EVALUATION OF ASEISMIC DESIGN PROCEDURES FOR REINFORCED CONCRETE FRAMES

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SUMMARY

The effectiveness of two aseismic design procedures, the UBC method and the inelastic response spectrum approach, are evaluated by subjecting frames so-designed to earthquake ground motions and computing ductility demands by inelastic time-history analysis. Neither method produced consistently satisfactory results because yielding was not uniformly distributed over the frame and larger than desired ductility demands occurred locally. The inelastic spectrum approach, although apparently more exact, did not produce significantly better results than the simpler code method. A major cause of large local yielding is a lack of balance between member strengths at a joint and research is needed to develop design rules in this respect.

INTRODUCTION

During the past several years our ability to analyze structures for the effects of earthquake ground motions has increased dramatically. The considerable time, effort and money which has been expended has, in many respects, been rewarding. We have certainly learned a great deal about the behavior of structures during earthquakes. However, we are still far short of the ultimate goal, i.e., the development of reliable techniques for the design, as opposed to mere analysis, of structures to withstand earthquakes. Specifically, reference is made here to the design of conventional building structures which are intended to behave inelastically during the design earthquake. It is not clear which is the best and most practical analytical method for use in the design process, nor how one translates the results of such analysis into predictions of damage. The research reported herein was an attempt to address some of the issues. Design is considered here in the narrow sense, i.e., the proportioning of the members of a structure whose type and geometry have been established.

With regard to methods of design analysis, there are several possibilities. First, there is the approach specified by most current codes, i.e., the use of equivalent static loads, much reduced from the forces actually expected, coupled with elastic analysis and conventional allowable stresses or load factors. While this may be considered satisfactory on the basis of past experience, the results of the analysis bear little relation to the actual behavior of the structure. The procedure implies inelastic response, but provides no indication of how much yielding of the structure is to be expected. It denies the designer any control over the amount or location of the inelastic behavior. One step beyond the conventional code procedure is the use of elastic dynamic analysis rather than equivalent static loads. Two methods have been employed, modal analysis using response spectra and time-history analysis using real or artificial ground motions. The response spectrum method is preferable because of its relative simplicity and because

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a spectrum can reasonably be constructed to envelop possible ground motions at a site, although it is not possible to predict the details of ground motion. Regardless of which method is used, elastic dynamic analysis is not satisfactory because it fails to predict the nature of inelastic behavior or any reliable measure of the risk of failure.

It is concluded from the above discussion that some method of inelastic dynamic analysis should be employed. However, this involves some difficulties which have not yet been overcome. Time-history analysis could be utilized, and the tools for this approach are available. However, this is not really a practical design procedure. The other method which has been suggested involves the use of an inelastic response spectrum (Ref. 1) together with a pseudo-elastic modal analysis. This is attractive because it is reasonably simple and permits the designer some control over the amount of yielding. However, such spectra are based upon the response of one-degree systems and the reliability of the method when applied to multi-degree systems has not been demonstrated. One of the purposes of the research reported herein was to further investigate this approach.

Even if a satisfactory inelastic analysis technique, practical for design purposes, is developed, the question remains as to which computed response parameter is a reliable indicator of damage. Much research (including that reported here) has been based upon maximum ductility ratios, but this is probably not a true indication of the amount of damage. Other quantities may be better measures. If inelastic analysis is to be the basis for design, further research is needed to establish design criteria which would ensure acceptable levels of damage.

The purpose of the research partially reported here was to evaluate the effectiveness of alternative aseismic design procedures. The evaluation is based upon inelastic time-history analysis of frames, the members of which had been proportioned by some fixed procedure. A satisfactory design is judged to be one which limits the maximum local ductility demands to the value intended and produces a reasonably uniform distribution throughout the frame. Local ductilities were computed at each end of each member due to artificial ground motions, generated to match a smoothed design response spectrum (see Fig. 1).

This paper is limited to a discussion of reinforced concrete building frames of four and ten stories. Two methods of design are investigated: (1) the UBC-1973 approach employing equivalent static loads, and (2) the inelastic response spectrum approach using pseudo-elastic modal analysis. In both cases the members were proportioned using ACI-318. The intent was to evaluate these procedures as they might be typically applied in practice. It is assumed that the reinforcement is properly detailed so as to provide the ductility required. A more complete description of the research may be found in Ref. 2.

