EFFECT OF STIFFNESS DEGRADATION ON SEISMIC RESPONSE OF CONCRETE FRAMES

Jagmohan Humar[I]

SUMMARY

Analytical studies of the seismic response of single storey concrete frames are presented and it is shown that stiffness degradation significantly increases ductility demand in short period systems. Studies of multistorey frames are presented next. It is observed that stiffness degradation again increases ductility demand but the effect is pronounced only in the upper storeys, reflecting the effect of higher modes. It is shown that inelasticity in a frame reduces the seismic bending moments and forces, but if the columns are designed to remain elastic while the girders yield, the reduction in the column moments and axial forces is not as large as that in the girder moments.

INTRODUCTION

A difficult part of seismic response studies of reinforced concrete building structures is the modelling of inelastic material behaviour. Elasto-plastic and bilinear moment curvature relationships, although appropriate for the analysis of steel frames, cannot be applied to reinforced concrete members which show a marked decrease in stiffness with increasing cyclic inelastic deformation (5).

Effect of stiffness degradation on the response of single-storey shear type buildings in which the floor beams are considered infinitely rigid were carried out by Clough (2) in 1968. Chopra and Kan (1) extended the investigation to multi-storey shear frames. In real buildings floor beams are flexible and the inelastic deformations are localized at certain sections of the members. Realistic estimates of the localized ductility demands can be obtained only from more sophisticated models of inelastic material behaviour.

A stiffness degrading model proposed by Takeda et al (5) is used in the present study to investigate the effect of stiffness degradation on the ductility demands in multistorey frames in which the beams are considered flexible. The validity of Takeda's model has been experimentally verified by Takeda and others (5) and also by Otani and Sozen (4). Attempt is made in this study to find a correlation between the elastic and inelastic response. In the analyses reported here, the ground motion used is such that its spectrum matches a selected design response spectrum. This motion is referred to as the design ground motion.

MOMENT ROTATION RELATIONSHIP

The moment-curvature curve shown in Fig.1 is used in deriving the moment-rotation relationship for a concrete member. Theoretically, for a given distribution of moments throughout the length of the member, it is

I Associate Professor, Department of Civil Engineering, Carleton University, Ottawa, Canada.

possible to calculate the end rotations by an integration of the curvature diagram. However, if sections of beam have become inelastic through cracking or yielding this integration of the curvature diagram and the subsequent analysis becomes a formidable computational task. Attempts have been made to simplify the above problem by making a reasonable assumption about the location of the inflexion point. The inflexion point is then assumed to be stationary at its assumed location. An end-moment-rotation relationship can thus be derived. This relationship is still non-linear but is represented by a series of straight line segments.

In this study it is assumed that the inflexion point is located at the centre of members. Since all frames analysed are symmetrical and gravity loads are not present, the assumption of a stationary inflexion point is quite reasonable. In fact, for a symmetrical single bay frame, the inflexion point will always be located at the centre of beams. In some of the columns the inflexion point will not be exactly at the centre, but this will not introduce any significant errors because most columns will be found to remain elastic.

Using the moment-curvature relationship of Fig.1, expressions can be obtained for θ_c, θ_y and θ_u the end rotations when the two end moments reach their values at cracking, yielding and ultimate respectively. Since M_c, M_y, and M_u, the moments at cracking, yielding and ultimate respectively, are known the three values of end-rotations derived as above will locate three points on the end-moment-end-rotation curve for the member. The complete moment-rotation relationship is assumed to consist of straight line segments joining these points and the origin and is shown in Fig.2.

The trilinear moment-rotation relationship can be further simplified if the discontinuity at cracking moment is disregarded. The relationship is then represented by two straight line segments, one from the origin to the yield point, shown by a dashed line on Figure 2, and the other from the yield point to the ultimate. As a part of this study the stiffness degrading response of an example frame was obtained with the two alternative hysteretic models, one with a trilinear primary curve, and the other with an idealized bilinear curve. It was found that there was no significant difference between the results obtained from the two sets of analyses.

HYSTERESIS MODEL

As stated earlier, reinforced concrete members do not exhibit the bilinear hysteresis relationship representative of the steel members. The behaviour in case of reinforced concrete is influenced by concrete cracking, reinforcing steel yielding and strain hardening, lack of fit of cracked surfaces and the Bauschinger effect in steel. The most important characteristic is that the reloading slope for the second and later cycles is not equal to that of the initial branch of the primary curve but can be represented by an almost straight line joining the unloading point and the maximum point obtained in the earlier cycles.

