible and must be stiffened. If the period is shorter then the structure will presumably satisfy minimum drift requirements but the designer may be able reduce the stiffness (and strength) of the members and obtain a more economical solution.

Steps (iv), (v) and (vi) then follow as before. It should be noted that the representative displacement of the structure can be calculated from the scaled displacements obtained in step (v), and it should agree with the choice made in step (iii b).

ESSENTIAL DIFFERENCE BETWEEN THE METHODS

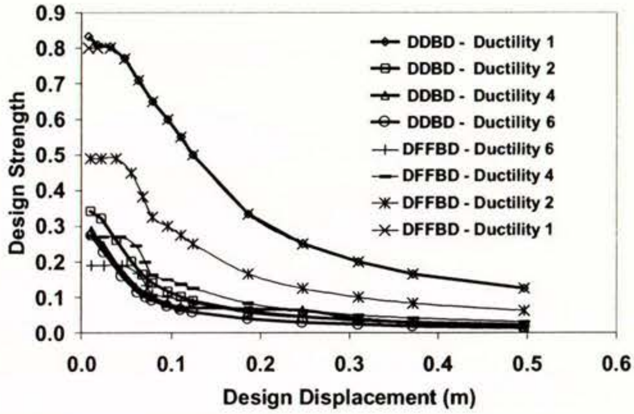
Although from an initial observation there appears to be a number of differences between the proposed "displacement focussed force based" and "direct displacement based" design methods all but one of these are superficial and adjustments to either method could be made to make them the same. The essential difference between the two methods is the "translation" that each uses, which allows a hazard spectrum to be interpreted to provide the expected structural strength (base shear) required to ensure the structure meets the designer's displacement criterion.

The FBD codes achieve this by prescribing strength reduction and displacement amplification factors. For example, the magnitudes of these factors for reinforced concrete shear walls in the IBC are "5½" and "5" (category 1B Table 1617.6), in the New Zealand code, "7.5" and "5", and both factors are "4" in the Eurocode. The magnitude of the force reduction factor governs the amount of non-linearity that is allowed for in the structural form, and is controlled by strain limitations, which depend upon material and detailing standards. The displacement amplification factor converts the limiting "elastic" displacement into the design displacement. As noted previously this is not the maximum deflection (see Forced Based Design method, step (iv)). The lack of consistency between the codes, with respect to the ratio of the force reduction factors to the displacement amplification factors, has been noted by others [6,7].

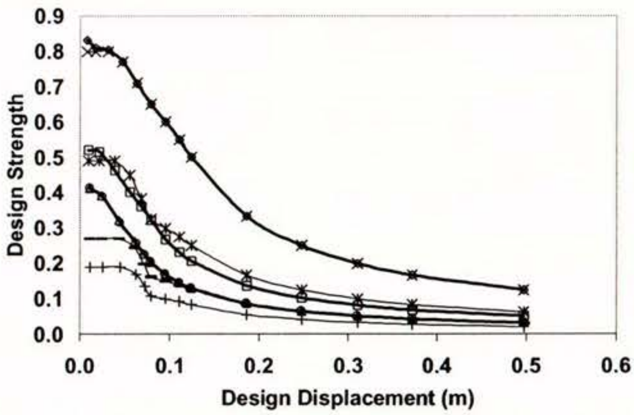
COMPARISON OF DESIGNS

To compare the ability of the two design methods to provide structures whose seismic response best fits the chosen representative displacement, a number of single degree of freedom structures were designed. These were designed to the New Zealand Loadings Standard [11] assuming their non-linear behaviour is bilinear, which is consistent with development of the code lateral strength coefficients, "C". The maximum displacement responses of each structure with code compatible earthquakes were then calculated from a non-linear time history analyses.

Fifty-six structures were designed using the proposed "Displacement Focused Force based design" method and a modified form of the "Direct Displacement based Design" method. The design displacements were chosen so that they matched the spectral displacements for the periods 0.2 to 1.0 sec in steps of 0.1 sec, and 1.5, to 4.0 sec in steps of 0.5, seconds as shown in Figure 3. (To simplify the comparison, the S_p, or D_A/F_R ratio, was taken as unity so that the displacement corresponded to the maximum value and not the design value.) The associated design strengths for the DFFBD method for ductilities 1, 2, 3, and 4 were obtained by converting the displacement spectrum to an elastic acceleration spectrum, (multiplying by $(2\pi/T)^2$) and dividing by the appropriate value of the force reduction factor, as given in Eq (2). Initially two sets of strengths were obtained for the structures designed using the DDBD method. The first set, using the "elastic-plastic" line in Figure 1, thus implementing the "equivalent" damping theory as specified in "Direct Displacement based Design", and the second set from the "(bilinear)" line of the same figure, which is based on the "substitute" damping approach, Judi [3]. The strengths resulting from these designs are plotted in Figure 4.



(a) DDBD "Equivalent" Damping and DFFBD

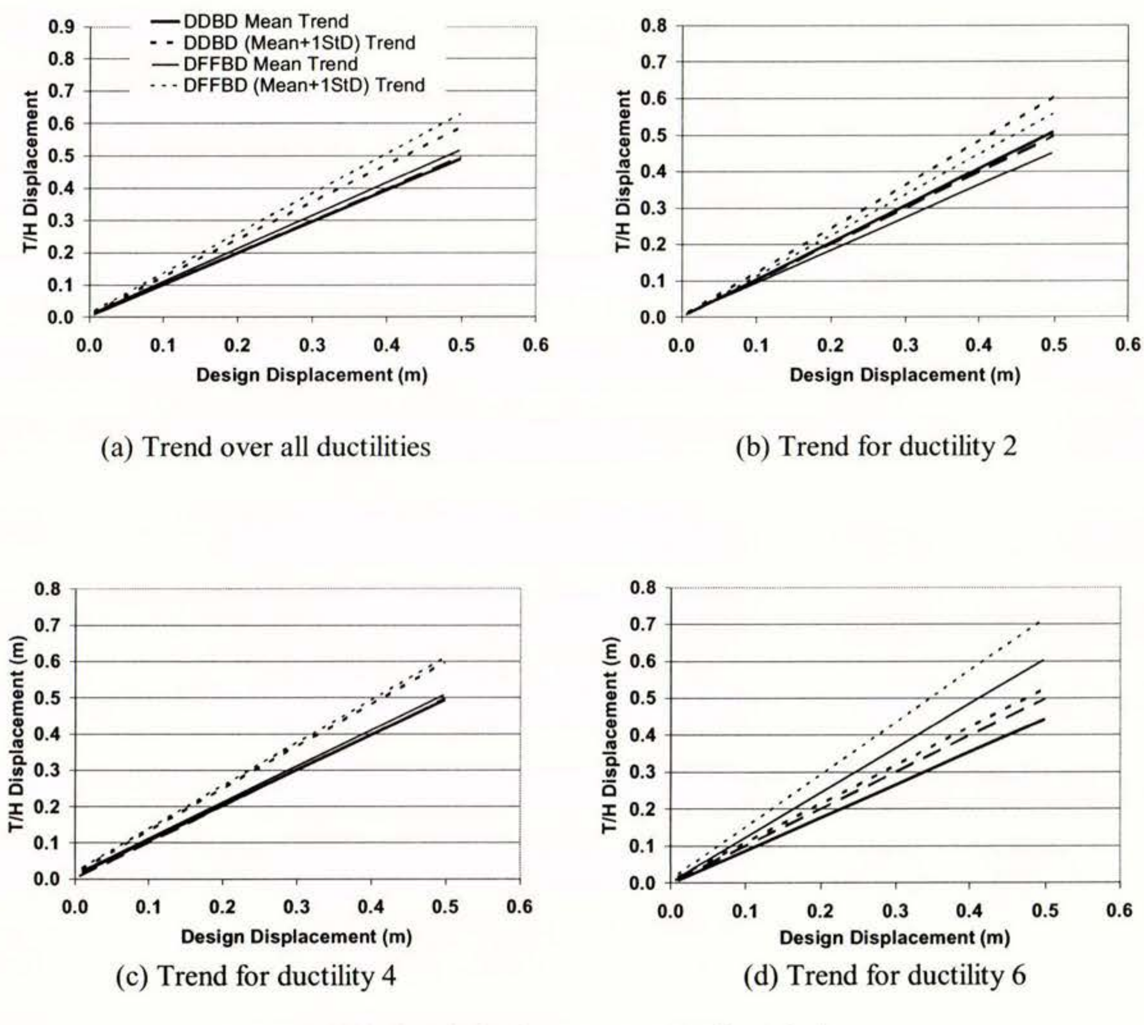


(b) DDBD "Substitute" Damping and DFFBD

Figure: 4 Comparison of Design Strengths

It has been found previously, Judi [3,4] that the "equivalent" damping transformation is inconsistent with the bilinear hysteric response, and it leads to a major underestimate of the required strength. Consequently, it is not surprising that there are large differences in the required strengths calculated for the structures using FBD and the DDBD with this form of damping. These are shown in Figure 4(a). The strength requirements for the structures designed to both the DFFBD and the modified form of the DDBD using the substitute damping method are similar, as shown in Figure 4(b). However, there is a trend showing those designed using the DFFBD method require less strength, particularly at the higher ductility levels.

The response of each of the 112 designs described in Figure 4(b) was calculated for four different ground motions. These earthquakes had 5% damped spectra that closely matched the New Zealand Loadings Standard [11] intermediate soil lateral force spectrum "C" for an elastic response. The results of these analyses are summarised in Figure 5, where the displacements resulting from the non-linear analyses are plotted against the design displacements.



(a) Trend over all ductilities

(b) Trend for ductility 2

(c) Trend for ductility 4

(d) Trend for ductility 6

Figure: 5 Calculated Displacements vs Design Displacements

Figure 5 (a) presents mean and mean plus one standard deviation plots for all displacements resulting from both design methods. This plot shows little difference between the two approaches. It should be noted that the magnitude of the variance in the results for either of the methods is largely a function of the variability between the characteristics of the chosen earthquakes. Figures 5 (b) – (d) illustrate the accuracy of the methods to control displacements at different ductility levels. The proposed DFFBD approach appears to be a little conservative for structures designed for smaller ductilities and possibly a little non-conservative for structures designed for larger ductilities. The displacement trends of the structures designed with DDBD procedure appear to be the reverse.

DISCUSSION AND CONCLUSION

It is contended in this paper that the essential difference between the force based design procedure and the direct displacement based design procedure is the "translation" that each uses to enable response spectra to be used to provide the basic required design strength of a structure. It is accepted that a focus on displacements in seismic design is desirable and it is proposed that additional steps be introduced into the current force based design method is a practical way to improve this well established approach. Fifty six simple single degree of freedom structures were designed to the spectrum of the New Zealand Loadings Standard [11], for a intermediate soil site using both design methods. Nonlinear time history analyses of these structures to four normalized earthquakes allowed a comparison of calculated displacements and expected design displacements. The mean of all of the results showed little difference between the methods. A breakdown of the results by ductility level showed that the FBD method was a little conservative at low ductility and possibly non conservative for high ductility structures. The trends for the DDBD were the reverse.

ACKNOWLEDGMENTS

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Influence of Hysteretic Form on the Basic Seismic Hazard Coefficients

by

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ABSTRACT

The hysteretic behaviour of different structural forms and materials varies widely. However, codes of practice generally only give one set of basic seismic hazard coefficients (response spectra) to cover all structural types. To assess the influence of different hysteretic forms on seismic response a large number of time history analyses are made using a number of different earthquake records and hysteretic models. In addition the influence of changing both the level of damping and the rate of strain hardening are examined.

The analyses indicate that the form of hysteretic response has only a relatively minor influence on the maximum displacement that is sustained. Varying the viscous damping level was found to make a significant difference to elastically responding structures but it had less effect on ductile structures. Changing the strain-hardening ratio was found to have only a small influence on behaviour.

1.0 INTRODUCTION

The form of hysteretic response of structures varies very significantly with the structural form, materials, detailing and characteristics of the foundation. However, in the Loadings Standard [1], and many other seismic codes of practice, only one set of response spectra are given to cover all hysteretic forms. In this paper the influence of different hysteretic models on seismic behaviour is described. Additional details on this work are described in reference 2.

Fig. 1 shows the hysteretic response obtained from a number of tests of different structural elements. It can be seen that these vary with both the materials and structural form. Fig. 1(a) shows a near bilinear response obtained from a test of a shear-yielding element for an eccentrically braced frame [3]. There was very little degradation on repeated loading cycles and with the application of the larger displacements cycles strain hardening increased the maximum force that could be resisted. A bilinear model provides a conservative representation of this behaviour. A similar hysteretic shape is obtained from reinforced concrete frames, which develop unidirectional plastic hinges [4]. The lateral force versus displacement for such a case is shown in Fig. 1(b). As the displacements are increased there is a limited amount of stiffness degradation. However, until the reinforcement buckles, the force versus displacement response is close to bi-linear.

Fig. 1(c) shows the force versus displacement response of a reinforced concrete beam subjected to cyclic inelastic loading [5]. In this case in-elastic deformation together with shear reversal results in shear deformation. This leads to a pinched response, that is the stiffness at low load levels is low, due to the opening and closing of diagonal cracks in the plastic hinge zone associated with the yielding of the stirrups. Under repeated loading to the same displacement some stiffness degradation occurs as additional yielding of the stirrups takes place. It should be noted that the unloading curves remain relatively steep, with little recovery in deflection when the load is removed.

Fig. 1(d) illustrates the lateral force versus displacement for a reinforced concrete column subjected to cyclic loading [6]. In this case the column sustained an axial load of "$0.21A_g f'_c$". There are two marked differences in the behaviour of columns when compared to beams. The first is that the axial load reduces the magnitude of the shear resisted by the stirrups. This limits the yielding of the stirrups and the width of the diagonal cracks. Consequently shear pinching in the force versus displacement relationship is reduced. The second difference is in the stiffness of the unloading curves. With beams this stiffness is high, as the cracks that form due to tension yield of the reinforcement remain open when the load is removed. With the column the axial load closes these cracks when the lateral force is removed and as a result there is greater recovery in displacement on un-loading.

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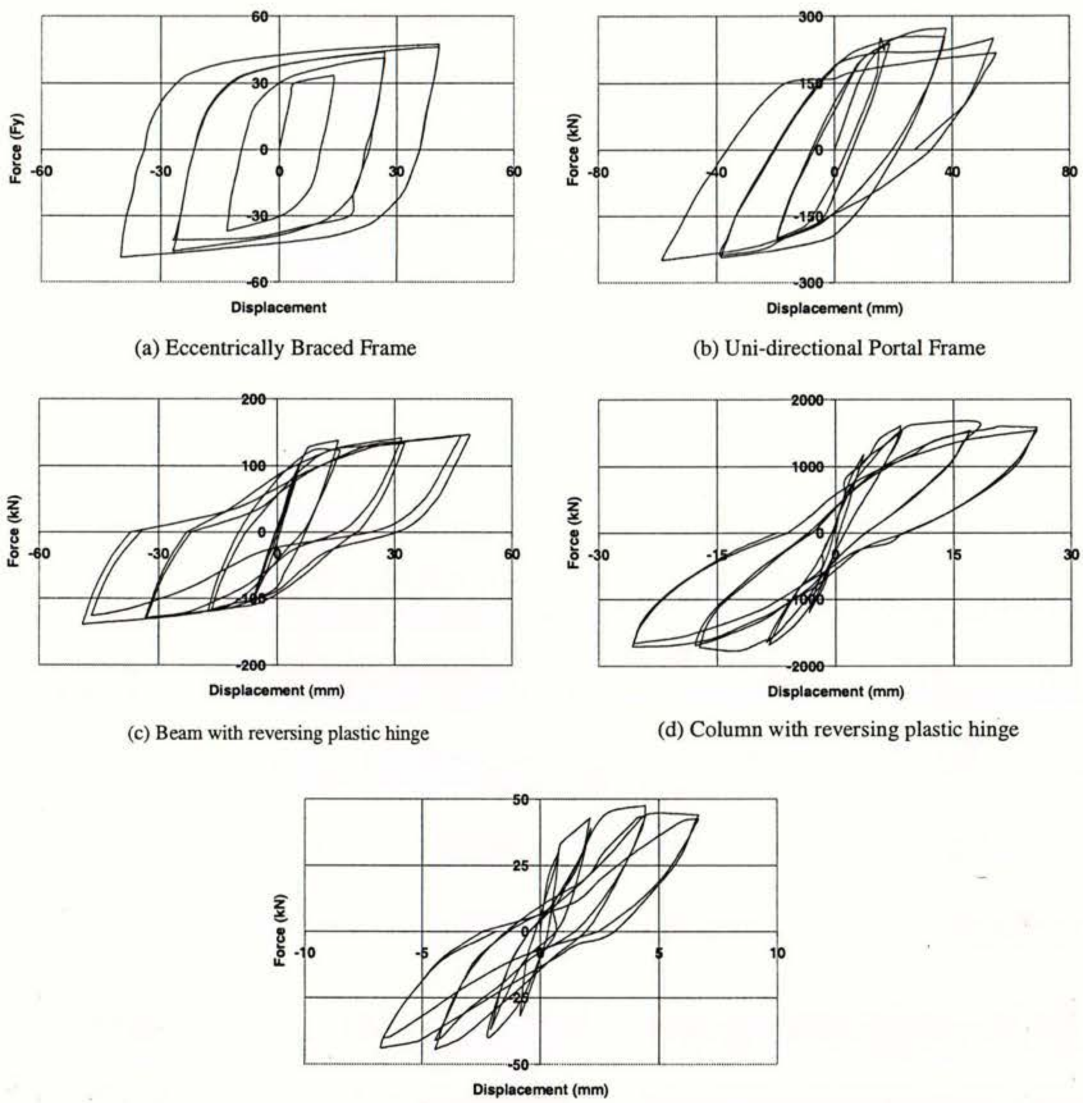


(a) Eccentrically Braced Frame

(b) Uni-directional Portal Frame

(c) Beam with reversing plastic hinge

(d) Column with reversing plastic hinge

(e) Masonry wall with reversing plastic hinge

Fig. 1 Force displacement relationships for different structural elements

Fig.1 (e) shows the lateral force displacement response for a reinforced masonry wall [7]. In this case appreciable stiffness degradation occurred in the loading curves as a result of high shear deformation in the plastic hinge zone.

All the load deflection relationships shown in Fig. 1 dissipate appreciable energy by hysteretic behaviour of the materials. In this respect they are different from two further extreme cases. The first of these is for prestressed concrete members with un-bonded cables. The response in this case is typically S shaped, with the loading and unloading curves lying very close to each other. Analyses using analytical models representing this behaviour indicate that the lack of energy dissipation increases the ductility demand by about 50 percent when compared to a bilinear response [8, 9]. The other extreme is for a tension braced pin jointed frame. A limited amount of energy may be dissipated by these structures with the diagonal bars yielding in tension. However, when load reversals occur the diagonal bars buckle and the stiffness drops to close to zero.

2.0 HYSTERETIC MODELS

A basic hysteretic model has been developed for a single degree of freedom analysis program [10]. The loading relationship is represented by three straight lines, of which the third line represents strain hardening relationship. A further two lines represent the unloading relationship, so that a half cycle of loading involving inelastic deformation is represented by five changes of stiffness. The gradient of each line is controlled by coefficients, which vary with both the maximum displacement that is reached and the sum of the inelastic displacements sustained along the strain hardening lines. This basic model allows a wide range of hysteretic relationships to be represented. In particular it allows stiffness degradation to be modelled under cyclic loading conditions where the previous maximum displacements sustained in each direction are not exceeded.