FRAMES DESIGNED BY CODE

Several frames were designed using an unmodified application of UBC-1973 (Zone 3) and ACI-318-71. It is not believed that use of later versions of these codes would change the general conclusions. The designs were evaluated by subjecting them to three artificial motions, all generated to match the Newmark-Blume-Kapur elastic spectrum with a peak ground acceleration of 0.33g (Fig. 1). It is believed that the amount of yielding produced by this level of input provides a reasonable evaluation of the design consistent with the intent of the code.

Typical of the results for low-rise frames are those shown in Fig. 2. These are for a four-story frame which is three bays wide. The plots display the maximum computed ductility demand in each story for the members indicated. Thus each value plotted is the largest of those computed at any end of those members. As expected, the three motions produce different results. However, in general, the ductility demands are not excessive and seem to imply a moderate amount of damage considerably short of collapse. The maximum demand for any motion at any point is 7.5 in a first-story interior column. The relatively small values in the upper stories result from the fact that member design in this region was controlled by gravity loads without earthquake. Although the intent of the Code in terms of ductility demand is not stated, it is concluded from these and other results that for low-rise frames the Code produces a reasonable and adequate design. However, more uniform yielding, with less in the first-story columns, would be desirable.

Results of a different character are shown in Fig. 3. These are for a ten-story, three-bay frame also designed by UBC/ACI. In this case the requirement that column strengths exceed girder strengths at a joint was deliberately ignored, but this is not entirely responsible for the nature of the result. The large ductility demands in the upper stories are certainly undesirable and probably unacceptable. This "whip-lash" effect has also been observed by other investigators and is commonly attributed to the effects of higher mode participation. In fact, inspection of the time-histories of interstory displacement in the upper stories do reveal oscillations at what appear to be second or third mode frequencies (somewhat lengthened by inelastic behavior). However, this is not a complete explanation. It appears, from inspection of computed local ductility ratios, that of greater importance is a lack of balance between member strengths at a joint.

It is significant that the girder ductility demands associated with positive bending at a joint are always much larger than those associated with negative bending. Following the code requirement, positive bending capacity was made one-half of negative capacity, even though this is larger than the seismic moment indicated by analysis. As a result, the sum of the column moment capacities at exterior joints far exceeds the positive girder capacity. This accounts for the large ductility demands displayed for the eighth and ninth story exterior girders and suggests that the positive bending strength of girders should be larger. It should be recognized that, once yielding has occurred at both girder ends, the combination of elastic gravity and earthquake moments is meaningless. On the other hand, the large column ductility demands in the eighth and ninth stories result from the fact that the column strength is considerably less than the girder strength in this particular design. An increase in column strength would improve its performance but might cause excessive damage to the girder.

On the basis of this limited investigation, it is tentatively concluded that the Code approach produces a reasonable result in terms of protection against catastrophic damage. However, in at least some frames of moderate height, excessive damage can occur in the upper stories. This is due in part to higher mode effects not adequately considered by the UBC (even the latest version). Of perhaps greater importance, and this applies to frames of any height), greater attention should be paid to the balance between member strengths at a joint to avoid excessive damage to the weak member. In this respect the results of static elastic analysis are of little use.

DESIGN USING INELASTIC RESPONSE SPECTRA

In has been proposed that design be based upon response spectrum modal analysis using the elastic spectrum reduced by factors dependent upon the desired maximum ductility ratio and to determine the required member capacities as would be done in an elastic design. It has the advantage that it accounts for higher mode participation and presumably gives the designer control over the amount of yielding. Fig. 4 shows inelastic spectra constructed according to rules suggested by Newmark and Hall (Ref. 1). The example shown is for a peak ground acceleration of 1g, 5 percent damping, and a ductility ratio of 5. Starting with a given elastic spectrum, two inelastic spectra, one for acceleration and one for displacement, are constructed by applying the factors μ and $\sqrt{2\mu-1}$ (μ = ductility ratio) as indicated. As applied herein, the procedure is to read the inelastic acceleration spectrum for each structural mode, apply the resulting inertia forces to obtain internal member forces, and then combine the modes by the square root of the sum of the squares. This pseudo-elastic procedure is of course approximate, but it would seem that the greater sophistication should produce a better result than the equivalent static load code approach.