Takeda's hysteretic model used in this study considers all of the above factors. A detailed description of the hysteresis rules can be found in reference 5. Some of the rules are illustrated in Fig.3. Nondegrading behaviour is modelled by assuming that during reloading the stiffness does not deteriorate and that the reloading slope is the same as the unloading slope. The corresponding moment-rotation relationship is shown in Fig.4.

DUCTILITY

In this study, ductility is used as a measure of inelastic deformation. It is defined as the ratio of the maximum absolute displacement to the yield displacement. Thus, as shown in Fig.5., rotation ductility μ is given by

$$\mu = \frac{\theta_{max}}{\theta_y} \tag{1}$$

RESPONSE OF SINGLE DEGREE OF FREEDOM SYSTEM

To investigate the nature of the relationship between the elastic and inelastic response of reinforced concrete frames and the effect of stiffness degradation on ductility demand, the single storey concrete frame shown in Fig.6 was analysed for 25 seconds of the design ground motion scaled to represent an earthquake with a maximum ground acceleration of 50% of gravity. The mass of the frame was adjusted to give eight different values of the period ranging from 0.3 to 2.7 sec., giving in effect eight different frames. For each frame an elastic analysis was carried out first with a damping equal to 5% of the critical. The yield strength of each member was then set at one fourth the maximum moment obtained in it during the elastic analysis. Inelastic analyses of the frames were then carried out, first with a non-degrading hysteresis model and then with a degrading hysteresis model The results of the analyses are presented below.

Figure 7 compares the deflection ductility requirements for the degrading and non-degrading systems. In non-degrading systems, the deflection ductility requirements are in general somewhat higher than the force reduction factor of 4 for periods lower than 1.5s. For frames with longer periods, the ductility requirements are smaller than the force reduction factor. Stiffness degradation seems to have a marked effect in the case of short period structures for which it increases the ductility requirements considerably. For longer periods, the degrading and non-degrading responses do not seem to differ much and the difference, if any, is not systematic. The localized rotation ductilities follow the same pattern as the deflection ductilities, although in all cases the former are higher than the latter.

RESPONSE OF MULTISOTREY FRAMES

To investigate the effect of inelasticity and stiffness degradation on the response of multistorey reinforced concrete frames, five and ten storey frames were analysed for the first 10s. of the design ground motion scaled to 50% g. As in the case of a single storey frame, an elastic analysis was carried out first with 5% damping. The yield strengths of the girders were then set at one-fourth the maximum moment obtained in them during the elastic analysis. The column strengths were however kept higher than the maximum moments obtained in the elastic analysis. This virtually eliminated the possibility of inelasticity in the columns.

Response of Five Storey Frames

As in the case of a single storey frame, the mass of this frame was adjusted to give four different frames with fundamental periods equal to 0.5s, 0.965s, 1.5s and 2.0s respectively. The detailed results of analysis for the frame with a period of 0.965s are presented here.

Figure 8 compares the relative storey displacements of the five storey frame for three different models: elastic, stiffness non-degrading and stiffness degrading. The elastic and the non-degrading frames show similar response but the degrading frame shows an accentuated response, particularly in the upper storeys. The girder ductilities are shown in Figure 9. The ductility requirements for the non-degrading frame are close to the reduction factor of 4 except in the uppermost beam. For the degrading frame the girder ductilities are significantly higher.

The increase in response due to stiffness degradation was found to be more pronounced for the frames with a shorter period. Thus in the frame with a period of 0.5s, stiffness degradation resulted in a substantial increase in both the relative displacements and girder ductilities, the increase being greater in the upper storeys. For the frame with a period of 2.0s the stiffness degradation did not have such a marked influence.

A response parameter of considerable interest is the effective reduction factor for the beam and column moments and column axial loads. An effective reduction factor is defined as the ratio of the elastic response value of a force or a moment to its inelastic response value. For the frames under consideration, if the primary moment-rotation relationships for the beams were to be perfectly elasto-plastic, the reduction factors for the beam moments should be uniformly equal to 4 assuming that all beams are strained beyond yielding. However, because the primary moment rotation relationships are bi-linear, the slope of the second branch being about 5% of the slope of the initial branch, the reduction factors are lower than 4 and are closer to 3 as seen in Figure 10. The interesting part is that the reduction factors for the moments in columns which remain elastic are considerably lower and are, in fact, about half those for the beams.

Response of Ten Storey Frame

Figure 11 shows the relative displacements for the three models of the ten storey frame. The non-degrading response is smaller than the elastic response. The degrading response is also smaller than the elastic response in the lower storeys but exceeds the elastic response in the upper storeys. This is clearly the influence of the higher modes of vibration which have a shorter period.

The influence of higher modes is again evident in the girder ductility ratios shown in Figure 12. In the lower five storeys, the degrading and the non-degrading models show similar responses but in the upper storeys the girder ductilities for the degrading model are significantly higher.