The coefficients defining the gradients of the five lines have been chosen to give four different models as described below.

1. The first of these is the bi-linear model, which provides a reasonable representation of the force displacement response of shear yielding steel elements and reinforced concrete frames, which form unidirectional plastic hinges, see Figs. 1(a) and (b).
2. The second model represents the behaviour of a reinforced concrete beam, which sustains a reversing plastic hinge, as illustrated in Fig. 1(c).
3. The third model represents the behaviour of a reinforced concrete column, which is subjected to a moderate axial load, as shown in Fig. 1(d).
4. The final model represents the behaviour of a masonry wall, as illustrated in Fig. 1(e).

These four models cover a wide range of hysteretic shapes met in practice. However, the extremes of prestressed concrete members with un-grouted cables and pin jointed tension braced frames have been excluded.

3.0 ANALYSES

The analyses were made using sixteen earthquake ground motions. Twelve of these were Californian earthquake ground motions, which are identified in ATC 40 [11] as suitable candidates for time history analyse. All these records were recorded at sites with stiff to medium ground conditions located at least 10km from fault rupture. The magnitude of each event was not less than 6.5 and the peak ground acceleration was at least 0.2g. Four of the records were from the Loma Prieta earthquake of Oct. 17th, 1989, four more from the Landers Earthquake, June 28th. 1992, and the final set of four were from the Northridge Earthquake of 17th. Jan. 1994. The remaining earthquake records were the El Centro 90° 1940, Edgecombe Matahina Dam 90°1987, Hachinohe Tokachi Oki 90° 1968 and Kern County Taft Lincoln Tunnel N21°E 1952.

In each series of analyses single degree of freedom structures were analysed for the 16 earthquake ground motions for 50 different structures, which had initial elastic periods of 0.1 to 5 seconds with 0.1 second steps. The analyses were made to determine the required strength for a nominated ductility level. In different sets of analyses the effect of changing the hysteretic model, the level of viscous damping and the strain hardening values were examined. Details are given in the next section.

4.0 RESULTS OF ANALYSES

4.1 Influence of hysteretic form on response

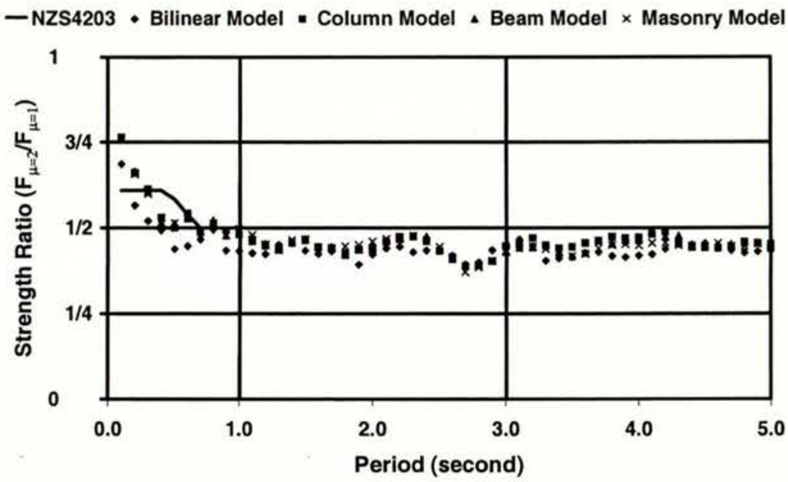
In this set of analyses the strength required for ductility levels of 2, 4 and 6 were determined for all the hysteretic models. All the structures were given a viscous damping level, ξ, of 5% and a strain hardening ratio, α, of 2.5%. The required strength for each individual period, ductility level and earthquake ground motion was divided by the corresponding strength required for elastic response to give a strength ratio. The average strength ratio obtained from the 16 ground motions for each period and specified hysteretic form are shown graphically for ductility levels of 2, 4 and 6 in Fig. 2.

From Fig. 2 it can be seen that the strength ratios for structures with a period greater than 1 second are close to the inverse of the ductility ratio, for all the hysteretic models. This corresponds to the equal displacement concept. For periods below the 1 second value the required strength is greater than that that implied by the equal displacement concept. As indicated in Fig. 2, the current New Zealand Loadings Standard [1] recognises this and the specified strengths for such structures are increased above the level implied by the equal displacement concept.

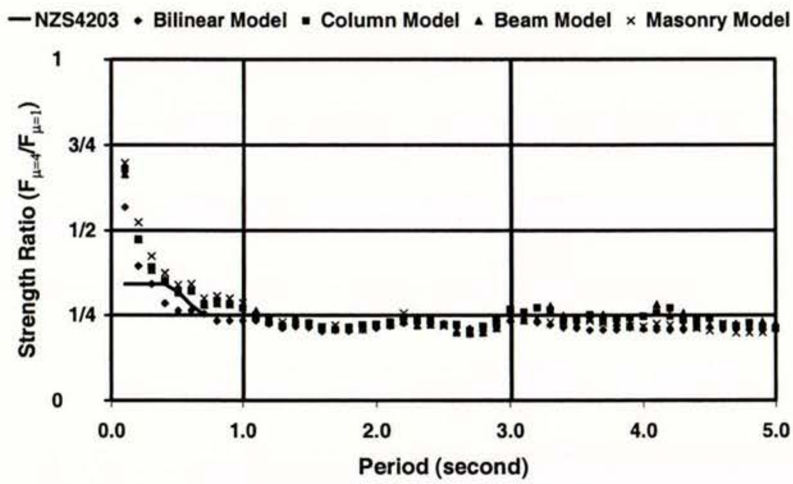
The analyses show that in general the hysteretic form has little influence on the maximum required strength for a given ductility level. It follows that the maximum displacement is also relatively insensitive to hysteretic form. This finding is consistent with conclusions found from a number of less extensive investigations [12, 13, 14, 15 & 16]. From a close examination of the results it can be seen that in the short period range, that is where the periods are less than 1 second, there is a small but significant difference in strength ratios, between the bilinear model and the other hysteretic forms. This difference increases as the ductility level increases. For a ductility of 2 the average strength required for the bilinear model is 89% of that required for the average of the other models. The corresponding values for the ductilities 4 and 6 are 82% and 79% respectively.

4.2 Influence of damping ratio on strength

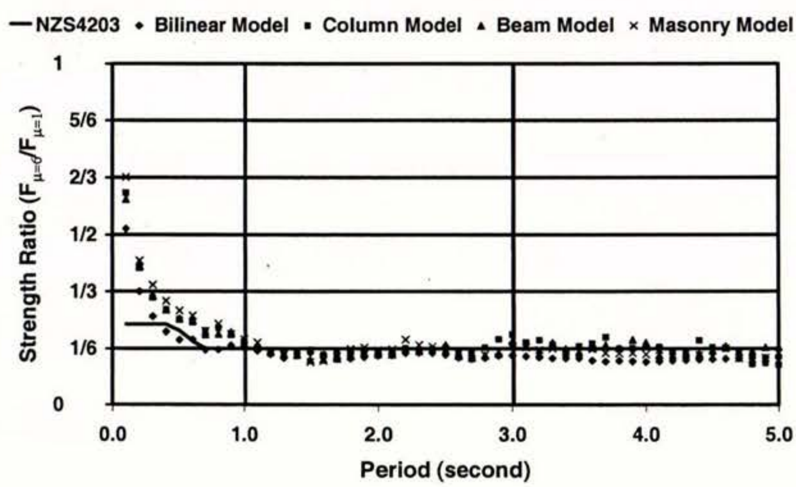
The analyses described in the first set were repeated but the viscous damping level, ξ, was set to 0.5% and then to 2%. In all these analyses the strain-hardening gradient was held constant at 2.5 percent. The required strength



(a) Strength ratio for ductility 2



(b) Strength ratio for ductility 4



(b) Strength ratio for ductility 6

Fig. 2: Required for ductility levels of 2, 4 and 6 as a proportion of that for a ductility of 1.0

found in each analysis was divided by the corresponding strength for the same earthquake record and ductility level obtained with 5% viscous damping to give the strength ratio. The averaged values found from the analyses are summarised in Table 1. As the period of a structure influences the result, the values have been listed for three period ranges, which are under the symbol "R" in the Table 1. Ranges 1, 2 and 3 are for the periods between of 0.1 to 1.0 seconds, 1.1 to 3 seconds and 3.1 to 5 seconds respectively.

The analyses show that changing the damping ratio had a very similar effect on all the models. Consequently only the average value for all 4 models is shown. From Table 1 it can be seen that decreasing the damping below 5% increases the strength required to maintain the specified level of ductility. For the elastically responding structures the required strength increase is appreciable, with the proportional increase in strength being greater for the shorter period ranges. The effect is considerably smaller for structures that have some ductility. Where the ductility is 2 or more, reducing the damping from 5% to 2% increases the required strength by an average of close to 12 percent. The corresponding value if the damping is reduced to 0.5% is an average increase in strength of 23 percent.

Table 1: Influence of viscous damping on strength ratio.

ξ %	μ	R	Av. All*	ξ %	μ	R	Av. All
0.5	1	1	1.72	2	1	1	1.31
		2	1.60			2	1.28
		3	1.46			3	1.21
0.5	2	1	1.27	2	2	1	1.16
		2	1.26			2	1.14
		3	1.22			3	1.11
0.5	4	1	1.20	2	4	1	1.13
		2	1.20			2	1.11
		3	1.20			3	1.08
0.5	6	1	1.20	2	6	1	1.13
		2	1.25			2	1.11
		3	1.26			3	1.12

* Average for all hysteretic models

5.3 Influence of strain-hardening ratio

To assess the influence of strain-hardening ratio on the required strength a further set of analyses were made. In this case the viscous damping level was maintained at 5% while the stain-hardening ratio, α, was first decreased from 2.5% to 0% and then increased to 5%. The individual strengths with each earthquake record and ductility level were divided by the corresponding value found for the structure with 2.5% strain hardening to give the strength ratio.

It was found that the change in strain hardening ratio had little effect on the beam, column and masonry models, with average strength ratios for each model in the different period ranges not changing by more than ±2%. The maximum change was 5% for the column model, and this occurred in the period range of 3.1 to 5 seconds with a ductility of 6. However, there was an appreciable influence on the bi-linear model. In this case the effect increased with the ductility level. Decreasing the strain-hardening ratio from 5% to 0% resulted in an increase in strength. For a ductility of 2 it was 3 percent, while for ductilities 4 and 6 it was 22 and 17 percent respectively. Increasing the strain-hardening ratio to 5% resulted in the corresponding required strength ratio decreasing by 2, 6 and 6 percent for the ductility levels of 2, 4 and 6 respectively.

6.0 CONCLUSIONS

1 Approximately 64 000 single degree of freedom analyses have been made using a range of earthquake ground motions to examine the influence of hysteretic form, viscous damping level and strain hardening characteristics, on the strength required for elastic and ductile structures. Structures that have load displacement responses, which can dissipate very little energy hysterically (ie prestressed concrete with un-bonded tendons), were excluded from the investigation.

2 It is shown that the hysteretic form has only a minor influence on the strength required for a given level of ductility. This finding agrees with previous studies [12, 13, 14, 15 & 16].

3 Reducing the viscous damping level below 5 percent is shown to have a significant influence on elastic response but a smaller effect on the strength required for structures with a ductility of 2 or more.

4 Changing the strain-hardening ratio from 2.5% to 0%, or 5 percent, is shown to have only a minor influence on the required strength with the beam, column and masonry models. However, with the bilinear model the significance of these changes increases with ductility. For a ductility of 6 reducing the strain-hardening percentage to zero increases the required strength by an average of 15 percent. The corresponding decrease in required strength when the strain-hardening ratio is increased to 5% is 6 percent.

ACKNOWLEDGEMENTS

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Direct displacement based design – a definition of damping

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ABSTRACT.
The response of a range of structures with different hysteretic rules was investigated for a variety of strong earthquake ground motions. It is shown that the use of substitute viscous damping, instead of equivalent viscous damping, with the direct displacement based design approach, improves the accuracy and generality of the method, and gives predictions which are marginally better than those obtained with force based design. Analyses of a wide variety of earthquake records show that the values of substitute viscous damping are relatively insensitive to the type of earthquake but they vary with the structural period, ductility level and hysteretic form.

1 INTRODUCTION

It is widely accepted that the maximum deformation imposed on a structure is an important indication of the level of damage that is sustained. As deformation is closely related to displacement, design methods seek to limit the maximum lateral displacement resulting from design level earthquakes.

The way in which the design displacement limit, generally expressed as a maximum inter-storey deflection, is considered in design is radically different with force based and displacement based design methods. With force based design (FBD) a ductility level is selected, design strengths are determined and the corresponding lateral deflections are found. With displacement based design the designer starts with a limiting displacement and determines the design forces and ductility level corresponding to that value. In both cases a trial and error approach is required. There are several methods of displacement based design that have been proposed in the literature. In this paper the "Direct Displacement Based Design" (DDBD) method as proposed by Kowalsky et al. (1994, 1995) is the main focus of the discussion.

2 BASIC APPROACH FOR FORCE BASED AND DISPLACEMENT BASED DESIGN

The fundamental generic steps involved in force-based and displacement-based design are briefly outlined below. More comprehensive descriptions of the different approaches may be found in Judi et al. (1998, 2000), Kowalsky et al. (1994, 1995) and Priestley & Kowalsky (2000). It should be noted that in practice some of the steps are simplified in codes of practice. In particular the variation of stiffness of members with reinforcement content is generally ignored in practice.

2.1 *Force Based Design*

A ductility level, which is appropriate to the form of structure, is selected together with a limiting lateral displacement. Trial member sizes of structural steel members, or for reinforced concrete section sizes and reinforcement contents, allow the initial stiffness values of members prior to yielding to be assessed. Using these an elastic analysis is made to find the design forces. In this analysis a level of damping appropriate to the elastically responding structure (typically 5%) is used, together with a design acceleration response spectrum for the chosen ductility level and hysteretic form. Generally codes of practice neglect the influence of hysteretic form on the design spectrum and only one, which is usually based on bilinear response, is given. A logical development of the approach would give response spectra for different hysteretic forms. In some codes the elastic response spectrum values are reduced to allow for ductility while in others the elastic spectrum is used. Where the second option is followed the design actions are found by dividing the analysis values to allow for the design ductility level. The assumed stiffness values are checked against member sizes required to sustain the actions found in the analysis and the process is repeated with revised stiffness values if necessary. The ultimate lateral deflections are assessed using a multiplier to increase the values found in the elastic analysis to allow for the expected inelastic deformation, which relates the elastic analysis deflections to the ultimate values. This multiplier, which is generally embedded in a table in the design code, depends on the initial elastic period of the structure and the design ductility level. Its value is derived using the "equal energy" and "equal displacement" concepts. The process is repeated until acceptable convergence is obtained between the resultant ultimate displacements and the predicted values.

2.2 *Displacement Based Design*

A limiting design displacement is selected. On the basis of assumed structural steel member size, or for reinforced concrete the section dimensions and reinforcement contents, the deflection that can be sustained at the effective elastic limit is assessed. This is the ductility 1 displacement. Dividing the design displacement limit by the ductility 1 displacement gives an estimate of the ductility, which can now be used to find an equivalent viscous damping for an associated elastically responding model. This model has its stiffness value based on the secant stiffness between zero and maximum displacement as illustrated in Figure 1. From a set of design displacement response spectra, constructed for different viscous damping values, the fundamental period of the model is found. This value leads to the model stiffness and hence to the design strength, as illustrated in Figure 1. The design strength is now checked against the initial assumed sizes and ductility 1 displacement. The process is repeated until satisfactory convergence is achieved. For multi-degree of freedom structures some assumption has to be made to distribute the strength into the structure.

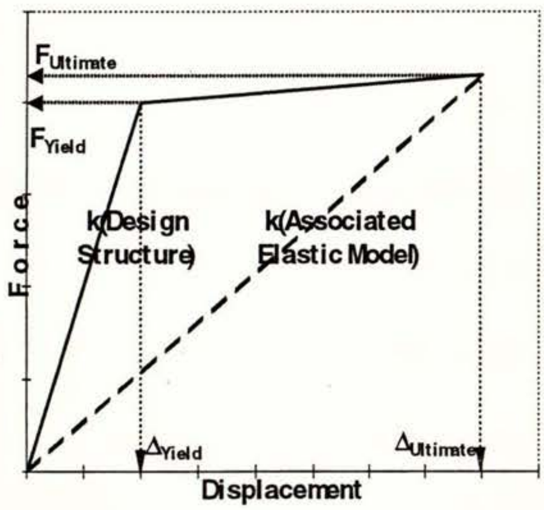

Figure 1 – The Design Structure & Associated Elastic Model in DDBD

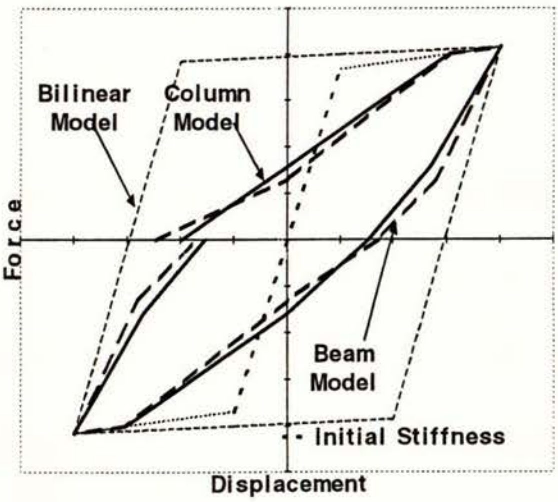

Figure 2 – The Hysteretic Models used in the Study

The force based design method is based on the equal displacement and equal energy concepts. With this approach the influence of different hysteretic models on response can be recognised by the use of different ductility acceleration response spectra for the different hysteretic forms, even though this is not done at present. Displacement based approaches are established on the assumption that a hysteretically responding design structure sustains the same displacement as an associated elastically responding model with an appropriate level of viscous damping. As noted above the associated elastic model has its stiffness value based on secant stiffness for the position of maximum displacement. A basic key to the approach is to find the appropriate damping value for this method. If this can be found differences in hysteretic form can be incorporated in the analysis by changing the level of viscous damping.

3 BACKGROUND TO DAMPING FOR ASSOCIATED ELASTIC MODEL IN DISPLACEMENT BASED DESIGN

Two different concepts have been proposed for finding the required level of viscous damping for the associated elastic model, namely "equivalent viscous damping" and "substitute viscous damping". Equivalent viscous damping value is derived by equating the energy dissipated in one cycle by the oscillator, which is displaced under steady state conditions between + and – its maximum displacement, to the viscous energy dissipated by the associated elastic model undergoing the same displacement. Substitute viscous damping is derived by equating the total energy dissipated by the hysteretic oscillator, when it is subject to a strong earthquake motion, to the corresponding energy dissipated due to viscous damping in the associated elastic model.

Equivalent viscous damping is based on a concept first proposed by Jacobsen (1930). He proposed that the maximum displacement of an oscillator with complex damping mechanisms, when subjected to steady state vibratory motion, could be found from an analysis of a viscously damped associated elastic model. The complex damping mechanism in the oscillator could include hysteretic behaviour due to yielding. He proposed that the appropriate level of viscous damping for the associated elastic model could be found by equating the energy dissipated by the oscillator to that dissipated by the associated elastic model.

