SIMPLIFIED DYNAMIC ANALYSIS FOR PREDICTION OF DAMAGE IN CODE DESIGNED BUILDINGS

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SUMMARY

This paper asserts the need for a simple analytical method for assessing the actual seismic performance of structures designed by the usual code methods, since these are generally based on the assumption of widely and uniformly distributed dissipation of energy. An extension to the substitute structure method of Shibata and Sozen is suggested which gives good results for coupled shear walls. Application to frame and mixed frame-shear wall structures now under investigation, is promising.

INTRODUCTION

Codes of practice provide equivalent forces which are to be used in a quasi-static analysis, leading to the yield forces and moments for which the structure should be designed. It is accepted that, in a major earthquake, the structure will yield, and deformation will exceed the yield values by some ratio known as the ductility demand. The structure is to be detailed so that the assumed ductility demand can be met. In any portion of the structure in which it is exceeded, excessive damage will result.

The procedures just described are predicated on the assumption that the structure is "well-behaved"; that is, that there are no severe discontinuities in stiffness and mass distribution, no unforeseen torsional moments, nor any unexpected variations in strength, which lead to early local yielding, so that energy absorption is widely distributed across resisting elements. In fact, this is very seldom the case. There is almost invariably a concentration of energy absorption and eventually of damage at certain levels or locations in the structure.

An elastic modal analysis gives a better estimate of the distribution of the elastic forces prior to yielding, but, in the application of such an analysis, it is again assumed that yielding will be widely dispersed. (Refs. 1,2). An inelastic analysis in space and time gives the most accurate prediction of the pattern of yielding and subsequent damage; but it is expensive and is based upon a particular earthquake record. To be sure that an envelope of possible response has been obtained, several records must be used. For buildings of moderate importance, the expense in time and money of such an elaborate procedure is not warranted, and, in any case, it is doubtful whether the accuracy is sufficient to justify it.

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It is contended, therefore, that a cheaper and quicker method is required to check a given design, as it nears completion, to see how well the code assumptions are fulfilled, and to identify potential trouble spots. (Refs. 3,4).

In elastic modal analysis, the available ductility is known in advance (based on the structural form and the intended level of detailing) and yield forces are predicted. What is needed is a procedure starting with the yield forces, which have been designed on the basis of the code forces and the gravity loads, and leading to the ductility demands in the various elements of the system. These are indicative of the probable damage pattern, and can be assessed accordingly. Such a procedure is useful for the investigation of existing buildings, as well as for the final design stage of new ones. It is essential in the design of coupled shear wall systems, where there are certain to be larger ductility demands in the coupling beams than are implied by code design procedures.

One method meeting these criteria has been advanced by Freeman (Ref. 4). That advocated by the writers has been described elsewhere (Ref. 5), but will be outlined briefly here, together with discussion of further applications.

PROPOSED METHOD

The procedure is an extension of the substitute structure method developed by Shibata and Sozen (Ref. 6). The latter is based on the fact that the secant stiffness of an elastic-plastic system is equal to the initial stiffness divided by the "damage ratio", μ. The latter, equal to the ductility in an elastic-perfectly plastic system, is known in the design stage - i.e. the target value is dependent on the structural form and intended level of detailing. The hysteretic damping associated with the actual structure is artificially replaced by viscous damping, also dependent upon the ductility, and thus a substitute elastic structure with reduced stiffness and substitute viscous damping can be modelled to give the real design forces. The details are given in Refs. 5 and 6.

The extension consists of an iterative procedure involving the following steps:

1. An elastic modal analysis is made of the structure, using, in the first iteration, the initial stiffnesses and appropriate damping values. The root-sum-square forces are calculated.
2. In subsequent iterations, the analysis is repeated, but those members whose end moments exceed their yield values, have their stiffnesses reduced to

$$k_{n+1} = k/\mu_{n+1} \tag{1}$$

$$\mu_{n+1} = \mu_n \frac{M_n}{M_y} \; \frac{1}{\left[1 + \frac{k'}{k}\left(\mu_n \frac{M_n}{M_y} - 1\right)\right]} \geqslant 1 \tag{2}$$

k = initial stiffness.
k' = post-yield slope of bilinear force/deformation relationship for member.
k_i = stiffness used in ith iteration.
μ_i = damage ratio in ith iteration.
M_n = larger root-sum-square end moment from nth iteration.
M_y = yield moment.