The reduction factors for beam moments, column moments and column axial loads are shown in Figure 13. As in the case of five storey frames, the beam reduction factors are less than 4 because of a bilinear moment relationship.

Also the reduction factors for moments in the columns that remain elastic are almost half those for beam moments. Reduction factors for column axial loads are of the same order as those for beam moments.

APPLICATION TO DESIGN

The results in this paper indicate that the correlation between the elastic and inelastic response of a multistorey reinforced concrete frame is at best qualitative. Reasonable estimates of the design member strengths in a frame expected to become inelastic during earthquake can probably be obtained by applying appropriate reduction factors to the forces calculated from an elastic analysis. A uniform force reduction factor does not however ensure uniform ductility demand. The ductility demand is in fact generally larger than the force reduction factor used. Stiffness degradation results in a further increase in the ductility requirement particularly in the upper storeys of the building.

Fortunately the force reduction factor and ductility requirements are of the same order of magnitude and the use of reduction factors to obtain design strengths is still quite reasonable provided such reduction factors are appropriately selected. Except in short period structures, the reduction factor selected for estimating girder strengths may be equal to or a little less than the ductility capacity. The reduction factor may be further reduced in the upper storeys of the frame. Also, if the inelasticity is to be confined to the girders, the reduction factors to be applied to the column moments should be about half those applied to the girder moments.

Gravity loads have not been accounted for in the analyses carried out for this study. If such loads are significant as compared to the earthquake loads, the inflexion point in a girder will no longer be stationary and the analysis will become very complex. Studies carried out on steel frames (3) have shown that when gravity load moments are taken into account both in the analysis and design, ductility requirements in griders are more uniform and closer to the force reduction factors. Gravity loads also result in a decrease in the effective reduction factor for column axial loads so that it is about half that applicable for girder moments. Similar considerations should be expected to apply for reinforced concrete frames, although definite conclusions can be drawn only on the basis of more detailed studies.

ACKNOWLEDGEMENT

This investigation was supported by National Research Council of Canada Grant A3719. The computer program used in this study was developed by A.L. Morris as part of his Masters thesis. Pui Hing Pang, graduate student, helped with the computations. Their contributions are acknowledged.

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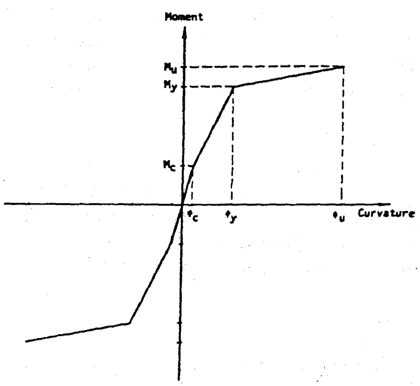