Jacobsen found that his theory, when implemented for mechanical systems under forced steady state vibration, was in close agreement with exact solutions for systems with relatively low levels of non-linearity. For the non-regular vibratory motion Jacobsen suggested that a time average damping would be more representative than the equivalent viscous damping, but that the equivalent viscous damping is more convenient to use

Gulkan & Sozen (1974) carried out dynamic tests on a series of reinforced concrete frames. They found that the substitute viscous damping value found in their tests, could be assessed with sufficient accuracy for seismic design purposes, by the equivalent viscous damping concept presented by Jacobsen. However, this deduction is only valid for structures that behave in a similar manner to their test specimens. These formed reversing plastic hinge zones, and as such they exhibited stiffness degrading hysteretic behaviour typical of reversing reinforced concrete plastic hinges.

Priestley & Kowalsky (2000) show that the equivalent viscous damping level varies with the hysteretic model. This is a convenient feature as it enables the DDBD method to accommodate a wide range of hysteretic behaviour modes. Moreover, Kowalsky et al. (1995) indicated that the equivalent viscous damping concept, by virtue of its geometric background, is a function of the ductility level anticipated for the design structure and it is independent of the structural period. Hence equivalent viscous damping was adopted into the DDBD method.

In some papers upper empirical limits were proposed for the equivalent viscous damping with these values depending on the hysteretic form. In particular Loeding, Kowalsky & Priestley (1998) recommended that empirical maximum values, which are significantly lower than the maximum analytical equivalent viscous damping values, be used. However, in other papers these upper limits do not appear to be used (Priestley & Kowalsky 2000).

Judi et al. (1998) found that the equivalent viscous damping approach resulted in non-conservative designs for the case of bilinear behaviour. The large area enclosed in the bilinear hysteretic loop to the maximum inelastic excursion gives a high level of equivalent viscous damping (~60% at a ductility of 6). With this value the direct displacement based design gives much lower strength levels than time history analyses indicate are required.

Judi et al. (2000) investigated the background of the substitute viscous damping, which Jacobsen referred to as 'time average damping'. They proposed that though it is more convenient to calculate the equivalent viscous damping, the use of substitute viscous damping is more logical. By carrying out a number of designs for single degree of freedom structures with different hysteretic relationships and evaluating these with time history analyses, they noted that this approach appeared to give more consistent results than designs based on the equivalent viscous damping concept. Based on a study of eight earthquake records they found that this concept is convenient for inclusion in DDBD design method. The justification for this is twofold. Firstly, it was found that the substitute viscous damping relationships are relatively insensitive to the earthquake record. Secondly, the substitute viscous damping concept presents a reasonable analytical representation of the design structure for the different hysteretic models, including the bilinear, and there is no need for artificial manipulations of the relationships as appears to be required with equivalent viscous damping. The substitute viscous damping was found to be dependent on the ductility level and to a lesser extent on the elastic period of the design structure. In the study reported in this paper use of equivalent and substitute viscous damping is examined in greater detail for a wider range of ground motions and hysteretic models than was the case in the previous paper.

4 DETERMINATION OF SUBSTITUTE VISCOUS DAMPING VALUES

In this investigation three different hysteretic models were used, namely a bi-linear model, a column model and a beam model. They are illustrated in Figure 2. Structural steel eccentrically braced frames and reinforced concrete frames that develop unidirectional plastic hinges have load deflection characteristics that approach those of the bilinear model. The column model is based on the response of a reinforced concrete column, which develops a reversing plastic hinge under cyclic loading. In this case both the loading and unloading stiffness values degrade, which gives it a greater tendency to self-centre that the bilinear model. The beam model is based on the response of a reversing plastic hinge in a reinforced concrete beam. It is similar to the column model except that, firstly the hysteretic loop becomes pinched in shape under repeated inelastic cyclic loading due to shear deformation in the plastic hinge zone, and secondly the unloading stiffness does not degrade to the same extent as occurs in the column model. The rules for the column and beam hysteretic models were developed from experimental results (Fenwick & Davidson 1994). It should be noted that both loading and unloading stiffness values degrade under steady state cyclic loading with these models in a similar way to that observed in many structural tests.

The substitute viscous damping values were assessed for 21 ground motions. The first group of four ground motions were the original records of Imperial Valley, El Centro NS Record 1940, Edgecombe Earthquake, Matahina Dam Base NS Record 1987, Hachinohe Earthquake, Tokachi Oki NS Record 1968, and Taft Earthquake, Kern County Record NE 1952 for soil sites. The second group of seismic events were artificial earthquakes that were produced by normalising the first family so that their elastic acceleration response spectra at 5% damping matched that of the Loadings Code NZS4203: 1992 for intermediate soils. The third group is that of ground motions that are characterised by long duration (>40sec) for soil sites. They are Hachinohe Earthquake, Tokachi Oki EW Record 1968, Olympia Earthquake, Seattle Army Base NW Record 1949, Michoacan Earthquake, Tacy EW Record 1985 and Chile Earthquake, Vina del Mar SW Record 1985. The last two groups of earthquakes investigated were those for near fault events both for soil and rock sites. The soil records are the Imperial Valley Earthquake, Meloland NS Record 1979, two records of Northridge Earthquake 1994, Sylmar NS and Rinaldi SW Records, Cape Mendocino Earthquake, Petrolia NS 1992 and Tabas Earthquake, Tabas NW Record 1978. The rock site events are Cape Mendocino Earthquake, Cape Mendocino NS Record 1992, Landers Earthquake, Lucerne EW Record 1992, Loma Prieta Earthquake, Los Gatos Presentation Centre NS Record 1989 and Kobe Earthquake, JMA EW Record 1995.

Figure 3 shows how the trend lines for substitute viscous damping values found from the first three groups of earthquakes change with hysteretic model, period and ductility level. The corresponding near fault values, for the last two groups, are shown in Figure 4. The reason for this separation is that due to the strong pulses in these records, design results with either FBD or DDBD with either damping approach, do not produce satisfactory convergence with time history analytical values. However, it can be seen that the substitute viscous damping values are similar to those of the other earthquake records.

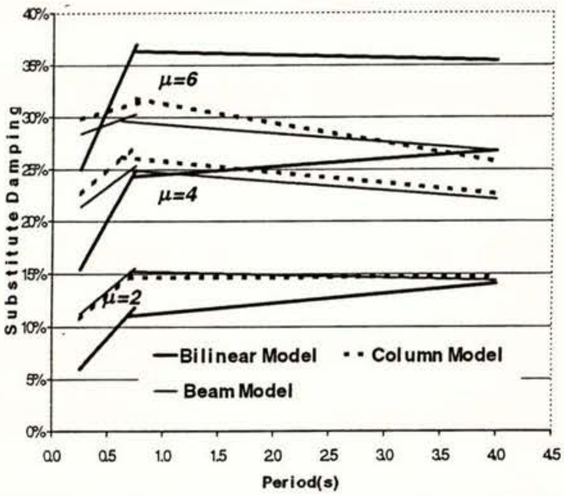

Figure 3 – Substitute Viscous Damping Variation (Excluding Near Fault)

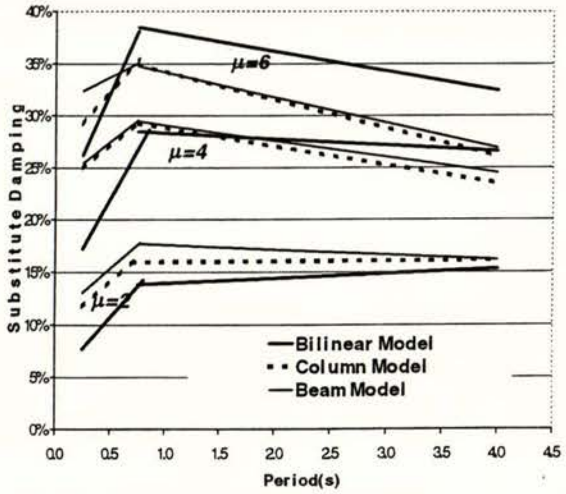

Figure 4 – Substitute Viscous Damping Variation (Near Fault)

From Figure 3 it can be seen that there is little difference in the values of substitute viscous damping for the column and beam models. For low values of ductility ($\mu \leq 2$) the substitute viscous damping obtained with the bilinear model is less than the corresponding values for the column and beam models, while the reverse is true for higher ductility levels (μ = 6). Another relevant observation that could be made from Figures 3 and 4 is that the substitute viscous damping values increase sharply with period in the period range of 0 to 0.7 seconds, while above this level the influence of period is relatively small. Table 1 contains a representative sample of the mean values and coefficients of variation for the substitute viscous damping values found from the analyses for the earthquake records in the first three groups. These values indicate the variation that was obtained with both the bilinear and column models.

Table 1: Substitute viscous damping values for bilinear and column models

Period (s)	$\mu = 2$				$\mu = 4$				$\mu = 6$			
	Bilinear		Column		Bilinear		Column		Bilinear		Column	
	Avg	CoV	Avg	CoV	Avg	CoV	Avg	CoV	Avg	CoV	Avg	CoV
0.25	5.7%	0.30	10.47%	0.15	14.4%	0.28	21.82%	0.25	23.3%	0.22	29.63%	0.16
0.50	9.5%	0.23	13.70%	0.16	22.2%	0.29	26.69%	0.08	34.3%	0.21	31.02%	0.08
0.75	11.5%	0.19	14.87%	0.14	23.9%	0.17	26.44%	0.10	35.3%	0.15	31.16%	0.11
1.00	11.7%	0.20	15.07%	0.11	24.7%	0.19	24.40%	0.12	36.3%	0.17	30.58%	0.14
1.00–4.00	12.7%	0.07	14.72%	0.06	25.5%	0.05	24.12%	0.05	35.9%	0.04	28.52%	0.09

Avg = Average, CoV = Coefficient of Variation

5 ANALYSIS AND DISCUSSION

Using the bilinear, elastic-perfectly-plastic model, a set of 14 structures was designed by FBD to the NZ Standard spectrum for intermediate soils for μ of 1, 2, 4 & 6. The elastic periods were 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.5, 2.0, 2.5, 3.0 & 4.0s. This gave a total of 14 design displacements and 56 designs.

The 14 design displacements found in this process were used with the DDBD method to determine the required strengths. In this process the yield displacements were varied so that ductility levels of 1, 2, 4 & 6 were achieved. As with the FBD there was a total of 56 designs. This process was carried out twice, firstly with the damping for the associated elastic model being based on equivalent damping values and secondly with substitute damping values.

The process described above was repeated with the bilinear model being replaced by the column model. Time history analyses were then made for all the 336 designed structures using the appropriate hysteretic model and the four earthquake ground motions (group 2) that were nomalised to the NZ Loadings Standard response spectrum. This gave a total of 1344 analyses. For each analysis the ratio of the maximum time history deflection to the design deflection was calculated. The results for all the analyses are shown for individual sets in Figures 5, 6, 7 & 8. One point stands out from these figures and that is that the use equivalent viscous damping with the bilinear model gives a very poor prediction of displacement particularly in the low deflection (period) range. However, with the column model it appears to give reasonable predictions.

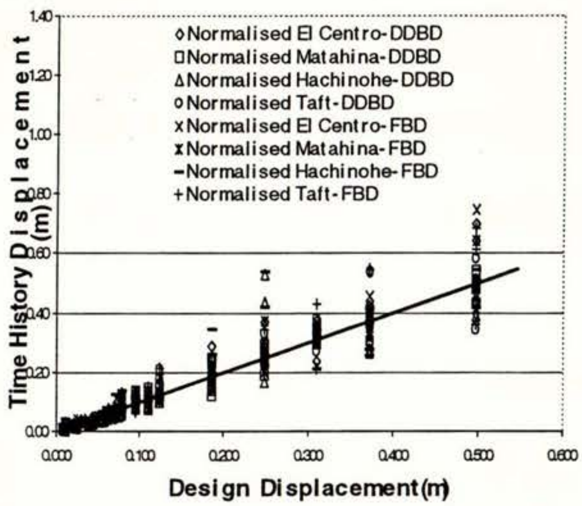

Figure 5 – T/H vs. Design Displacement Values (DDBD / Substitute – FBD) – Bilinear Model

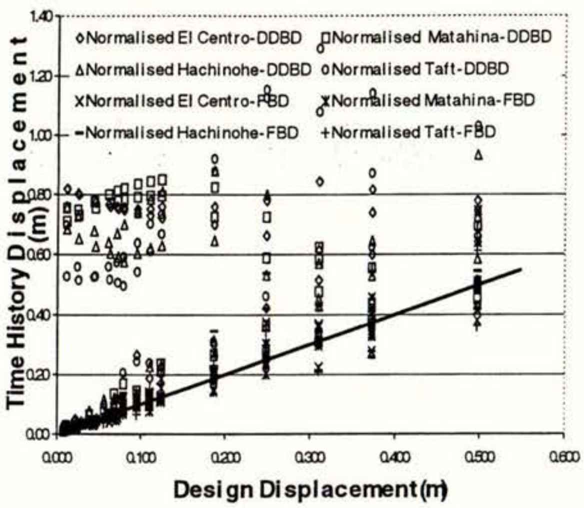

Figure 6 – T/H vs. Design Displacement Values (DDBD / Equivalent – FBD) Bilinear Model

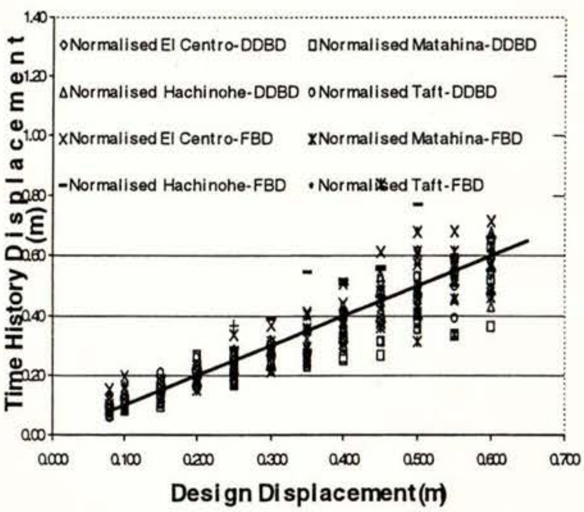

Figure 7 – T/H vs. Design Displacement Values (DDBD / Substitute – FBD) – Column Model

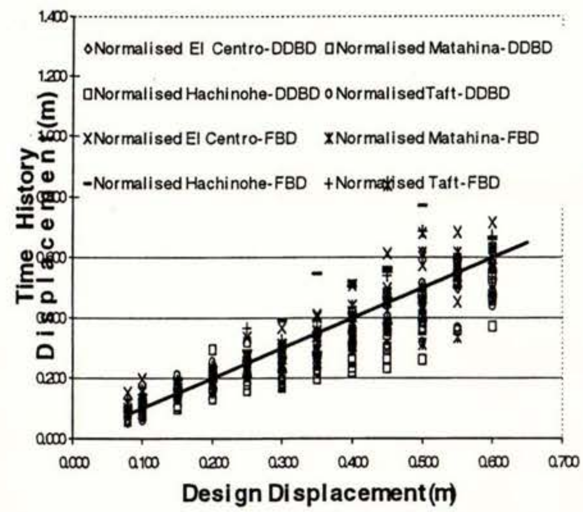

Figure 8 – T/H vs. Design Displacement Values (DDBD / Equivalent – FBD) Column Model

Table 2 summarises the results of all the analyses. From this it can be seen that direct displacement based design with substitute damping gives the best overall deflection predictions of the three methods. The use of equivalent viscous damping instead of substitute viscous damping reduces the accuracy of prediction. For the bilinear hysteretic response this substitution leads to inaccurate values and it clearly should be avoided, or, possibly as an alternatively empirical coefficients should be introduced to modify the damping values that are used. The margin in accuracy between force based design and direct displacement based design with substitute damping is not large, particularly for structures with periods greater than 0.7s.

Table 2: Average design results (T/H displacement/ design displacement ratio), for the normalised earthquake records

Hysteretic Model	Design Displacement (m)	DDBD - Substitute		DDBD - Equivalent		FBD	
		Avg	CoV	Avg	CoV	Avg	CoV
Bilinear	$\Delta \leq 0.079$	0.82	0.08	12.16	11.01	1.19	0.42
	$0.079 \leq \Delta \leq 0.500$	1.03	0.06	2.98	1.36	1.11	0.08
Column	$\Delta \leq 0.079$	0.99	0.04	0.91	0.02	1.31	0.14
	$0.079 \leq \Delta \leq 0.500$	0.91	0.08	0.84	0.09	1.06	0.12

6 CONCLUSIONS

1. Substitute viscous damping values have been derived from a range of different types of earthquake ground motions. The differences between these values are found to be relatively small. For design purposes it is possible to give values in tables or in graphical form.
2. Substitute damping values are found to depend strongly on ductility level. For period range between 0 and 0.7s there is sharp increase in the damping value with period. Above the 0.7s the variation depends on the ductility level. For a ductility of 2 the substitute damping value increases with period, for a ductility level of 4 there is little variation and for a ductility of 6 the substitute damping values decrease with period.
3. There was little difference between substitute damping values found with the column and beam hysteretic models. Both models represent stiffness degrading structures but the beam model has a pinched hysteretic typical of that found in structures which form reversing plastic hinge zones in reinforced concrete beams.
4. The comparison of the maximum displacements from time history analyses with design displacements obtained by the different methods indicates that
   - displacement based design with substitute viscous damping gives the best deflection predictions
   - use of equivalent viscous damping instead of substitute viscous damping with displacement based design reduces the accuracy and gives misleading values where the hysteretic response is bilinear or close to bilinear
   - the difference in accuracy obtained by direct displacement based design with substitute viscous damping and force based design is not very significant.
5. The use of substitute viscous damping with displacement based design overall gives more consistent deflection predictions than the use of equivalent viscous damping. This substitution also avoids the need for the empirical manipulations of equivalent viscous damping values, as has been suggested by Loeding et al (1998).

ACKNOWLEDGEMENTS

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Displacement Focussed Force Based Seismic Design

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Abstract

The paper explains how the currently used seismic design method, "Force Based Design", can be modified to allow the method to be "focussed" on displacements. To compare the ability of the proposed method, the "Displacement Focussed Force Based Design" with the "Direct Displacement Based Design" method to provide structures whose seismic response best fits chosen displacements, a number of single degree of freedom structures were designed. The maximum displacement responses of each structure from code compatible earthquakes were then calculated from nonlinear time history analyses. The mean of the displacements calculated for structures designed to both methods were not significantly different.

1. Introduction

The force based design (FBD) method is well established as the procedure used to assign the required strength to structural members in seismic design. The method requires the designer to check the magnitude of the expected structural displacements, but not to focus the design on them. As it is widely accepted that damage to a structure is closely related to the magnitude of the displacements sustained, it is important that design methods seek to control displacement levels. The direct displacement based design (DDBD) method has been recently proposed as an alternative design procedure that focuses on the displacement profile of the structure. Although the intention of the method is worthy, one of the difficulties of its introduction is that it requires a retraining of current designers. This paper offers an alternative compromise, where a small additional step is introduced into the existing well known force based method, to focus the design on to structural displacements.

2. Force Based Design

Before introducing the proposed modification to the force based design method it is worthwhile to review the method as it is currently implemented, commenting on some of its weaknesses.
The following are the basic steps in the force based design method.
(i) A structural form and material are selected, for example a ductile reinforced concrete frame. Following this decision, the codes provide guidance to the maximum displacement ductility (μ) that may be used in the design. Unfortunately structural ductility gives only a poor measure of material strains and section curvatures, and in irregular structures checks of strain levels in critical members may be required.
(ii) Member stiffnesses are estimated. These will be used for an elastic analysis of the structure to obtain member actions and the expected maximum deflections of the structure for the design level earthquake. For steel structures, these values are straight forward. However, for reinforced concrete members an effective stiffness is required and codes are used for guidance. This is a topic for debate as different codes provide different advice. For example, the (New Zealand Concrete Standard, 1992) recommends that the flexural stiffness of a rectangular reinforced concrete beam should be taken as 0.4 I_{gross} where the (Eurocode 8, 1998) suggests that the full section stiffness should be used in the analysis. Recently (Priestley, 1998), (Kowalsky and Priestley, 2000) have called for the results from the latest research to be used, where it is claimed that for concrete members, "stiffness is proportional to strength" and not to gross section properties.
(iii) The period of vibration is calculated. For

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regular structures, where T<2s, the "equivalent static method" of analysis may be used and for this method the period of the first mode of vibration (T) is calculated from a "Rayleigh" equation. For more complex structures, the mode shapes and natural periods of vibration are calculated.