For each iteration after the first, a smeared damping value for each mode is calculated as suggested by Shibata and Sozen:

$$\beta = 0.02 + 0.2(1 - 1/\mu^{1/2}) \tag{3}$$

$$\beta_r = \frac{\sum_i p_i^r \beta_{si}}{\sum_i p_i^r} \tag{4}$$

$$p_i^r = \frac{L_i}{6\,(EI)_{si}} \left[(M_{ai}^r)^2 + (M_{bi}^r)^2 - (M_{ai}^r)(M_{bi}^r)\right] \tag{5}$$

where β = substitute damping for member with damage ratio μ.
β_r = smeared damping for rth mode.
β_{si} = substitute damping for ith member.
L_i = length of ith member.
M_{ai}^r, M_{bi}^r are the moments at the ends of the ith member in the rth mode.
$(EI)_{si}$ is the cracked rigidity of member i.

3. When all the member forces are either below or within a tolerable limit of their yield values, the analysis is halted. The damage ratios, or the ductility demands implied thereby, are the required values.

COMMENTS ON PROPOSED METHOD

A twofold convergence criterion has been used: all maximum end moments should be less than or within 5% of the moment capacity of the member; and a limit should be placed on the change in damage ratio from the penultimate iteration to the last. This limit was set at 1% or 2% of the last damage ratio (if that was greater than 5) or at an absolute value of 0.1 if the final damage ratio was less than 5. A convergence speeding procedure was found useful: when there was a monotonic trend in any damage ratio, it was overcorrected somewhat for the next iteration.

When the convergence criteria are met, the solution is that corresponding to a substitute structure with the correct moments and damage ratios for the selected substitute damping, which satisfies the modal analysis. Whether or not this solution is unique is currently under investigation. For structures with a limited number of load paths, such as a pair of coupled shear walls, good agreement with a non-linear time step analysis has generally been obtained. In a large frame structure with several bays and hence several alternate load paths, the results are less certain. They appear to be more sensitive to the accuracy with which the substitute damping represents the actual hysteretic damping and to the convergence criteria. A study of these effects is proceeding.

APPLICATION OF PROPOSED METHOD

The layout of the principal structures studied is shown in Fig. 1. The structures were represented by line members with rigid joint areas; nodes were placed on the neutral axes of the uncracked members, although it is recognized that this position would not really be fixed during dynamic response. Viscous damping was included at 2% of critical to represent the effect of non-structural components, since hysteretic damping of the structure itself is, of course, included automatically in the non-linear analysis. (Note that the first term in eq. [3] represents the damping when $\mu = 1$: when the member is still elastic.) All test structures were assumed to have fixed bases, representing the typical case where walls terminate in the larger walls of the basement parking structure. Otherwise member sizes and loads were chosen to be reasonably representative of practical cases.

Five storey coupled walls were studied, with coupling beam capacities of 60 Kips-ft and 100 Kips-ft; peak ground acceleration was set at 20% and 50% of gravity, and the masses were varied by a factor of 4 to examine the effect of period change. Also a ten storey coupled wall and a 16 storey coupled wall connected to an extra uncoupled wall similar to the structure discussed by Fintel and Ghosh, Ref. 7, were tested. In the latter case, 2% viscous damping was used, rather than 10%, as discussed above. Furthermore, it was found that, at the high ductility demands occurring in this structure, the 5% strain hardening used by Fintel and Ghosh led to ultimate moments equal to about twice the yield moments. To be consistent with the program written for the proposed method, based on elastic-perfectly plastic response, the strain hardening was reduced to 0.5%. These changes are not, of course, necessary for the method; they were only required to be consistent with the parameters previously adopted for the present study. This structure had a stiffness change at midheight, as well as variations in the masses, so that it provided a severe test of the method.

A 7-storey 8-bay frame and a 7-storey 5-bay single shear wall building, which are intended to be representative of more general structures, were also tested by the proposed method.