Figure 1
Moment Curvature Relationship for Reinforced Concrete

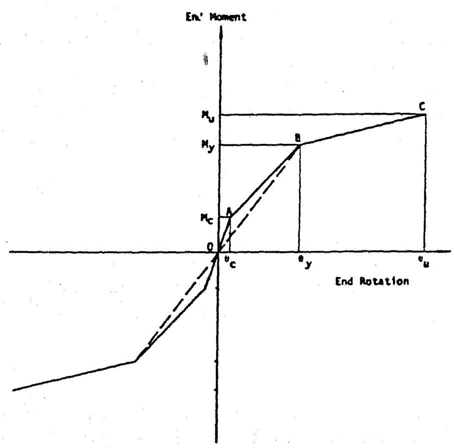


Figure 2
Primary Moment-Rotation Curve

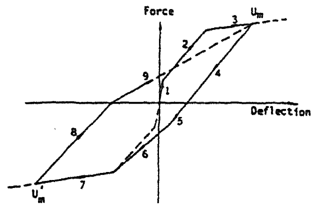


Figure 3
Takeda's Hysteresis Rules

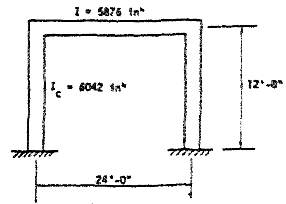


Figure 6
Single Storey Reinforced Concrete Frame

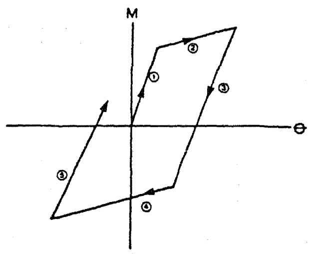


Figure 4
Non-Degrading Hysteresis Loops with Bilinear Spline

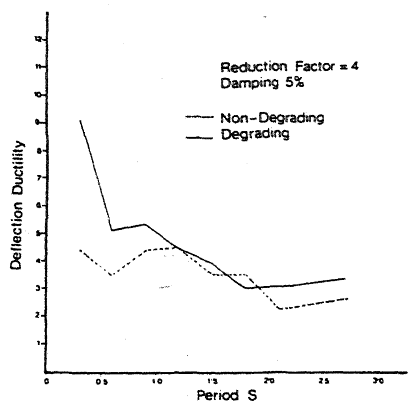


Figure 7
Deflection Ductilities in Single Storey Frames

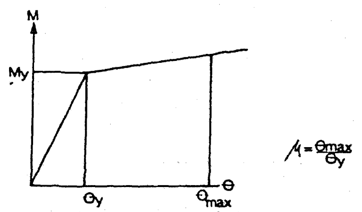


Figure 5
Rotation Ductility

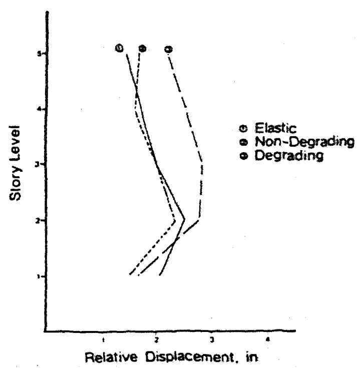


Figure 8
Response of Five Storey Frame

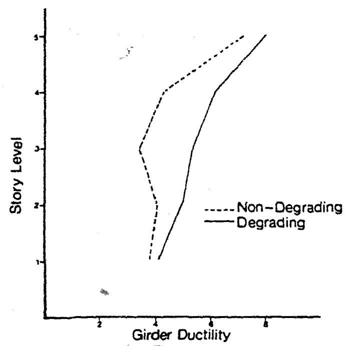


Figure 9
Girder Ductilities in Five Storey Frame

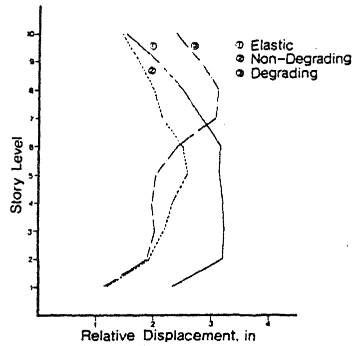


Figure 11
Response of a Ten Storey Frame
Note: 1m = 25.40 mm.

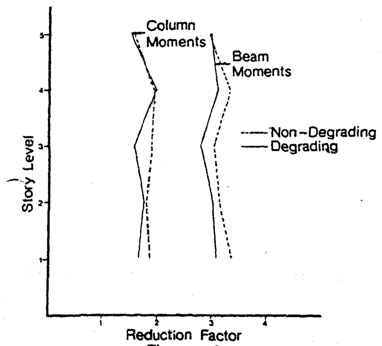


Figure 10
Reduction Factors for the Five Storey Frame

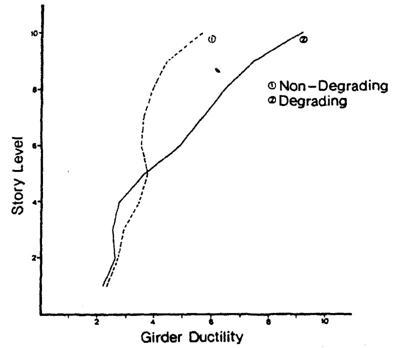
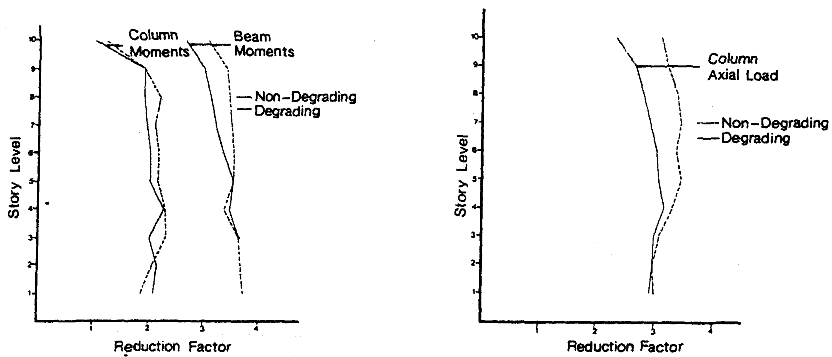


Figure 12
Girder Ductilities in Ten Storey Frame



(a) Reduction Factors for Beam and Column Moments (b) Reduction Factors for Column Axial Loads

Figure 13
Reduction factors for Ten Storey Frame