(iv) The base shear for the structure is calculated. For the "equivalent static method" the (New Zealand Loadings Standard, 1992) provides equation (4.8.1),

$$V = C\, W_t \qquad (1)$$

where C is the lateral force coefficient whose value depends upon a risk factor, the soil type, the seismic hazard, the period and the chosen structural ductility and W_t is the seismic weight of the structure. In essence, C is a scaled ductility "μ" acceleration spectrum. For the New Zealand Loadings Standard, the ductility μ spectrum $C_h(T,\mu)$ is obtained by multiplying the 5% elastic uniform hazard spectrum for the site by:

$1/\mu$ for T>0.7sec
$1/[(\mu - 1)\, T/0.7 + 1]$ for 0.45< T < 0.7
$1/[(\mu - 1)\, 0.45/0.7 + 1]$ for T< 0.45 .

For a response spectrum analysis an initial base shear is calculated using the uniform hazard spectrum. The code then requires the results of this analysis to be scaled so that the base shear is not less than a specified proportion of the equivalent static base shear. For irregular structures, in the elastic range, this approach provides for a more accurate distribution of member actions than the equivalent static method.

(v) For the equivalent static method, a linear analysis is performed where "equivalent lateral inertia forces" are applied to the structure. The sum of these forces is equal to the base shear. For both the equivalent static and response spectrum methods of analysis, the member actions that result are the required strengths of the critical members. The expected deflection profile of the structure for a design level earthquake is obtained by multiplying deflections from these analyses by the ductility factor. The deflections and structural actions are increased to allow for P-delta effects.

(vi) The scaled displacements found in (v) are compared with code allowable values. Typically, if the code values are exceeded, members are stiffened and the analysis is repeated. If the displacements are satisfactory the designer proceeds with the detailed design of the members where capacity design procedures are used to ensure that the structure behaves in a predictable way.

2.1. Discussion

A number of criticisms of the force based design method have been expressed, (Priestley, 1997) and a new design method, the "Direct Displacement Based Design (DDBD)" method has been proposed in an attempt to resolve them (Kowalsky and Priestley 2000). This is not the only approach and many inadequacies of the force based design method can be addressed by minor modifications to the current method, where the introduction of a new design approach would require a major re education of designers. The introduction of a new design method appears to be undesirable unless it provides additional advantages, for example by developing safer designs, more economic solutions or a faster design process.

It is not the intent of this paper to spell out possible solutions to all of the inadequacies of the force based design approach. However, the authors believe that there are rational solutions available. Two of the main areas that give rise to inconsistencies in the force based method are in the choice of ductility factor, and for reinforced concrete structures, the selection of effective member stiffness.

(i) Whether the code directed choice of ductility factor is suitable for the structure is an issue that should be reviewed by the engineer during the design process. It must be remembered that structural ductility is the "system" or "effective" ductility. The choice of this factor should be controlled by material strain demands of critical members. A simple example of how the use of code values without further consideration of the structural form can lead to incorrect use of the ductility factor is in the design of is a wall structure on a flexible foundation. The code, (Standards, 1992) allows a structural ductility of "5" for walls. If however as illustrated in Fig. 1, the effect of the foundation flexibility is to increase the yield displacement of the system to twice that which would occur if the base of the wall was "fixed", then the allowable structural ductility would be reduced to "3". In addition there would be an increase in period resulting from the reduced initial stiffness of the system. Consequently the base shear could be significantly different from that calculated for a fixed base wall structure. To determine the allowable structural ductility for a non standard structural form is straight forward. For the flexible base wall example, a linear analysis could be performed with and without the foundation flexibility to determine the wall contribution to the ductility one displacement of the system. The total allowable displacement of wall with the flexible foundation is then the ductility one displacement of the system plus four times the wall ductility one displacement (assuming the wall is to be detailed suitable as a ductility five structure).

(ii) The other issue, that of the relationship between stiffness and strength is already addressed by designers of steel structures. To implement a procedure for reinforced concrete design requires the initial stiffness required in step (ii) of the FBD method to be calculated from an estimated steel reinforcement layout. Then the member strengths calculated in step (v) must be compared with the estimated reinforcement. If these are not sufficiently close, the stiffness estimates for step (ii) can be adjusted and steps (iii) to (vi) repeated.

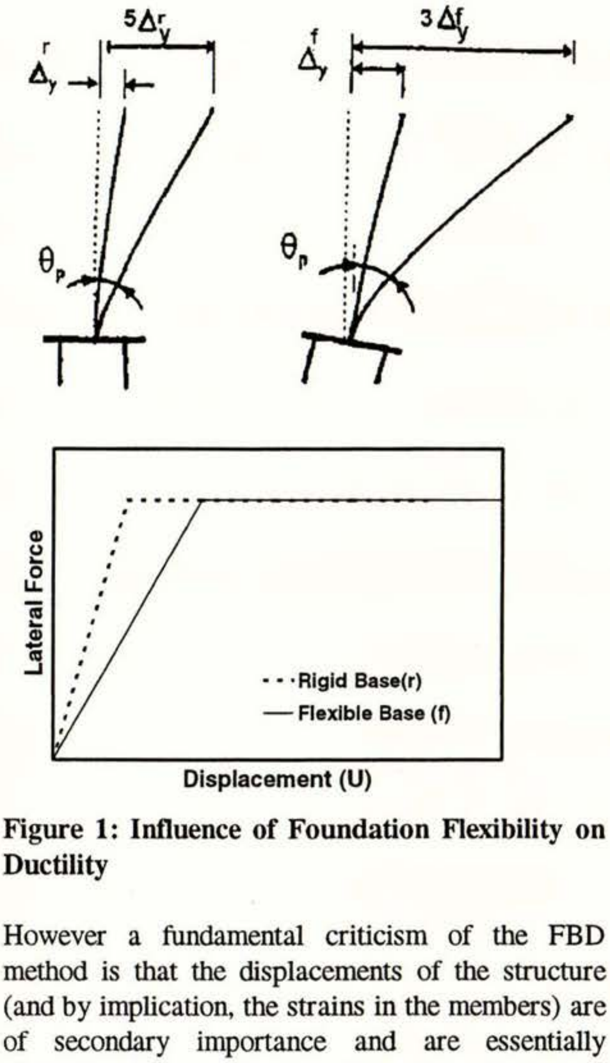


Figure 1: Influence of Foundation Flexibility on Ductility

However a fundamental criticism of the FBD method is that the displacements of the structure (and by implication, the strains in the members) are of secondary importance and are essentially checked against code allowable values at the last step of the design process. Displacements, it is contended by critics of the method, reflect the level of damage in a structure and the design process needs to be focussed on them. Further, if the design method is focussed on developing structures that have an equally likely chance of damage during an earthquake, the building stock would behave more predictably. The authors agree with this contention.

3. The Direct Displacement Based Design Method

To address the required focus on displacements as a design parameter the "direct displacement based design (DDBD)" method (Kowalsky and Priestley 2000) devises a new approach whereby the acceptable displacement of an equivalent single degree of freedom (SDF) structure, Δ_d, is selected. The magnitude of this displacement is determined after consideration of strains and curvature limitations of critical members in the structure. In many cases code interstorey deflection criteria may also govern the magnitude of this displacement. As for the force based design procedure the proposal of an equivalent single degree of freedom structure allows the use of design response spectra. In contrast with the FBD method where the lateral force coefficient (C) of a ductile single degree of freedom system is used to obtain the base shear of the structure, the DDBD method equates Δ_d to the displacement of an elastically responding oscillator. The damping of the oscillator is defined as "equivalent" and its value depends upon the estimate of the ductility of the structure and its dominant hysteretic form. A typical graph providing this "translation" is shown in Fig. 2(a). With the magnitude of "equivalent" damping estimated, a secant period of the equivalent (SDF) oscillator (T_s) can be found from a "displacement" design spectrum as shown in Fig.2(b). An estimate of the base shear of the structure is simply the stiffness associated with that period

$$K_s = W_t/g\,(2\pi / T_s)^2 , \qquad (2)$$

multiplied by Δ_d. In most designs, an iterative process is required to ensure that initial estimate of the structural ductility is consistent with the calculated base shear. A full description of the method with worked examples is presented in (Kowalsky and Priestley, 2000).

In essence the method starts with a structural displacement and provides the designer with an appropriate base shear and a set of member design strengths. The force based method as currently used, provides the designer with the base shear and member strengths from an estimate, or code advised, ductility factor. In this method there is no focus on the expected structural displacements and they are typically only checked for code exceedence.

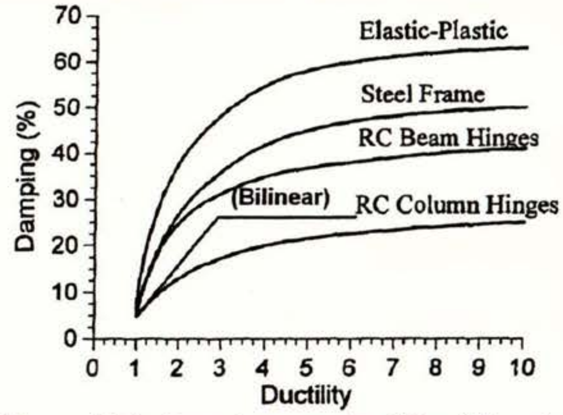


Figure 2(a) Damping vs ductility (Kowalsky, 2000)

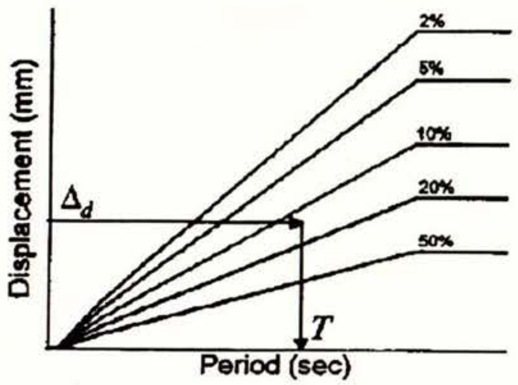


Figure 2(b) Schematic of Displacement Spectra (Kowalsky, 2000)

4. The Displacement Focussed Force Based Design Method

As the FBD method is used regularly by structural designers it seems reasonable to provide a minor modification to the existing process to "focus" the method on displacements. The proposed modification requires an additional two steps to be inserted after step (iii). These are:

(iii a) Determine the magnitude of the displacement Δ_d taking into account member strain and curvature demand limitations. This is consistent with the recommendations in the DDBD method.

(iii b) Check whether the stiffness of the structure satisfies the limiting displacement. This is achieved by use of a displacement spectrum consistent with the FBD method. Displacement spectra that are consistent with the New Zealand Loadings Standard lateral force coefficient for intermediate soils are shown in Fig 3. These are developed by multiplying the elastic or ductility one values by $g^*(T/(2\pi))^2$.

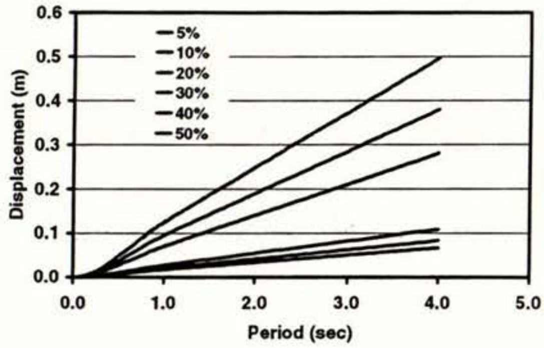


Figure 3 Displacement Spectra for NZ Code

To use these spectra the period of the structure, T, calculated in step (iii) must be compared with T_c, the period associated with Δ_d/μ. If it is longer then the structure is too flexible and must be stiffened. If the period is shorter then the structure will presumably satisfy minimum drift requirements but the designer may be able reduce the stiffness (and strength) of the members and obtain a more economical solution. It is important to note that for displacements where Δ_d/μ is greater than 0.08m, (T is greater than 0.7 seconds), the FBD methodology infers that these decisions are independent on the choice of structural ductility factor. However if Δ_d/μ is less than 0.08m, in theory, the designer has the additional flexibility of choosing an alternative ductility for the structure. However, for design purposes the displacements at different levels of ductility are practically the same.

Steps (iv), (v) and (vi) then follow as before. It should be noted that an equivalent single degree of freedom displacement can be calculated from the scaled displacements obtained in step (v), and it should agree with the choice made in step (iii b). For multi storey buildings, interstorey drift needs specific attention as the equivalent single degree of freedom displacement used in this method (and the DDBD method) does not take into account the effects of torsion of the structure which would be included if a three dimensional response spectrum analysis was performed in step (iii).

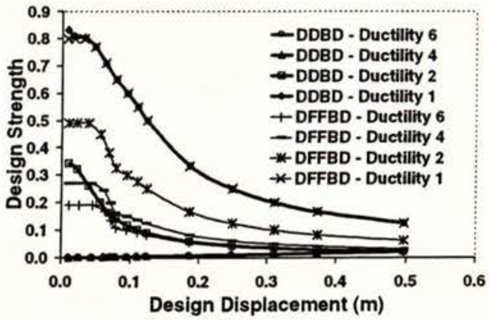
5. Essential Differences between the Methods

Although from an initial observation there appears to be a number of differences between the proposed "displacement focussed force based" and "direct displacement based" design methods all but one of these are superficial and adjustments to either method could be made to make them the same. The essential difference between the two methods is the "translation" that each uses, that allows response spectra to provide the expected structural strength (base shear) required to ensure the structure meets the designer's displacement criteria.

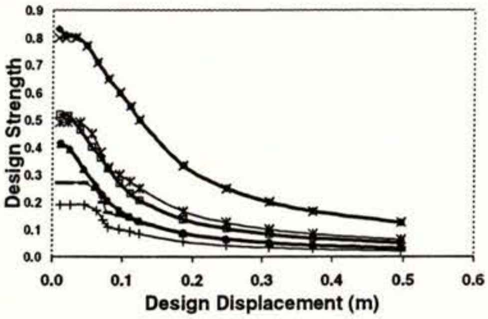
6. Comparison of Designs

To compare the ability of the two design methods to provide structures whose seismic response best fits the chosen representative displacement, a number of single degree of freedom structures were designed. These were designed to the New Zealand Loadings Standard assuming their nonlinear behaviour was bilinear, which is consistent with development of the code lateral strength coefficients, "C". The maximum displacement response of each structure from a code compatible earthquake was then calculated from a nonlinear time history and recorded.

Fifty six structures were designed using each design method. The design displacements were chosen so that they matched the spectral displacements for the periods 0.2, 0.3, 0.4 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5 and 4.0 seconds and ductilities 1, 2, 3, and 4 in Fig 3. The associated design strengths for the DFFBD method are obtained from Table 4.6.1, (Standards, 1992). The strengths of the structures designed using the DDBD method were obtained using the "elastic – plastic" line in Fig. 2(b) and the "(bilinear)" line of the same figure. The first of these lines are the result of implementing the "equivalent" damping theory, the second is an empirical recommendation for code implementation, (Priestley, 1998b). The strengths resulting from these designs are illustrated in Fig. 4. It has been found that the equivalent damping transformation is inconsistent for bilinear hysteretic forms (Judi et al, 2000, 2001) and so it is not surprising that there are anomalies in the required strengths calculated for the structures using DDBD with equivalent damping as shown in Fig.4(a). The strength requirements for the structures designed to both the DFFBD and the DDBD (using the code recommendations for damping) methods as shown in Fig. 4(b) these values are similar. However, there is a trend showing those designed using the DFFBD method require less strength, particularly at the higher ductility levels.



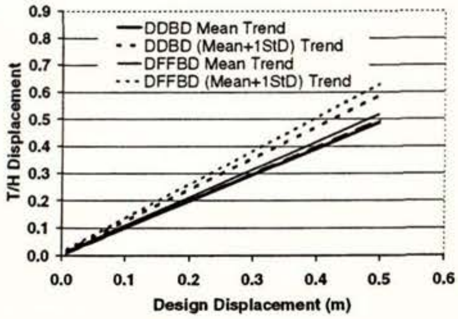
(a) "Equivalent Damping" and DFFBD



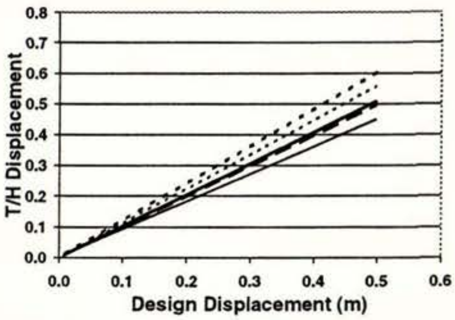
(b) "Empirical Damping" and DFFBD

Figure 4 Comparison of Design Strengths

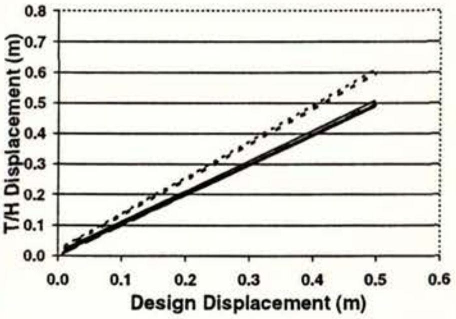
The response of each of the 112 designs described in Fig. 4(b) was calculated for four different ground motions. These earthquakes had 5% damped spectra that closely matched the New Zealand Loadings Standard, (1992) intermediate soil lateral force spectrum "C". The results of these analyses are summarised in Fig. 5, where the displacements resulting from the nonlinear analyses are plotted against the design displacements. Fig. 5 (a) presents mean and mean plus one standard deviation plots for all displacements resulting from both design methods. This plot shows little difference between the two approaches. It should be noted that the magnitude of the variance in the results for either of the methods is largely a function of the variability between the characteristics of the chosen earthquakes. Figs. 5 (b) – (d) illustrate the accuracy of the methods to control displacements at different ductility levels. It appears that the "transformation" chosen by the New Zealand Loadings, Standard, (1992) is a little conservative for structures designed for lower ductilities and possibly a little non conservative for structures designed for higher ductilities. The displacement trends of the structures designed with DDBD procedure appear to be the reverse.



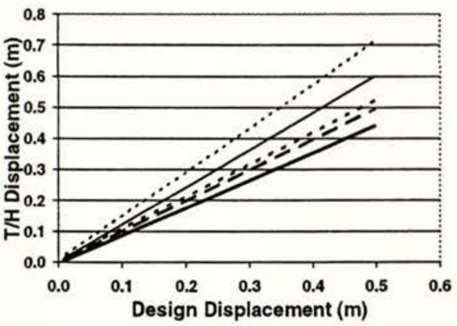
(a) Trend over all ductilities



(b) Trend for ductility 2



(c) Trend for ductility 4



(d) Trend for ductility 6

Figure 5 Calculated Displacements vs Design Displacements

7. Discussion and Conclusions

It is contended in this paper that the essential difference between the force based design procedure and the direct displacement based design procedure is the "translation" that each uses to enable response spectra to be used to provide the basic required design strength of a structure. It is accepted that a focus on displacements in seismic design is desirable and it is proposed that additional steps be introduced into the current force based design method is a practical way to improve this well established approach. Fifty six simple single degree of freedom structures were designed to the spectrum of the New Zealand Loadings Standard, (1992) for a intermediate soil site using both design methods. Nonlinear time history analyses of these structures to four normalized earthquakes allowed a comparison of calculated displacements and expected design displacements. The mean of all of the results showed little difference between the methods. A breakdown of the results by ductility level showed that the FBD method was a little conservative at low ductility and possibly non conservative for high ductility structures. The trends for the DDBD were the reverse.