Earthquake Spectrum A (Ref. 6), which was developed by Shibata and Sozen in their original work, was employed with the proposed method in the test of the 5, 10 and 16 storey coupled shear wall structures; the C.I.T. synthetic C-2 spectrum was used in the analysis of the 7-storey 5-bay single shear wall building and the 7-storey 8-bay frame.

RESULTS OF COMPARISONS

Typical results for the 5-storey structures are shown in Figs. 2 to 5. The proposed method showed the correct distribution of damage ratios, but was excessively conservative at lower periods. This may be due to spectrum A being a rather conservative envelope at this period. At higher periods the method worked well, although it is clear that the true response varies widely and unpredictably with earthquake input, and that great accuracy is not attainable by any modal method.

Results of the 10 storey walls are shown on Figs. 6 and 7. At these periods use of spectrum A with the proposed method yielded a close upper bound on the response, with an accurate prediction of the form of the damage ratio distribution.

In the case of the 16-storey structure, Figs. 8 to 10 show that the procedure again gave good qualitative results, with reasonable quantitative accuracy. The dependence of the actual detailed response on earthquake input is again evident.

The results of the analyses for the 7-storey mixed frame-shear wall and 7-storey 8-bay frame, shown in Figs. 11 and 12, indicate that good results can possibly be obtained with more general structures. However, as noted above, these results are sensitive to the substitute damping and convergence criteria, which are presently under further study; the damage ratios reported should therefore be considered as preliminary only.

CONCLUSIONS

The proposed procedure provides a relatively rapid method of analysing frames, coupled shear walls and mixed frame-shear wall systems. It is no more expensive to perform than the cost of the usual elastic modal analysis of frame structures, but it also defines the actual distribution of ductility demand throughout the structure. The procedure can be used for the investigation of existing buildings, as well as for the final design stage of new ones. The results obtained by this approach are generally in good agreement with more sophisticated and expensive non-linear analysis techniques. Further study of the method of handling substitute damping and the convergence criteria is needed.

ACKNOWLEDGEMENTS

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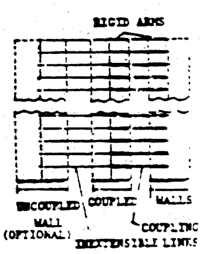


FIG. 1 GENERAL TEST STRUCTURE CONFIGURATION

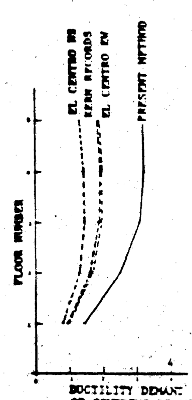


FIG. 2 5-STORY WALL

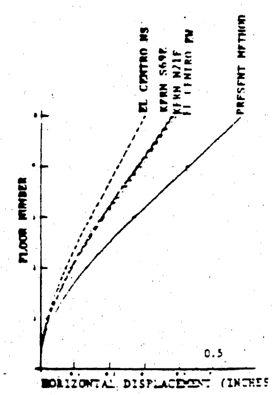
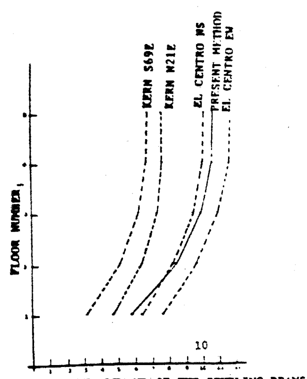
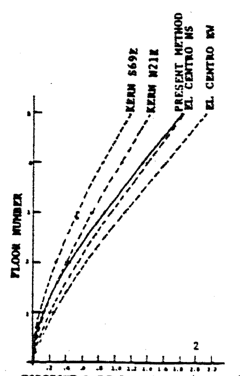


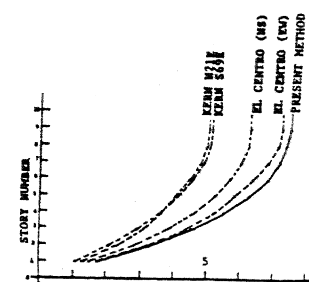
FIG. 3 DISPLACEMENT ENVELOPES FOR THE 5-STORY WALL