8. ACKNOWLEDGMENTS

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Influence of Hysteretic Form on Seismic Behaviour of Structures

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Conference

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ABSTRACT: In the seismic design spectra in the Loadings Standard no recognition is given for the influence of differing hysteretic behaviour associated with different materials, structural types and detailing standards. In practice hysteretic behaviour varies widely, from the near bilinear typical of eccentrically braced frames to the pinched forms associated with masonry.

To investigate this a series of time history analyses were made with different ground motions and a range of hysteretic models. It is shown that the hysteretic form, provided that appreciable energy can be dissipated, has only a small influence on the maximum displacement that is sustained. Varying the viscous damping level was found to have a significant influence on elastically responding structures but a smaller effect on structures with some ductile capacity. Increasing the strain-hardening ratio was found to have a small influence on the required strength for a given ductility.

1.0 INTRODUCTION

The form of hysteretic response of structures varies very significantly with the structural form, materials, detailing and characteristics of the foundation. However, in the Loadings Standard [SANZ, 1992], and many other seismic codes of practice, only one set of response spectra are given to cover all hysteretic forms. Recently a number of proposals have been made to adopt a different method of seismic design, namely Displacement Based Design. These approaches, of which Direct Displacement Based Design [Priestley and Kowalsky, 2000] is one, recognise the influence of hysteretic form on seismic response. With this method structures which develop pinched load deflection response under cyclic loading are designed for a higher strength level than structures with a near bilinear response. This paper sets out to examine the influence of hysteretic form, viscous damping and strain hardening percentages on seismic response.

2.0 HYSTERETIC FORMS

As illustrated in Fig. 1 the hysteretic form varies with the materials and structural type. Fig. 1 (a) shows a near bilinear response, which was obtained from a test of a shear-yielding element for an eccentrically braced frame [Popov et al, 1987]. There was very little degradation on repeated loading cycles. As larger displacements were applied strain hardening increased the maximum load sustained. A bilinear model provides a conservative representation for this behaviour.

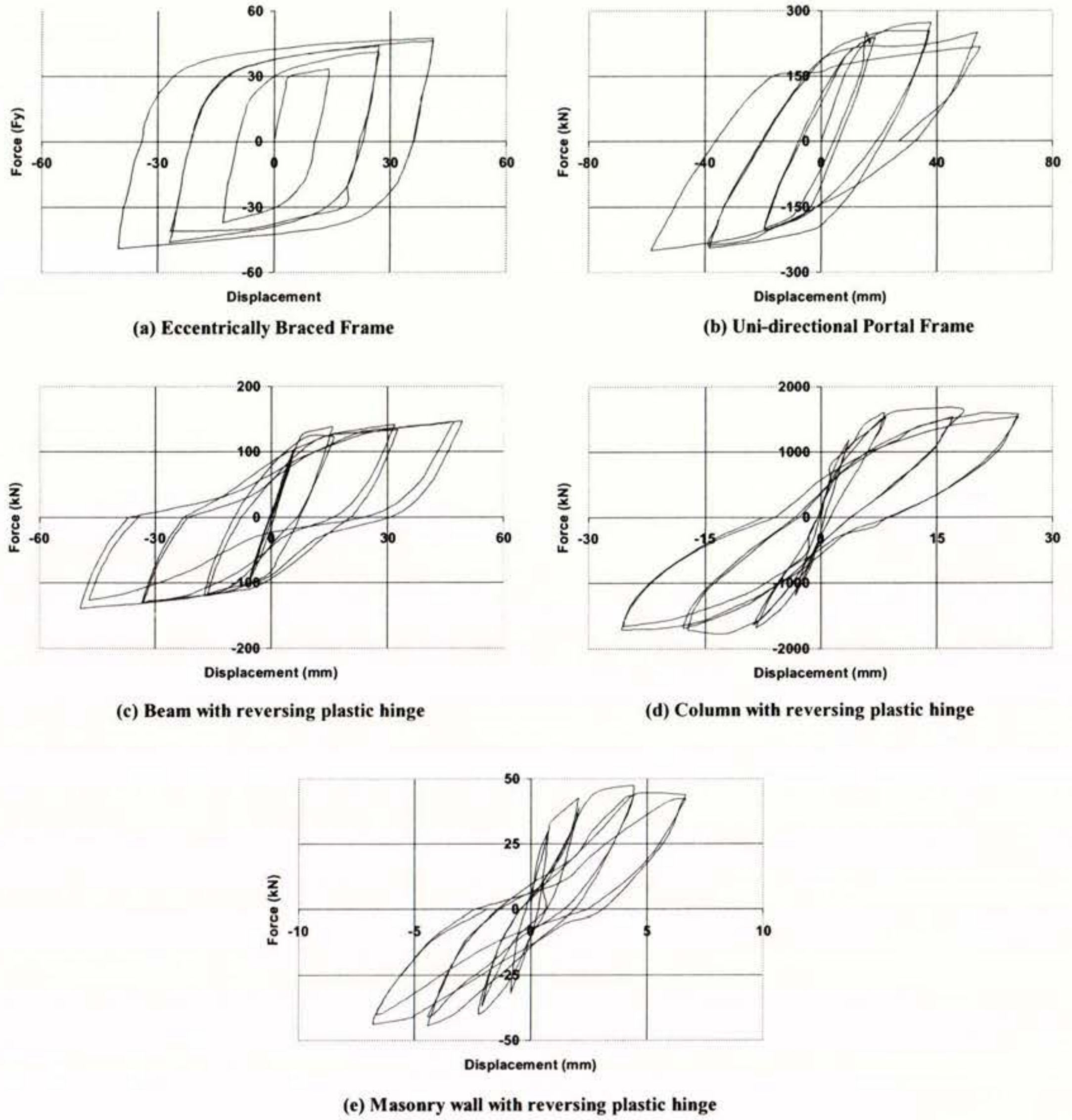


Fig. 1 Force displacement relationships for different structural elements

A similar hysteretic shape is obtained from reinforced concrete frames, which develop unidirectional plastic hinges [Megget and Fenwick, 1989] when subjected to inelastic cyclic loading. The lateral force versus displacement for such a case is shown in Fig. 1 (b). As the displacements are increased there is a limited amount of stiffness degradation. However, until the reinforcement buckles the response can be realistically represented by a bilinear model.

Fig. 1 (c) shows the force versus displacement response of a reinforced concrete beam subjected to cyclic inelastic loading [Fenwick et al, 1981]. In this case a reversing plastic hinge formed. The shear reversal results in a pinched response, which arises due to the yielding of the stirrups and the opening and closing of diagonal cracks in the plastic hinge. Under repeated loading to the same displacement some stiffness degradation occurs as additional yielding of the stirrups takes place. It should be noted that the unloading curves remain relatively steep, with little recovery in deflection when the load is removed.

Fig. 1 (d) shows the lateral force versus displacement for a reinforced concrete column subjected to cyclic loading [Gill, 1979]. In this case the column sustained an axial load of

"$0.21A_gf'_c$". There are two marked differences in the behaviour of columns as compared to beams. The first is that the axial load reduces the magnitude of the shear resisted by the stirrups. This limits the yielding of the stirrups and the width of the diagonal cracks. Consequently pinching of the response curve is greatly reduced. The second difference is in the stiffness of the unloading curves. With beams this stiffness is high, as the cracks that form due to tension yield of the reinforcement remain open when the load is removed. With the column the axial load helps to close these cracks when the lateral force is removed, and as a result there is greater recovery in displacement.

Fig.1 (e) shows the lateral force displacement response for a masonry wall [Brammer, 1995]. The relationship has some similarity to that of the reinforced concrete beam, which forms a reversing plastic hinge. However, with the masonry wall the stiffness degradation of the loading curves is higher as a result of greater shear deformation.

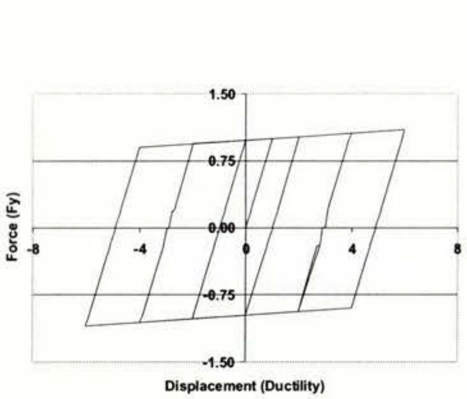
All the load deflection relationships shown in Fig. 1 dissipate appreciable energy by hysteretic behaviour of the materials. In this respect they are different from two further extreme cases. The first of these is for prestressed concrete members reinforced with unbonded cables. The response in this case is typically S shaped, with the loading and unloading curves lying very close to each other. Analyses using analytical models representing this behaviour indicate that the lack of energy dissipation increases the ductility demand typically by 50 percent when compared to a bilinear response [Priestley and Tao, 1993, Ouzounova, 1998]. The other extreme is for a tension braced pin jointed frame. A limited amount of energy may be dissipated by these structures with the diagonal bars yielding in tension. However, when load reversals occur the diagonal bars buckle and the stiffness drops to close to zero.

3.0 HYSTERETIC MODELS

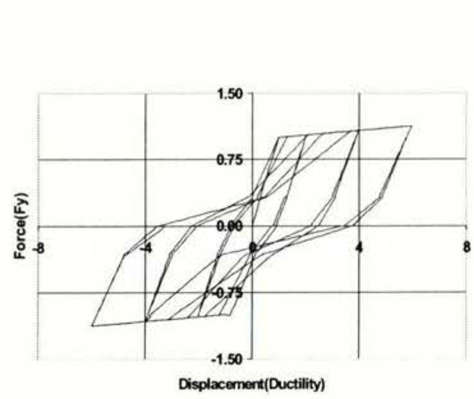
A basic hysteretic model has been developed for a single degree of freedom analysis program [Fenwick and Davidson 1994]. The loading relationship is represented by three straight lines, of which the third line represents strain hardening relationship. A further two lines represent the unloading relationship, so that a half cycle of loading involving inelastic deformation is represented by five changes of stiffness. The gradient of each line is controlled by coefficients, which vary with both the maximum displacement that is reached and the sum of the inelastic displacements sustained along the strain hardening lines. This basic model allows a wide range of hysteretic relationships to be represented. In particular it allows stiffness degradation to be modelled under cyclic loading conditions where the previous maximum displacements sustained in each direction are not exceeded.

The coefficients defining the gradients of the five lines have been chosen to give four different models for the analyses described in this paper. They are described below.

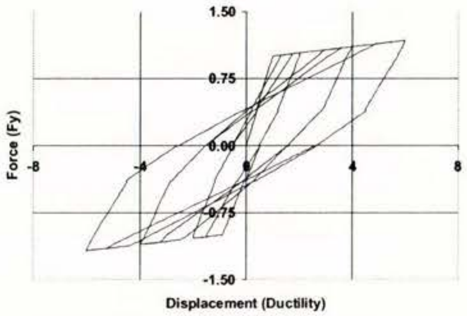
1. Bilinear model is illustrated in Fig. 2 (a). This is a standard model.
2. Beam model is illustrated in Fig. 2(b). The coefficients were derived from the results of reinforced concrete beams tested under cyclic loading such that they formed reversing plastic hinges [Fenwick et al 1981]. This relationship is representative of the lateral force displacement response of well detailed ductile concrete moment resisting frame structures, which form reversing plastic hinges. Pinching of the load deflection response and high unloading stiffness values are characteristics of this model.
3. Column model is illustrated in Fig.2(c). The coefficients were derived from test results of a reinforced concrete column [Ang, 1981], which sustained an axial load of $0.2\ A_g\ f'_c$. As previously explained this gives unloading stiffness values that decrease as the magnitude of inelastic displacement increases.



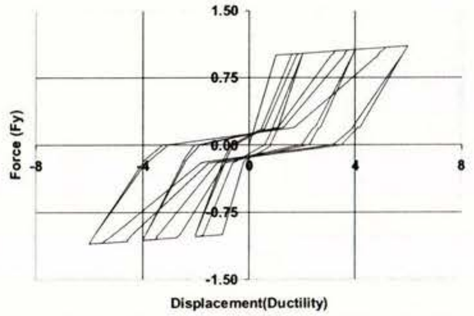
(a) Bilinear model



(b) Beam model



(c) Column model



(d) Masonry Model

Fig. 2 Analytical hysteretic models

4. The masonry model was developed from test results obtained from a wall test [Brammer, 1995]. As illustrated in Fig.2 (d) this is similar to the beam model but with greater pinching of the load deflection relationship.

These four models cover a wide range of hysteretic shapes met in practice. However, the extremes of prestressed concrete members with un-grouted cables and pin jointed tension braced frames have been excluded.

4 ANALYSES

The analyses were made using a series of twelve earthquake ground motions, which are identified in ATC 40 [1996] as suitable candidates for time history analyses. All these records were recorded at sites with stiff to medium ground conditions located at least 10km from fault rupture. The magnitude of each event was not less than 6.5 and the peak ground acceleration was at least 0.2g. There are several records for each earthquake event.

Loma Prieta Earthquake, Oct. 17th, 1989

1. Hollister, South Street and Pine drive, channel 1-90°
2. Hollister, South Street and Pine drive, channel 3-0°
3. Gilroy #2 Hwy, Bolsa Road Motel, Channel 1-90°, Gavilan, College Water Tank
4. Gilroy #2 Hwy, Bolsa Road Motel, Channel 3-0°, Gavilan College Water Tank

Landers Earthquake, June 28th. 1992.
5 Joshua Tree fire Station, channel 1-90°
6 Joshua Tree fire Station, channel 3-0°
7 Yermo Fire station, channel 1-360°
8 Yermo Fire station, channel 3-270°

Northridge Earthquake, 17th. Jan., 1994
9 Moorpark, channel 1-180°
10 Moorpark, channel 1-90°
11 Century City, Lacc North, channel 1-90°
12 Century City, Lacc North, channel 3-360°

In each group single degree of freedom structures were analysed for the 12 earthquake ground motions for 48 different structures. These had initial elastic periods of 0.3s to 5s with 0.1s steps. The analyses were made to determine the required strength for nominated ductility levels. In different sets of analyses the effect of changing the hysteretic model, the level of viscous damping and the strain hardening values were examined. Details are given in the next section.

5.0 RESULTS OF ANALYSES

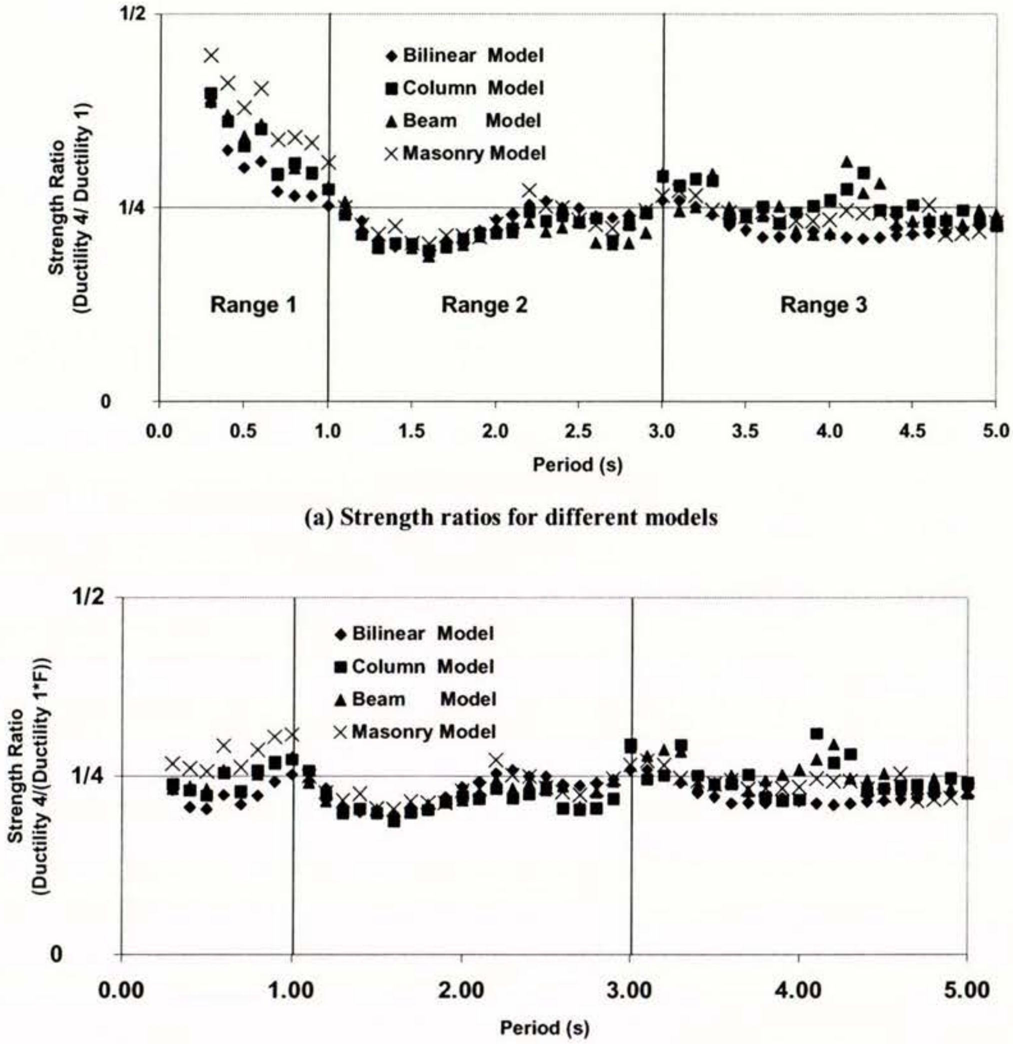
5.1 Influence of hysteretic form on response

In this set of analyses the strength required for ductility levels of 2, 4 and 6 were determined with all 4 hysteretic models. All the structures were given a viscous damping level, ξ, of 5 percent and a strain hardening ratio, α, of 2.5 percent. The required strength for each individual period, ductility level and earthquake ground motion was divided by the corresponding strength required for elastic response. The values obtained for each period and specified hysteretic model were averaged for the 12 ground motions. Fig. 3 shows the results obtained for ductility 4. Very similar relationships were obtained for ductility levels of 2 and 6.

Fig. 3 (a) shows that the strength ratio for structures with a period greater than 1 second is close to the inverse of the ductility ratio, for all the hysteretic models. This corresponds to the equal displacement concept. For periods below the 1second value the required strength ratio is greater than that that implied by the equal displacement concept. The current NZ Loadings Standard [SANZ, 1992] recognises this and the specified strength for structures with initial fundamental periods in the range of 0 to 0.7 seconds is increased above the level implied by the equal displacement concept.

One approach is to assume the required strength can be defined as the strength given by the equal displacement concept times a factor, F. As indicated in Fig. 3 (a) the value of "F" is close to 1 for the initial period in excess of 1 second. Values of the factor "F" are implied by the Loadings Standard in Table 4.6.4, and these have been calculated and listed in Table 1. However, an alternative set of values for "F" is also proposed. From Fig.3 (a) it can be seen that the increase in strength ratio is close to linear between a period of 1 and 0.3 seconds, with the increase corresponding to close to 1/6. A similar value occurs in the ductility 6 results and a slightly smaller increase occurs with the ductility 2 values. Assuming the value of 1/6 applies to the three ductility levels the proposed values for "F" have been found and listed in Table 1. Taking the results, such as those shown in Fig. 3 (a), and dividing by the appropriate values of "F," should ideally, lead to a uniform normalised value of strength ratio with period. The results for ductility 4 using the proposed values of "F" are shown in Fig. 3 (b). It can be seen that for practical purposes the desired result is obtained.

To enable the many analytical values to be interpreted the structures were divided into 3 groups, depending on the periods. Ranges 1, 2 and 3 were for structures with periods in 0.3 to 1s, 1.1 to 3s and 3.1 to 5s respectively. For the range 1 values the strength ratios were divided by the "F" factors and listed in Table 2, with the values in brackets corresponding to the "F" factors calculated from the Loadings Standard. The averaged values for each range are shown in Table 2.



(a) Strength ratios for different models

(b) Strength ratios normalised by factor (F)

Fig. 3 Influence of hysteretic model on strength ratio

Table 1: Values of factor F as defined by the Standard and as proposed

Period (s)	From Standard Ductility			As Proposed Ductility		
	2	4	6	2	4	6
0.3	1.22	1.36	1.44	1.33	1.67	2.00
0.4	1.22	1.36	1.44	1.29	1.57	1.85
0.5	1.16	1.28	1.31	1.24	1.48	1.71
0.6	1.08	1.12	1.14	1.19	1.38	1.57
0.7	1.00	1.00	1.00	1.14	1.29	1.43
0.8	1.00	1.00	1.00	1.09	1.19	1.29
0.9	1.00	1.00	1.00	1.05	1.09	1.14
1.0	1.00	1.00	1.00	1.00	1.00	1.00

From Table 2 and Fig. 3 it can be seen that for the three ductility ratios and the bilinear, beam and column models the strength ratios were almost identical. On average the masonry model requires an increase of 8 percent more than for the other models. The general conclusion is that the form of hysteretic model made little difference to the strength.

Table 2: Average strength ratios normalised by "F"

ξ%	α %	μ	Range *	Bi-linear	Beam	Column	Masonry	Av. All	Code
5.0	2.5	2	1	0.434	0.453	0.454	0.486	0.457	
				(0.443)	*(0.463)*	*(0.462)*	*(0.495)*	*(0.466)*	0.5
			2	0.449	0.433	0.432	0.460	0.444	0.5
			3	0.469	0.459	0.465	0.480	0.468	0.5
5.0	2.5	4	1	0.244	0.247	0.248	0.280	0.255	
				(0.228)	*(0.253)*	*(0.252)*	*(0.286)*	*(0.255)*	0.25
			2	0.231	0.218	0.224	0.232	0.226	0.25
			3	0.222	0.232	0.256	0.244	0.239	0.25
5.0	2.5	6	1	0.147	0.169	0.169	0.192	0.169	
				(0.149)	*(0.176)*	*(0.175)*	*(0.200)*	*(0.175)*	0.167
5.0	2.5		2	0.156	0.147	0.159	0.167	0.155	0.167
5.0	2.5		3	0.143	0.176	0.188	0.177	0.171	0.167

* Range 1 = 0.3 –1s; Range 2 = 1.1-3s and Range 3 = 3.1 to 5s

5.2 Influence of damping ratio on strength

The analyses described in the first set were repeated but the viscous damping level, ξ, was set first to 0.5 percent and then to 2 percent. The required strength found in each analysis was divided by the corresponding strength for the same earthquake record and ductility level obtained with 5 percent viscous damping. The results are summarised in Table 3, together with values calculated from equations given by both Kawashima [1995] and Eurocode 8 [1996].

From Table 3 it can be seen that decreasing the damping below 5% increases the strength required. For the elastically responding structures in the period range 1 (0.3 to 1.0s) the strength increase is similar to that predicted by Kawashima and Eurocode 8. However, for ranges 2 and 3 the average strength increase is close to 7/8 and 2/3 of that in range 1 respectively. With a ductility of 2 or more the effect of a reduction in viscous damping is smaller. In this case the strength increase is typically 35% of that given by the Eurocode 8 and Kawashima equations.

Table 3: Influence of viscous damping on ratio of strength required to 5%damping

ξ%	α%	μ	Range	Bi-linear	Beam	Column	Masonry	Av. All	Std. Equ.
0.5	2.5	1	1	1.72	1.72	1.72	1.72	1.72	
	2.5		2	1.60	1.60	1.60	1.60	1.60	
	2.5		3	1.46	1.46	1.46	1.46	1.46	
0.5	2.5	2	1	1.22	1.26	1.27	1.32	1.27	Kawashima
	2.5		2	1.23	1.24	1.25	1.31	1.26	1.75
	2.5		3	1.28	1.18	1.19	1.23	1.22	
0.5	2.5	4	1	1.12	1.18	1.23	1.27	1.20	EC8
	2.5		2	1.18	1.20	1.19	1.25	1.20	1.67
	2.5		3	1.21	1.17	1.16	1.25	1.20	
0.5	2.5	6	1	1.14	1.19	1.18	1.28	1.20	
	2.5		2	1.24	1.27	1.21	1.29	1.25	
	2.5		3	1.22	1.21	1.23	1.36	1.26	
2	2.5	1	1	1.31	1.31	1.31	1.31	1.31	
	2.5		2	1.28	1.28	1.28	1.28	1.28	
	2.5		3	1.21	1.21	1.21	1.21	1.21	
2	2.5	2	1	1.14	1.15	1.16	1.19	1.16	Kawashima
	2.5		2	1.11	1.14	1.14	1.18	1.14	1.33
	2.5		3	1.13	1.10	1.10	1.12	1.11	
2	2.5	4	1	1.08	1.12	1.15	1.17	1.13	EC8
	2.5		2	1.07	1.10	1.11	1.15	1.11	1.32
	2.5		3	1.09	1.03	1.05	1.15	1.08	
2	2.5	6	1	1.09	1.13	1.13	1.16	1.13	
	2.5		2	1.09	1.13	1.10	1.19	1.11	
	2.5		3	1.09	1.07	1.05	1.26	1.12	

5.3 Influence of strain-hardening ratio

This set of analyses was similar to those in the previous section except the strain hardening ratio, α, was first decreased from 2.5% to 0% and then increased to 5%. The viscous damping level was maintained at 5 percent. In this case only the bilinear and masonry models were analysed. The individual strengths with each earthquake record and ductility level were divided by the corresponding value found for the structure with 2.5% strain hardening. The results of these analyses are given in Table 4.

Table 4: Influence of strain hardening level on strength ratio to 2.5% strain hardening

α	μ	Bilinear Model Period range				Masonry model Period range			
		0.3 – 1s	1.1 – 3s	3.1 – 5s	Av.	0.3 – 1s	1.1 – 3s	3.1 – 5s	Av.
0	2	1.01	1.03	1.02	1.02	1.00	1.00	1.00	1.00
	4	1.09	1.09	1.10	1.09	1.01	1.00	1.01	1.01
	6	1.13	1.16	1.17	1.15	1.01	1.02	1.02	1.02
5	2	0.98	0.98	0.99	0.98	1.00	1.00	1.00	1.00
	4	0.95	0.95	0.94	0.95	0.99	1.00	1.02	1.00
	6	0.94	0.93	0.93	0.93	0.99	0.99	1.04	1.01

From Table 4 it can be seen that changing the strain-hardening ratio percentage has no appreciable influence on the masonry model. However, there is a significant influence with the bilinear model when the ductility level exceeds 2. For ductility levels of 4 and 6 reducing the strain-hardening percentage to zero increases the required strength by an average of 9 and 15 percent respectively. The corresponding change when the strain hardening ratio was increased to 5 percent was to give an average decrease in strength of 5 and 7 percent respectively.

6.0 CONCLUSIONS

1 Approximately 36 000 single degree of freedom analyses have been made using a range of earthquake ground motions to examine the influence of hysteretic form, viscous damping level and strain hardening characteristics, on the strength required for elastic and ductile structures. Structures which have load displacement responses, which can dissipate very little energy hysterically (ie prestressed concrete with unbonded tendons), were excluded from the investigation.

2 It is shown that with hysteretic form has only a minor influence on the strength required for a given level of ductility. This finding agrees with previous smaller studies [Anaganostopoulas and Roesset, 1974, Otani, 1980]

3 Reducing the viscous damping level below 5 percent is shown to have a significant influence on elastic response but an appreciably smaller effect on the strength required for structures with a ductility of 2 or more.

4 Changing the strain-hardening ratio from 2.5 percent to zero, or 5 percent, is shown to have only a minor influence on the required strength with the masonry model. However, with the bilinear model the significance of these changes increases with ductility. For a ductility of 6 reducing the strain-hardening percentage to zero increases the required strength by an average of 15 percent. The corresponding decrease when the strain hardening ratio is increased to 5 percent is 7 percent.

ACKNOWLEDGEMENTS

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DAMPING FOR THE NONLINEAR STATIC PROCEDURE IN ATC-40

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ABSTRACT

The *Nonlinear Static Procedure* in ATC 40 is based on the capacity spectrum method. With this approach the deflection of a yielding structure resulting from seismic excitation is assumed to be equal to that of a viscously damped linear model with the secant stiffness of the yielding structure. The required level of damping is based on the "equivalent" viscous damping theory of "Jacobsen", in which the damping level is determined by equating the energy dissipated viscously in the linear model in a cycle to that dissipated by the yielding structure when subjected to the maximum displacement. As it was found that equivalent viscous damping is not totally appropriate for all cases, the ATC 40 procedure multiplies the equivalent viscous damping value by an empirical factor, κ, which varies with ductility level and hysteretic form.

An alternative approach of using substitute viscous damping instead of an empirically modified equivalent damping is assessed. To find substitute viscous damping values the total earthquake input energy into the yielding structure is equated to the energy dissipated viscously by the linear model. The use of substitute viscous damping is shown to improve the accuracy of prediction and to give a method of assessing the influence of different *hysteretic* behaviors.

INTRODUCTION

Nonlinear Static Procedures (NSP) have been promoted by ATC-40 as suitable for determining the seismic performance of existing concrete buildings. In particular the capacity spectrum method, developed by Freeman et al [1975] has been recommended. This approach uses an iterative procedure to match the strength and displacement capacity of the structure to the seismic demand. It requires the engineer to check for the satisfactory performance of the structure and its components at a balance or "performance" point, which is the condition where the design capacity (strength and displacement) matches the seismic demand.

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The steps in this procedure are as outlined below.

1. From a pushover analysis of the structure a plot of base shear versus roof deflection is generated. This is then scaled to be equivalent to a single degree of freedom capacity plot by dividing the base shear by the effective first mode mass M_1, and the roof deflection by Γ_1, ϕ'_N, where

$$M_1 = \frac{(\sum_{j=1}^{N} m_j \phi^1{}_j)^2}{\sum_{j=1}^{N} m_j \phi^1{}_j{}^2}$$

$$\Gamma_1 = \frac{\sum_{j=1}^{N} m_j \phi^1{}_j}{\sum_{j=1}^{N} m_j \phi^1{}_j{}^2}$$

ϕ'_N is the top floor component of the first mode.
The resultant relationship is referred to as the capacity spectrum. At different ductility displacements along the capacity spectrum effective damping values, β_{eff}, are calculated to represent the energy dissipated by the structure.

2. The traditional design spectrum with 5% damping, in which the elastic acceleration is plotted against period, is converted to a spectrum where the acceleration is plotted against the displacement. This Acceleration-Displacement-Response-Spectrum (ADRS) is a demand spectrum with 5% damping.

3. A number of demand spectra may be developed for different levels of damping. These spectra relate to the demand on a linear oscillator, labeled as associated elastic model (AEM). The different damping levels represent the effective damping in yielding structures with different extents of nonlinear behavior. The demand spectrum for a given level of damping is found by multiplying the 5% demand spectrum by spectral reduction factors, "SR_A" and "SR_V".

4. The capacity and demand spectra are plotted together as shown in Fig. 1. The "performance" point for the structure is defined as the intersection of the capacity and demand spectra. It is located at the point, (d_p, a_p), where the effective damping of the structure, β_{eff}, equals the damping of the intersecting demand spectrum.

To implement this procedure, a relationship is required to equate the seismic behavior of the nonlinear system to that of an associated elastic model. The ATC-40 document [1996] achieves this by implementing the concept of "equivalent" viscous damping. However, to ensure satisfactory performance of the concept an empirical damping modification factor, κ, has been

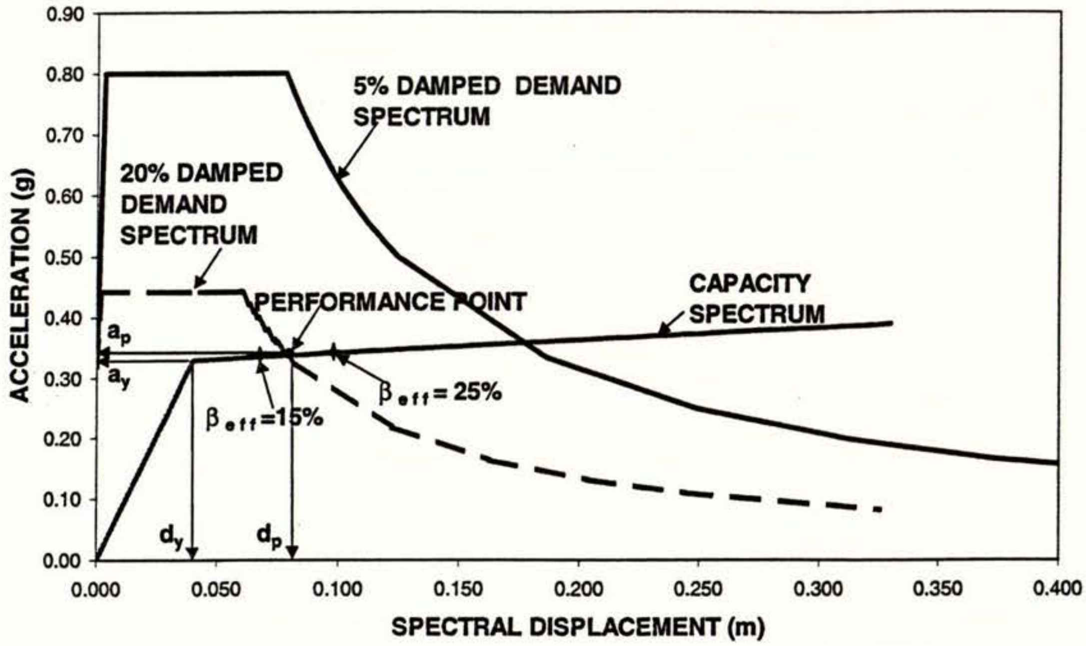

Figure 1. Capacity and demand spectra and performance point.

introduced. This factor reduces the magnitude of the equivalent damping to allow for stiffness degradation that may occur in the structure. In addition it also limits the magnitude of damping that can be used for structures that have "full" hysteretic curves, that is type "A" structures (near bi-linear response). These limits have been imposed, *judgmentally*, as the document acknowledges that the "equivalent" damping concept "overestimates the realistic levels of damping" without explicit reference to the background of these limits.

EQUIVALENT VISCOUS DAMPING

With the ATC-40 method, "effective damping" for a structure is based on "equivalent viscous damping". Jacobsen [1930] proposed the theory of equivalent viscous damping as a tool in the study of "steady forced vibration of damped single degree of freedom systems under the influence of a sinusoidal varying disturbing force". He proposed that alternative oscillators (analogous to associated elastic models) could have viscous damping as an attenuating force to replace systems with complex damping mechanisms. In the development of the theory the original system with complex damping (which might include in-elastic hysteretic response) and the elastically responding viscously damped systems, were both in a steady state excitation sustaining the same level of deformation. Thus Jacobsen's equivalent damping was concerned with equating the energies of the two oscillators in a specific steady state cycle, that is, equating the area under the hysteretic curve at maximum deformation to the area under the damping force versus displacement curve for a linear viscously damped system. This equivalence in areas leads to the development of Eq. 8-6 in the ATC-40 document, as reproduced below-

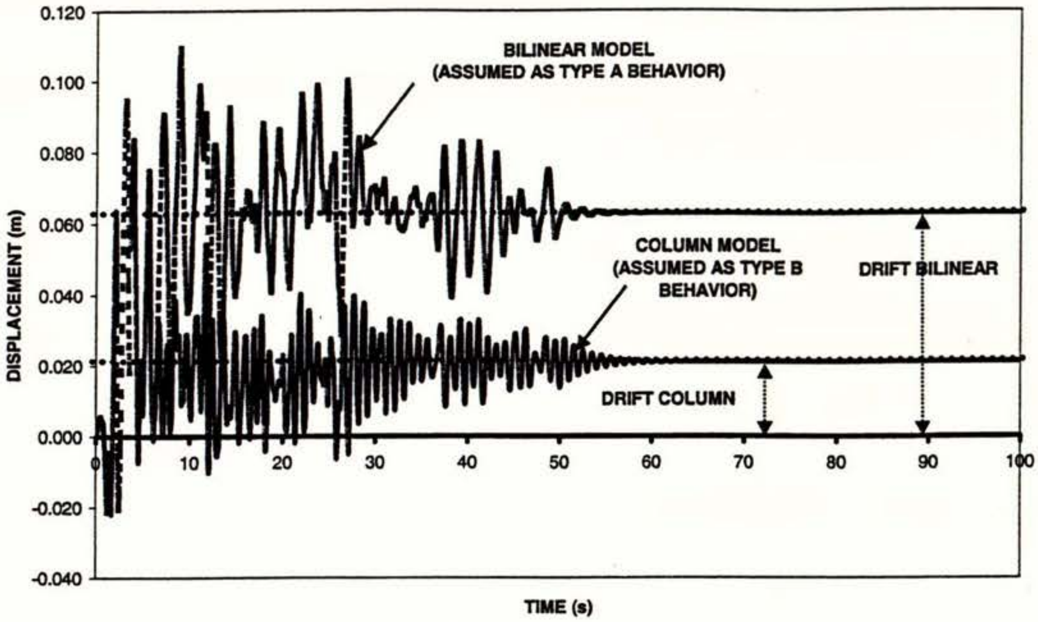

Figure 2. Displacement time history for single degree of freedom structures subjected to El Centro (1940) NS ground motion.

$$\beta_o = \frac{63.7\,(a_y\, d_{pi} - d_y\, a_{pi})}{(a_{pi}\, d_{pi})} \qquad \text{(ATC40- Eq. 8-6)}$$

It has been attractive for researchers in the past to use this simple relationship. It appears reasonable and provides a semblance of a theoretical basis for the development of obtaining the effective damping. However, this approach overlooks the fact that the response of a nonlinear system to a transient seismic loading cannot be described as "steady state", which oscillates around a zero mean displacement from a maximum negative to a maximum positive value and back. In general, the nonlinear seismic response of a structure tends to "drift" and at the time of maximum displacement it is most likely oscillating about a non-zero mean position. Consequently, as pointed out by Jennings [1968] equivalent damping overestimates the effective damping. Fig. 2 illustrates this point by showing the time history response of two oscillators. In both cases the oscillators sustain inelastic deformation early in the earthquake and then they start to oscillate about an offset position.