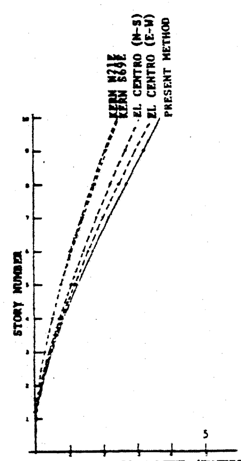
DUCTILITY DEMAND OF THE COUPLING BEAMS
FIG. 4 5-STORY WALL
(MASS=4 TIMES ORIGINAL RUN)



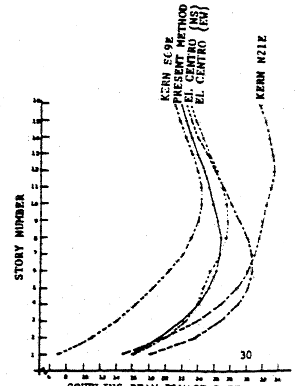
HORIZONTAL DISPLACEMENT (INCHES)
FIG. 5 DISPLACEMENT ENVELOPES
FOR THE 5-STORY WALL
(MASS=4 TIMES ORIGINAL RUN)



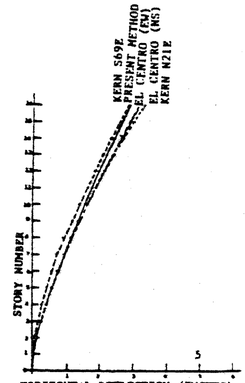
COUPLING BEAM DAMAGE RATIO
FIG. 6 TEN STORY WALL.



HORIZONTAL DISPLACEMENT (INCHES)
FIG. 7 DEFLECTION ENVELOPES
FOR THE 10-STORY WALL



COUPLING BEAM DAMAGE RATIO
FIG. 8 DAMAGE RATIOS FOR THE 16-STORY
COUPLED WALL WITH ATTACHED
UNCOUPLED WALL



HORIZONTAL DEFLECTION (INCHES)
FIG. 9 DEFLECTION ENVELOPES
FOR THE 16-STORY
COUPLED WALL WITH ATTACHED
UNCOUPLED WALL

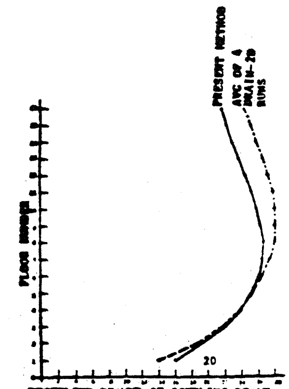


FIG. 10 AVERAGE DUCTILITY DEMAND OF COUPLING BEAMS FOR THE 16-STORY COUPLED WALL WITH ATTACHED UNCOUPLED WALL

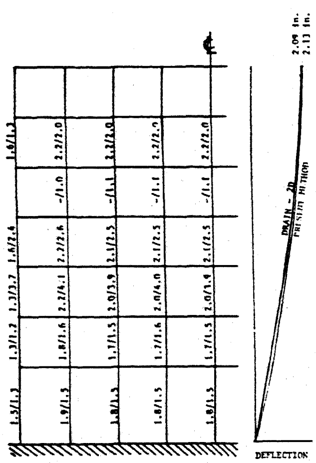


Fig. 11 DUCTILITY DEMANDS EXCEEDING UNITY FOR 6-BAY FRAME. VALUE FROM PRESENT METHOD/VALUE FROM DRAIN-2D

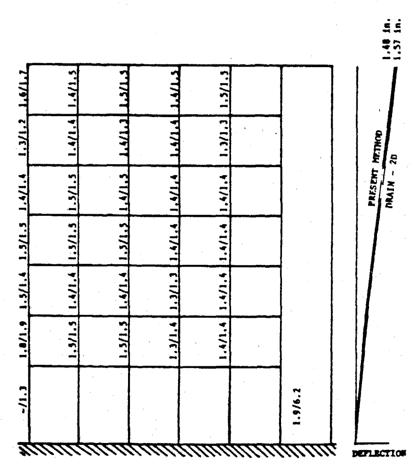


Fig. 12 DUCTILITY DEMANDS EXCEEDING UNITY FOR MIXED FRAME SHEAR WALL SYSTEM. VALUE FROM PRESENT METHOD/VALUE FROM DRAIN-2D