EFFECTIVE DAMPING IN ATC 40 METHOD

With the ATC 40 method effective damping values have to be found. The value of displacement at a performance point is assessed and from this a ductility value can be found. This enables a value of equivalent damping, β_o, to be determined from Eq. 8-6 in ATC 40. The effective damping value, β_{eff}, is then found from Eq. 8-8 in ATC 40, which is reproduced below. The process is repeated until the effective damping value at the selected displacement on the capacity spectrum is equal to the damping associated with an intersecting demand spectrum.

The effective damping is given by

$$\beta_{eff} = \kappa \beta_o + 5 \qquad \text{(ATC40- Eq. 8 - 8)}$$

where the 5 represents the viscous damping level in the basic elastic acceleration response spectrum and κ is an empirical, or judgment factor, as described in a previous section.

SUBSTITUTE DAMPING

Gulkan and Sozen [1974] conducted a series of dynamic tests, which involved in-elastic displacements on one story, single bay, reinforced concrete frames. These were detailed to behave in a ductile manner with plastic hinges forming in the columns. With this form of plastic hinge, the Takeda hysteretic rules provide a good description of the force deflection characteristics of the test units.

From their test results and analytical studies Gulkan and Sozen proposed that the response of a viscously damped elastic system during the passage of an earthquake would be similar to that of a ductile structure, if its viscous damping was determined as equal to "substitute viscous" damping. The substitute viscous damping coefficient was defined as the value required so that the energy, viscously dissipated by a linear structure, is equal to the earthquake input energy to the nonlinearly responding structure. Thus,

$$c \int_0^t \dot{v}^2(\tau)\, d\tau = -m \int_0^t \ddot{v}_g(\tau)\, \dot{v}(\tau)\, d\tau \tag{1}$$

where, c is the substitute viscous damping constant, v is the displacement of the structure (the dot refers to the differentiation with respect to time), $\ddot{v}_g$ is the ground acceleration, t is the duration of the excitation and τ is a time variable. Gulkan [1974] assumed that the relative velocity of the associated elastic structure is the same as the ductile structure. With this assumption Eq. 1 can be rearranged to give,

$$\beta_{Substitute} = \frac{T_{Substitute} \int_0^t \ddot{v}_g(\tau)\, \dot{v}(\tau)\, d\tau}{4\pi \int_0^t \dot{v}^2(\tau)\, d\tau} \tag{2}$$

where $\beta_{Substitute}$ and $T_{Substitute}$ are respectively the substitute viscous damping coefficient and substitute period of the associated model. Gulkan and Sozen [1974] assumed that the substitute frequency $\omega^2_{Substitute}$ could be taken as the ratio of the measured maximum absolute acceleration to measured absolute maximum displacement. Thus the associated elastic model was given the secant stiffness.

IMPLEMENTATION OF SUBSTITUTE VISCOUS DAMPING IN ATCT 40

The influence of various factors on substitute damping values can be found by analyzing the motions of single degree of freedom structures with different hysteretic models and a range of earthquake ground records. In this way relationships can be derived to relate the influence of ductility, hysteretic form and period on substitute damping.

For this paper the substitute damping values were calculated from four normalized earthquakes whose 5% damped spectra closely resemble the standard ATC-40 spectrum with C_a, T_a and T_v equal to 0.33g, 0.125 seconds and 0.625 seconds respectively. Eqs. 4 and 5, which are given below, were derived from the results of these analyses. It has been shown by Judi et al [2001] that the earthquakes chosen for this calculation do not substantially influence the substitute damping values. As seen in Fig. 3 and Eqs. 4 and 5 the substitute-damping coefficient depends upon the ductility demand, the hysteretic form and to a lesser extent upon and the elastic period of the structure.

The implementation of substitute damping into the ATC-40 NSP is straightforward. At an estimated performance point displacement the ductility is calculated as

$$\mu_i = {}^{d_{pi}}\!/_{d_y} \qquad (3)$$

The substitute viscous damping associated with that level of ductility is then found from equations of the form of Eqs. 4 and 5, which are given below. The performance point for the structure is that point on the capacity spectrum where the damping value, $\beta_{Substitute}$, is equal to the damping value in intersecting demand curve. For "type A" structures, which behave with a near bi-linear response, the substitute damping takes the form,

$$\beta_{Substitute} = [10.5\,(\mu_i - 1)^{0.8} + 1.4\,T - 0.6\,] > 5 \qquad For\ 0.4 \leq T \leq 3.0s \qquad (4a)$$

$$\beta_{Substitute} = [10.5\,(\mu_i - 1)^{0.8} + 3.6\,] > 5 \qquad For\ 3.0s \leq T \qquad (4b)$$

but with an upper limit for $\beta_{Substitute}$ of 60% over the complete period range.

Ductile reinforced members that have been designed and constructed to comply with modern seismic design codes, when subjected to reversing in-elastic deformation undergo some stiffness degradation with cyclic loading. For this case the effective damping values are given by

$$\beta_{Substitute} = [17.5\,(\mu_i - 1)^{0.39}\,] > 5 \qquad For\ 0.4s \leq T \qquad (5)$$

These values were found using a hysteretic model derived from tests on columns in which the shear and confinement reinforcement met the requirements of modern codes of practice. It has

Table 1: Performance Points for "Type A" Structures

Struct. No.	Period (s)	Yield acc. (g) and displ. (m)		Effective Damping values				Substitute Damping values			
		a_y	d_y	d_p	a_p	β	μ	d_p	a_p	β	μ
1	0.5	0.449	0.028	0.041	0.454	19.9	1.481	0.066	0.465	13.6	2.37
2	0.5	0.168	0.010	0.072	0.193	46.3	6.93	0.066	0.191	40.01	6.35
3	1.0	0.250	0.062	0.091	0.253	24.0	1.47	0.139	0.258	13.2	2.24
4	1.0	0.083	0.021	0.151	0.096	40.2	7.33	0.139	0.095	43.5	6.75
5	2.0	0.125	0.124	0.181	0.126	24.1	1.46	0.268	0.127	14.0	2.16
6	2.0	0.063	0.062	0.226	0.067	38.5	3.64	0.276	0.068	30.5	4.44
7	3.0	0.083	0.186	0.276	0.084	24.0	1.49	0.386	0.085	14.8	2.08
8	3.0	0.042	0.094	0.341	0.045	38.2	3.63	0.406	0.045	31.1	4.32

Table 2: Performance Points for "Type B" Structures

Struct. No.	Period (s)	Yield acc. (g) and displ. (m)		Effective Damping values				Substitute Damping values			
		a_y	d_y	d_p	a_p	β	μ	d_p	a_p	β	μ
1	0.5	0.449	0.028	0.043	0.455	19.1	1.53	0.064	0.464	17.4	2.28
2	0.5	0.168	0.010	0.097	0.203	29.0	9.27	0.076	0.195	35.2	7.30
3	1.0	0.250	0.062	0.101	0.254	20.8	1.626	0.125	0.256	15.3	2.01
4	1.0	0.083	0.021	0.196	0.101	29.0	9.515	0.163	0.097	36.6	7.91
5	2.0	0.125	0.124	0.201	0.126	20.9	1.618	0.251	0.127	15.4	2.02
6	2.0	0.063	0.062	0.291	0.068	28.8	4.686	0.296	0.068	28.4	4.77
7	3.0	0.083	0.186	0.306	0.084	21.1	1.649	0.376	0.085	15.4	2.02
8	3.0	0.042	0.094	0.436	0.046	28.8	4.643	0.441	0.459	28.2	4.70

been assumed in this paper that these expressions represent the behavior of "type B" structures as described in ATC 40, though the correspondence is not exact.

It can be seen from Table 1 for the "type A" structures, that the ATC approach consistently predicts behavior described by higher effective damping and consequently smaller displacements compared to values determined by the substitute damping approach. For "type B" structures, as listed in Table 2, the damping, and hence predicted performance is similar for both approaches when the ductility demand is higher. For cases where the ductility demand is approximately two, the displacements predicted by using effective damping are consistently lower than those using substitute damping.

To determine which of the two damping approaches provides the more accurate estimate of the structural performance, nonlinear time history analyses were performed on the sixteen structures described in Tables 1 and 2. Each of these structures were subjected to ground motions of seven earthquakes, which are listed in Table 3. They were selected from those listed in Table 4-9 of ATC-40. Each earthquake was scaled for each structure. The scale factor was chosen so that the spectral ordinate of the 5% earthquake spectrum associated with the elastic period of the structure equaled that the ATC demand spectrum for 5% damping. The scale factors that were used are listed in Table 3.

Table 3: Scale factors used for ground motions in nonlinear analyses

Period (s)	Northridge Century City Ch1	Northridge Moorpark Ch3	Northridge Moorpark Ch1	Landers Yermo Ch1	Landers Joshua Tree Ch3	Loma Prieta Gilroy #2 Ch1	Loma Prieta Hollister 2 Ch3
0.5	1.668	2.502	1.891	1.812	1.196	1.086	0.681
1.0	1.916	2.211	2.156	1.536	1.248	1.236	0.500
2.0	1.599	2.907	2.499	2.246	1.890	0.994	0.659
3.0	2.268	3.883	4.219	2.428	3.590	2.029	0.941

The results of these analyses are summarized in Table 4 and presented in Figs. 4 and 5. In these figures the computed displacements are plotted against predicted displacements. It can be seen that there is a considerable scatter in the results. However as ATC 40 (section 4.5.1) allows the "mean" response of seven or more nonlinear analyses to be used as the representative response. The linear trend line of these averages can be used to help with a comparison study. The "dashed" and "dotted" lines are trend lines for the "effective" and "substitute" damping results respectively. From Table 4 and the trends shown in Fig. 5 for "type A" structures, it is clear that the substitute damping approach provides a more consistent estimate for the displacements than

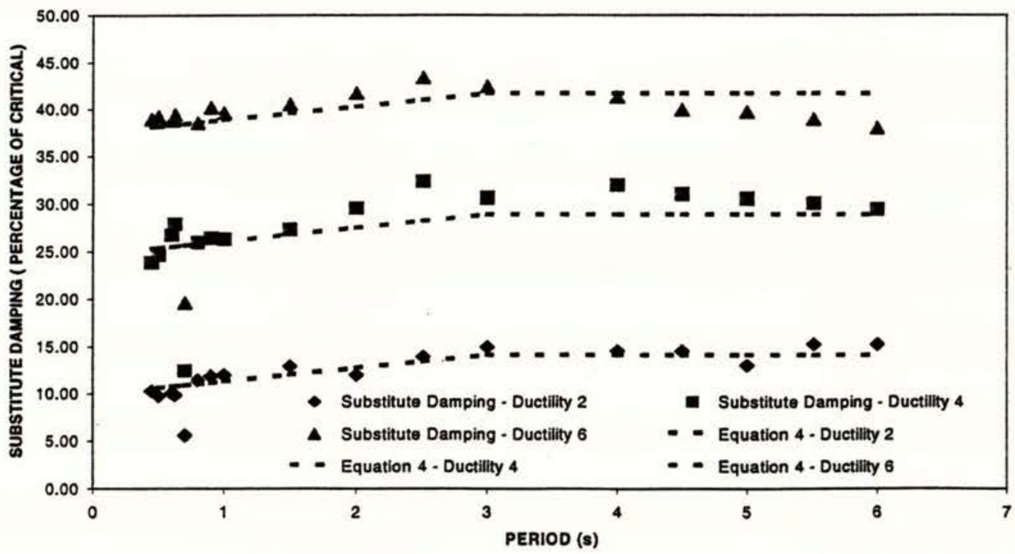

Figure 3. Substitute damping values for bi-linear (type A) response.

the ATC 40 modified effective damping method. For the "type B" structures the substitute damping approach appears to provide marginally more consistent predictions.

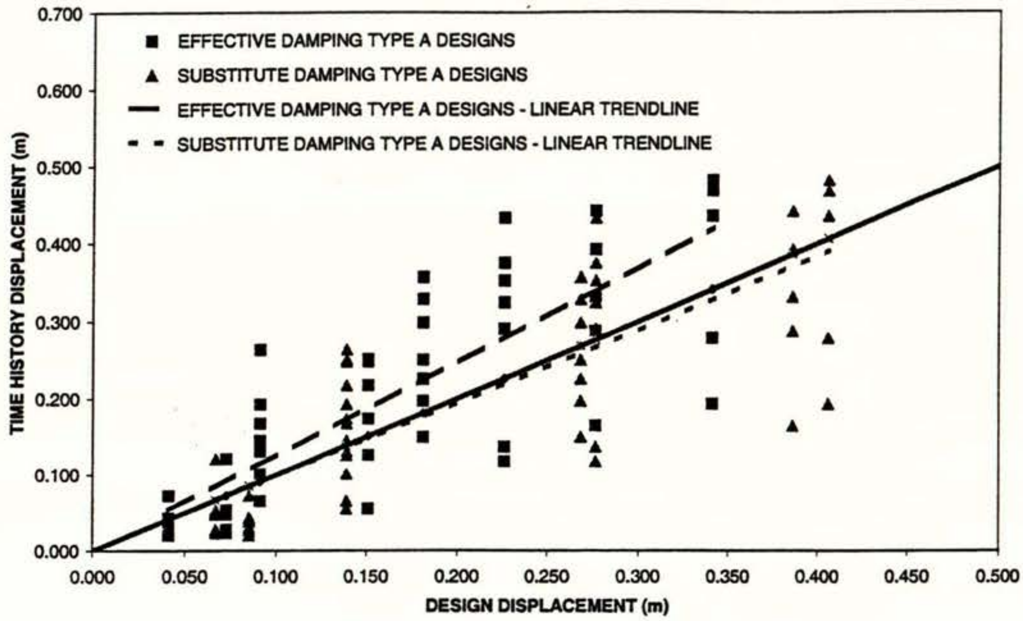

Figure 4. Time history results for Type A behavior structures.

Table 4: Summary of results of time history analyses

(a) Bilinear hysteretic model results

Period (s)	Effective Damping values		Substitute Damping values	
	Av. TH / Predicted*	Std. dev[ϕ]	Av. TH / Predicted	Std. dev
0.5	0.81	0.44	0.86	0.44
1.0	1.46	0.63	1.22	0.40
2.0	1.35	0.46	1.01	0.35
3.0	1.19	0.32	0.93	0.26

*Average of time history displacements divided by predicted displacement

[ϕ] Standard deviation of results for each period

(b) Column model (degrading stiffness hysteretic model)

Period (s)	Effective Damping values		Substitute Damping values	
	Av. TH / design	Std. dev	Av. TH / design	Std. dev
0.5	0.70	0.25	0.91	0.27
1.0	0.70	0.43	0.95	0.39
2.0	0.99	0.34	0.82	0.27
3.0	0.94	0.35	0.83	0.31

DISCUSSION AND CONCLUSIONS

The results summarized in Table 4 and illustrated in Figs. 4 and 5 show the trends that have been observed in many analyses of simple nonlinear systems [Judi 2000]. That is, for seismic analysis, the substitute damping formulation provides a more consistent approach for determining the damping for an associate elastic model than the equivalent damping concept. This is particularly the case when the associated elastic model is required to match the behavior

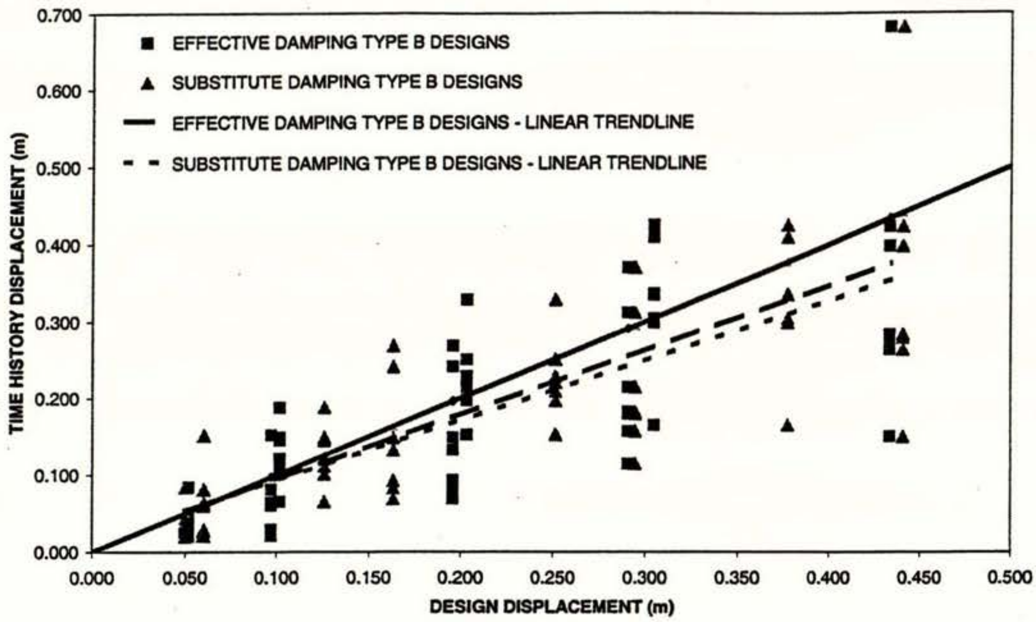

Figure 5. Time history results for Type B behavior structures.

of a bilinear "type A" structure. If equivalent viscous damping is to be used, as noted in the ATC40 procedure, it must be modified empirically. The authors propose that the substitute damping approach should be used in the Nonlinear Static Procedure of ATC40 as simple formulae are easily developed for different hysteretic forms. The time history analyses examined in this paper indicate that the use of substitute damping improves the accuracy and reduces the scatter in the predicted displacement.

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Displacement Focused Seismic Design Methods - A Comparative Study

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ABSTRACT: Two methods of seismic design, namely Direct Displacement Based Design and Displacement Focused Force Based Design, are reviewed together with the Capacity Spectrum Method for the analysis for existing structures. Modifications to these three approaches are proposed to enable them to more accurately predict the influence of different hysteretic behaviour on response. To assess the relative accuracy of these methods a range of single degree of freedom structures were proportioned by each of the methods for different hysteretic behaviour. The responses of these proportioned structures to four different earthquake records were then determined. A comparison of the maximum displacement recorded in each time history analysis, shows that there is little to pick between the three design approaches in terms of accuracy.

1 INTRODUCTION

During the last decade the concept of "performance based design" has received considerable attention. The objective is to ensure that under design level earthquakes predetermined damage levels are not exceeded. As damage is principally a function of material strain levels, and these can be related to displacements, performance based design methods generally focus on displacements.

The currently widely accepted method of seismic design, namely force based design, has been criticised for having displacements checked only at the end of the design process with no apparent focus on these values. With the displacement focused method a structure can be proportioned (tuned) to sustain a predetermined displacement level under design earthquake ground motion.

A number of different displacement focused design methods have been proposed. In this paper three of these are considered, namely-

- Direct displacement based design, which was proposed by Priestley and his co-workers [Kowalsky et al. 1994]
- Capacity spectrum method, one of the non-linear static procedures described in [ATC40]. This is based on a method initially proposed by Freeman et al. [1975].
- Displacement focused force based design, which is a modification of forced based design [Judi et al. 2001a].

These methods are described and several series of single degree of freedom structures are designed using methods based on these approaches. The maximum displacements obtained in time history analyses with a number of different earthquake records are compared with the design values. While this paper deals only with single degree of freedom structures, techniques have been proposed which enable the approaches to be extended to multi-degree of freedom structures [ATC 40 1996, Loeding et al. 1998].

2 SEISMIC DESIGN METHODS

2.1 *General*

In a major earthquake, structures that have been designed to behave in a ductile manner, referred here as "design structures", respond non-linearly due to yielding, cracking and crushing of material, and different forms of damping. However, in all the seismic design methods the design actions are assessed from elastically responding models, of which there are two forms. In this paper the term "associated elastic model" is given to the form used in the direct displacement based and capacity spectrum design methods, while the term "analytical elastic model" describes the form used in the force based and displacement focused force based methods.

With the "associated elastic model" the elastic member properties are based on the secant stiffness between the origin and the design maximum displacement, see Figure 1. To allow for the energy dissipated in an earthquake, the model is given an increased level of viscous damping. However, with the "analytical elastic model" the member properties are based on the stiffness values appropriate for load cycles sustained prior to yield or significant non-linear behaviour of material. For reinforced concrete an equivalent elastic stiffness is chosen to allow for non-linearity due to tension stiffening and axial load effects, as illustrated in Figure 1. The viscous damping associated with this model represents the energy dissipation of the design structure in its pre-yielding load cycles and the nonlinear behaviour of the structure is described by the level of "ductility" sustained.

In all the design methods it is assumed that the maximum acceptable design displacement, Δ_d, is determined on the basis of either, material strains, as these give a measure of structural damage, or acceptable inter-storey drift limits, as these give a measure of non-structural damage. As currently presented the influence of P-delta effects on the deformation of structures is ignored in the direct displacement based design and the capacity spectrum methods. For the sake of comparison of the three methods in this paper it is also neglected in the displacement focused force based design approach. However, it should be noted that P-delta effects can have a major influence on structures where low lateral strengths are permitted for ductile structures, as is the case for structures designed to meet the minimum requirements of the New Zealand Loadings Standard [NZS4203-1992].

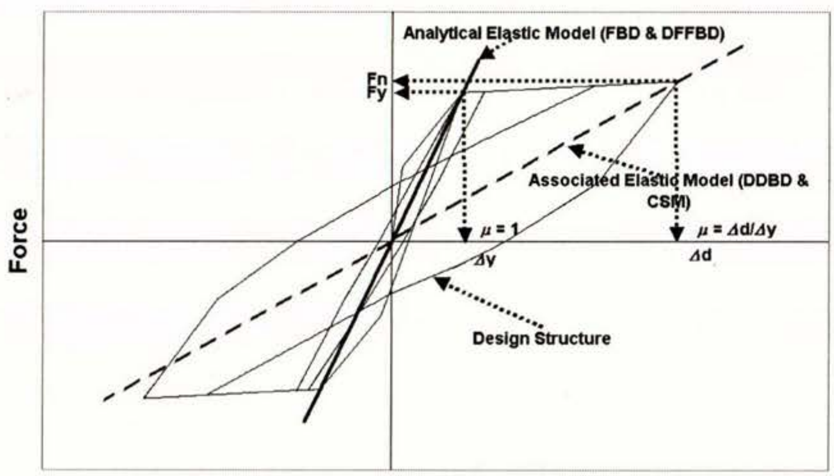

Figure 1 Relationship between load – deflection response of design structure and elastic models used in analyses

2.2 *Direct Displacement Based Design*

The direct displacement based design method was set up for the design of new structures. The following steps are involved with this method as originally proposed.

1. From the design displacement, Δ_d, and an assessment of the ductility one displacement, Δ_y, a first estimate is made of the displacement ductility, μ, which is equal to Δ_d/Δ_y.
2. The viscous damping level of the associated elastic model is determined from the hysteretic form and ductility, using relationships such as those given in [Priestley et al. 2000]. The values vary with ductility and they are calculated using the "equivalent damping" concept. With this the viscous damping levels are found by equating the energy dissipated by the associated elastic model to that dissipated by the design structure when they are both subjected to a load cycle with peak displacements of ± the design displacement, Δ_d.
3. From the design displacement, Δ_d, and damping level, ξ, the period of the associated elastic model can be read off from displacement versus period response spectra as shown in Figure 2. In this figure the elastic displacement spectra are overlaid on the elastic acceleration spectra in one plot.
4. Using the period from the previous step the base-shear can be assessed from acceleration versus period response spectra for different damping levels. With this value and knowledge of the strain-hardening characteristics, the required yield strength of the design structure can be assessed, as illustrated in Figure 2.
5. From the required strength and details of the members the initial stiffness can be assessed, and hence the ductility one displacement, Δ_y, can be found.
6. The ductility one displacement found in step 5 is compared with the assessed value in step 1. If there is a significant discrepancy between these two values a new estimate of Δ_y is made and steps 1 to 6 are repeated until convergence is obtained.

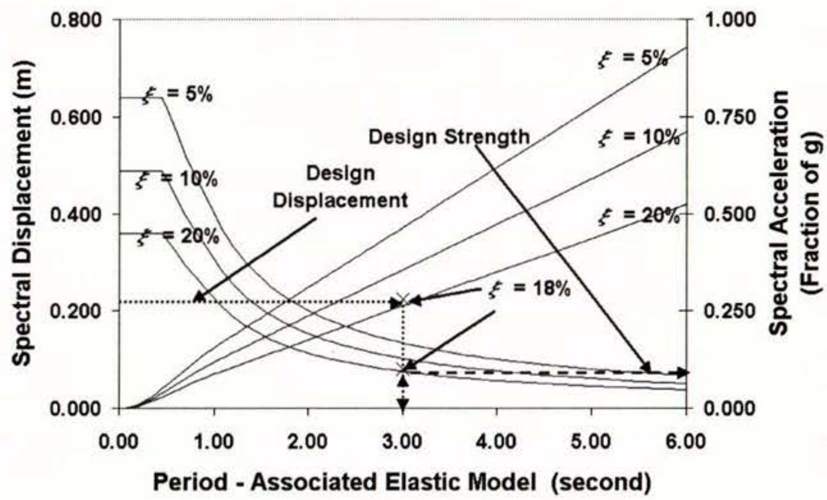

Figure 2 Direct Displacement Based Design

2.3 *Capacity Spectrum Non-linear Static Procedure*

The capacity spectrum non-linear static procedure method described in ATC40, which as outlined below, is intended for the analysis of existing structures. As the details of the structure are known its initial stiffness can be found. The process is illustrated in Figure 3. The following steps are required.

1. Spectral acceleration versus spectral displacement response spectra, for varying levels of viscous damping, are developed for associated elastic models.
2. A model of the design structure is subjected to a push over analysis to give a lateral force divided by mass versus displacement trace and this is superimposed on the response spectra.
3. From the analytical lateral force divided by mass versus displacement trace for the structure

the ductility one displacement, Δ_y, can be determined. This is used to define the displacement ductility values at different positions along the displacement trace.

4. Positions along this trace are assigned effective damping values. These depend on the ductility at the point being considered and the characteristics of the hysteretic response.
5. The predicted maximum displacement corresponds to a balance point where the effective damping value on the lateral force versus displacement trace intersects the design response spectrum for the associated elastic model with the same damping value.

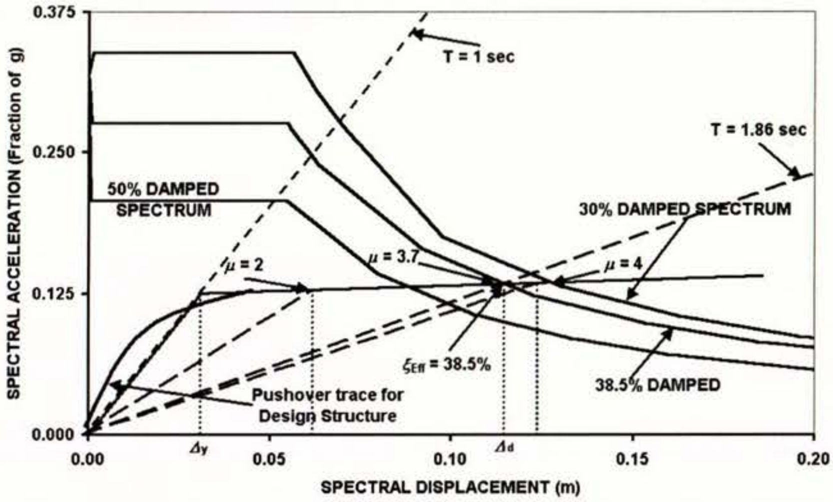

Figure 3 Capacity Spectrum Nonlinear Static Procedure

The effective damping values for the points on the lateral force displacement curve are calculated for different hysteretic relationships by taking the equivalent damping value found assuming a bi-linear response and multiplying by a factor, κ. This factor has a maximum value of 1 and smaller values are used where the hysteretic forms show significant stiffness degradation, as less energy is dissipated. The κ factor acts as a calibration factor to improve the accuracy of the method.

As shown in the next section the direct displacement based design method and the capacity spectrum non-linear static procedure are in essence different versions of the same concepts.

2.4 *Modification of Direct Displacement Based Design and Capacity Spectrum Methods*

The direct displacement based design method can be modified to eliminate the need to determine the period of the associated elastic model. From Figure 3 it can be seen that the period of an associated elastic model is only required as a link between the displacement and acceleration response spectra. This link is unnecessary if spectral displacement versus spectral acceleration response relationships were drawn directly. With this modification it can be seen that the capacity spectrum and direct displacement based design methods are identical in concept. There are a number of minor differences between the two approaches as outlined below.

- Different rules are used to develop the spectral acceleration versus spectral displacement relationships with different levels of viscous damping. However, the corresponding values are in close agreement.
- With the Capacity Spectrum method the calibration factor, κ, is applied to the equivalent damping coefficients found for bi-linear hysteretic response. With direct displacement based design the equivalent damping values are taken from a series of relationships determined for different hysteretic forms.

In a detailed study of the direct displacement based design method it was found that equivalent damping overestimates the damping values of the associated elastic models when the hysteretic forms exhibited limited stiffness degradation. This leads to an under-estimate of the required strength for this range of structures. Replacing "equivalent" damping by "substitute" damping, which is described in the next paragraph, was found to eliminate the need for the calibration factor, κ, in the capacity spectrum method. This change also led to improved accuracy of prediction of the direct displacement design method [Judi et al. 2000, 2001b] over a wide range of hysteretic forms.

Substitute damping is defined as the level of viscous damping of an associated elastic model, which causes it to dissipate the same energy as the design structure when subjected to the full earthquake ground motion. In theory substitute damping values depend on the ground motion. However, analyses of a wide range of structures with different hysteretic responses to different ground motions have shown that the influence of the earthquake record is small. For practical purposes substitute-damping values can be determined on the basis of the ductility level, fundamental period of structure and hysteretic form. Expressions have been developed for these values [Judi et al. 2002b].

2.5 *Displacement Focused Force Based Design*

Force based design is widely used for seismic resistant structures and as such it is embedded in many seismic codes of practice. With a few modifications it can be changed into a displacement focused force-based design. A major advantage of this method is its similarity to existing practice.

The following steps are involved in displacement focused force based design.

1. An analytical elastic model of the structure is developed using estimated member sizes and details. From this model the fundamental period, T, is found.
2. From the design displacement versus period response spectrum the period T_d, which corresponds to the design displacement, Δ_d, is read off. If T is equal to or less than T_d then the displacement of the structure will be less than the design value. If it is larger than this value the members need to be stiffened. If T is appreciably less than T_d then a more efficient structure may be obtained by reducing the member sizes so that the period more closely approaches the critical value, T_d.
3. With the period T calculated, the required strength can be selected from the set of design (acceleration) spectra for the different ductility levels.

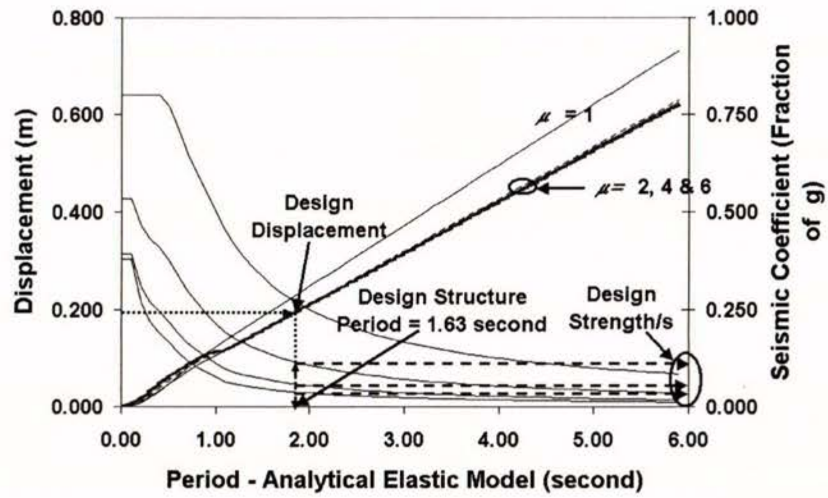

Figure 4 Displacement Focused Force Based Design

4. The process described above is illustrated in Figure 4. In this figure, the construction of the spectra is based upon the NZS4203:92 intermediate soil ductility one basic seismic hazard acceleration coefficient. The ductility 2, 4 and 6 acceleration spectra have been developed using the scale factors determined from the analysis of 16 earthquakes as described by [Judi et al. 2002a]. The displacement spectra have been constructed from these using a simple bilinear envelope form. One point should be noted where this process is used with reinforced

concrete. Many codes of practice recommend that second moment of area values are taken as some fraction of the value based on the gross section. These recommendations vary significantly, for example Eurocode 8 [1994] recommends that the gross section properties are used for beams while in the NZ Structural Concrete Standard the corresponding recommendation for rectangular beams is to take 40% of the gross value. Neither recommendation recognises the influence of reinforcement content on stiffness. To allow for this factor the stiffness values used in step 1 should be based on assessed reinforcement proportions and these should be checked against those required for strength in step 3. If necessary the steps 1 to 3 should be repeated until convergence is obtained.

As with direct displacement based design the period of the elastic model can be used to link the displacement period and acceleration period response spectra. Hence one step may be removed from the design process by working directly with acceleration spectra, in a manner which is similar to the capacity spectrum method.

3 DESIGNS AND TIME HISTORY ANALYSES

As noted in section 2 the accuracy of both the "Direct Displacement Based Design" method and the "Capacity Spectrum Method" is improved if the viscous damping values for the associated elastic models are based on substitute damping rather than equivalent damping [Judi et al 2000, 2002]. Relationships that link substitute damping values with ductility, period of structure and the hysteretic form have been developed from analyses with a range of earthquake ground motions [Judi et al 2002].

To compare the three methods of design/analysis, a range of structures were designed using Displacement Focused Force Based Design, and a modified form of Direct Displacement Based Design. The modification involved using substitute damping instead of equivalent damping. In addition a range of structures were analysed using the Capacity Spectrum Method, which also was modified in that substitute damping was used instead of equivalent damping for an elastic- plastic hysteretic response and the calibration factor, κ. The designs/analyses were repeated for three different hysteretic models, namely-

Elastic plastic response,

Column model, which was based on load deflection response observed in tests It exhibits stiffness degradation of both loading and un-loading curves [Fenwick et al. 1994],

Masonry model, which was also based on test results, and which exhibits greater stiffness degradation of the loading curves than the column model. It was developed using the approach proposed by Fenwick et al. [1994].

Table 1 Details of Designs/Analyses of Structures

Method of design	Number of structures – Hysteretic model			Period range (seconds)	Ductility range or values
	Bilinear	Column	Masonry		
Direct displacement based design	90	90	87	0.3 – 4.75	1.25- 6
Capacity spectrum method	87	86	85	0.3 – 4.75	1.25- 6
Displacement focused force based design	54	54	54	0.3 – 4.75	2, 4, & 6

In all cases zero strain hardening was assumed with a base level 5% viscous damping. The required strengths were found from the design spectrum for intermediate soils in the New Zealand Loadings Standard [NZS4203-1992]. Details of the designs/analyses are summarised in Table 1. Each designed/analysed structure was subjected to four earthquake records, which were scaled so that the peak response of a 5% damped elastic oscillator, with a period equal to that of the design structure, was equal to that implied by the design spectrum. The four earthquake records that were used were-

- Loma Prieta Earthquake 1989, Hollister, South St. and Pine Dr., Channel 1-90°
- Landers earthquake, 1992, Joshua Tree Fire Station, Channel 3-0°
- Northridge Earthquake 1994, Moorpark, Channel 1-90°
- Northridge Earthquake 1994, Century City, LACC North, Channel 3-360°.

In all cases the ratio of the peak displacement found in the time history analysis to the design displacement was calculated.

4 RESULTS OF ANALYSES

Each of the design structures noted in Table 1 was subjected to the four earthquakes ground motions, normalised as detailed in section 3. The ratios of the resulting time history displacements to the design displacements were calculated and averaged for the four ground motions. To study the effects of periods and ductility on the results, they were grouped into three period ranges, namely, 0.4 to 1.0s, 1.0 to3.0s and 3.0 to 5.0s. The averages were also regrouped into three ductility, μ, subsets of $\mu<2$, $2<\mu<4$ and $4<\mu<6$. These groups are shown in Tables 2 and 3.

Table 2 Results variation with periods

		Bilinear		Column		Masonry	
		AVG	**CoV**	**AVG**	**CoV**	**AVG**	**CoV**
DDBD	**0.4-1.0**	0.983	0.126	0.988	0.094	1.153	0.106
	1.0-3.0	1.137	0.084	0.957	0.016	1.098	0.055
	3.0-5.0	1.023	0.051	0.886	0.055	0.972	0.062
CSM	**0.4-1.0**	0.800	0.136	0.831	0.090	0.961	0.075
	1.0-3.0	1.070	0.095	0.841	0.013	0.912	0.035
	3.0-5.0	0.985	0.027	0.796	0.053	0.825	0.054
DFFBD	**0.4-1.0**	1.127	0.097	1.065	0.126	1.128	0.154
	1.0-3.0	1.198	0.054	1.073	0.036	1.209	0.071
	3.0-5.0	1.030	0.047	0.964	0.083	1.038	0.101

Several observations can be made of the results of the analyses.

- The three methods are comparable in their effectiveness in predicting structural displacements with the three hysteretic models.
- The time history results showed a lot of scatter, which is in part due to the normalisation method, which was used for the four ground motions.

Table 3 Results variation with ductility

		Bilinear		Column		Masonry	
		Avg	**Cov**	**Avg**	**CoV**	**AVG**	**CoV**
DDBD	$\mu \leq 2$	0.896	0.171	0.967	0.126	1.003	0.146
	$2<\mu \leq 4$	0.956	0.197	0.907	0.117	1.077	0.144
	$4<\mu \leq 6$	1.164	0.173	0.965	0.177	1.194	0.226
CSM	$\mu \leq 2$	0.890	0.166	0.881	0.094	0.991	0.122
	$2<\mu \leq 4$	0.865	0.186	0.768	0.096	0.859	0.123
	$4<\mu \leq 6$	0.975	0.165	0.745	0.132	0.810	0.175
DFFBD	$\mu \leq 2$	1.092	0.066	1.056	0.101	1.159	0.135
	$2<\mu \leq 4$	1.100	0.130	1.023	0.105	1.135	0.164
	$4<\mu \leq 6$	1.136	0.126	1.017	0.155	1.053	0.186

- The results indicate that for the bilinear model, DDBD and CSM methods tend to underestimate the displacements, i.e. conservative design strengths; while the DFFBD methods is more likely to overestimate structural displacements. This trend is reduced for the three methods with the stiffness degrading models showing better convergence to the optimal value of unity in the tables.

5 CONCLUSIONS

It can be concluded from the above that a displacement focus in seismic design is not a feature that is associated with a specific seismic design philosophy but rather a procedural modification that can be implemented in any approach, such as force based design. The three methods investigated showed comparable convergence in results for the range of hysteretic models reviewed. Hence this could be seen as a support for Displacement Focused Forced Based Design as its adoption does not require a major departure from a widely accepted existing design approach.

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