Presented here are two reinforced concrete frames, of four and ten stories, designed by the response spectrum method and evaluated by time-history analysis. They are similar to those designed by code and previously discussed. The elastic design spectrum is for a peak ground acceleration of 0.33g and 5 percent damping (Fig. 1). The design ductility ratio was taken as 4. The members were proportioned according to ACI-318 except that no load factors were applied, and the requirement that column strengths at a joint exceed girder strengths was ignored. Although this requirement may be desirable, its use would obscure the intent of the investigation.

The results of the time-history analysis for the four-story frame are shown in Fig. 5. Compared to the four-story code design, the columns of this frame are considerably weaker, but the girders are about the same. The ductility demands, on the average, are reasonably close to the target value of four, although Motion 1 produced some fairly large values. The columns performed quite well. The upper story girders were subject to little yielding because gravity loads controlled the design. The second story exterior girders yielded more than intended, because the column moment capacities considerably exceeded the positive moment girder capacities. Comparing the two designs (Figs. 2 and 5), the response spectrum approach produced a slightly better distribution of yielding over the frame, but the average ductility demands were about equal. It is concluded that the response spectrum method provided some improvement, but not a major one, over the simpler code approach.

When the ten-story frame was designed by response spectrum, the result, compared to the code design, was moderately stronger girders and weaker columns. The results in Fig. 6 show that the "whiplash" effect is still present, although the large ductility demands occur at a lower story than for the code design (Fig. 3). In fact, the ductility demands, on the average, are somewhat larger than for the code design. This is surprising, since the response spectrum method presumably takes into account the higher mode effects. As in the code design, the largest ductility demands occurred where there was a poor balance between column and girder strengths. The large demand on the interior, seventh-story column resulted from low strength compared to the negative girder moment capacity. The large demand on the exterior girders in the seventh and eighth stories occurred because the positive moment strength was small compared to those of the exterior columns.

Comparing Figs. 3 and 6, it must be concluded that for the ten-story frame the inelastic response spectrum method provided no improvement over the simpler code approach. Considering all frames investigated, including some not presented here, the spectrum method, although apparently more accurate, provides marginal, if any, improvement in the design of reinforced concrete frames.

CONCLUSIONS

On the basis of this limited investigation, the following tentative conclusions are drawn regarding the aseismic design of reinforced concrete frames. (1) At the present time there apparently is no reliable yet practical design method which will ensure a desired general level of yielding and prevent excessive local yielding at certain points in the frame. (2) Neither the code nor the inelastic response spectrum methods investigated here gave consistently satisfactory results. However, the computed response did not indicate collapse and, if prevention of collapse is the objective, both methods might be considered satisfactory. (3) The inelastic response spectrum approach, although apparently more exact, did not produce significantly better designs than the simpler code approach. However, it has the potential for becoming a reliable yet relatively simple method of design and should be further developed. (4) Large local yielding occurs when there is a poor balance between member strengths at a joint. It is not known how to determine a proper balance, but the results of elastic analysis are a poor indicator. Further study of this subject is required. (5) Design must be based on inelastic behavior and the procedure should give the designer control over the amount of yielding. The analytical tools are available. Future research should concentrate on the design rather than the analysis problem.

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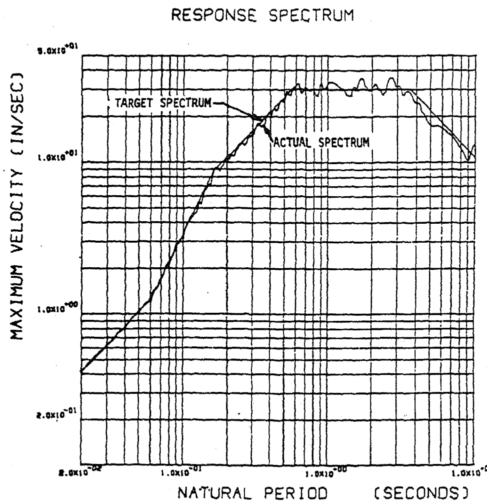

Fig. 1. Response Spectrum of Typical Artificial Motion

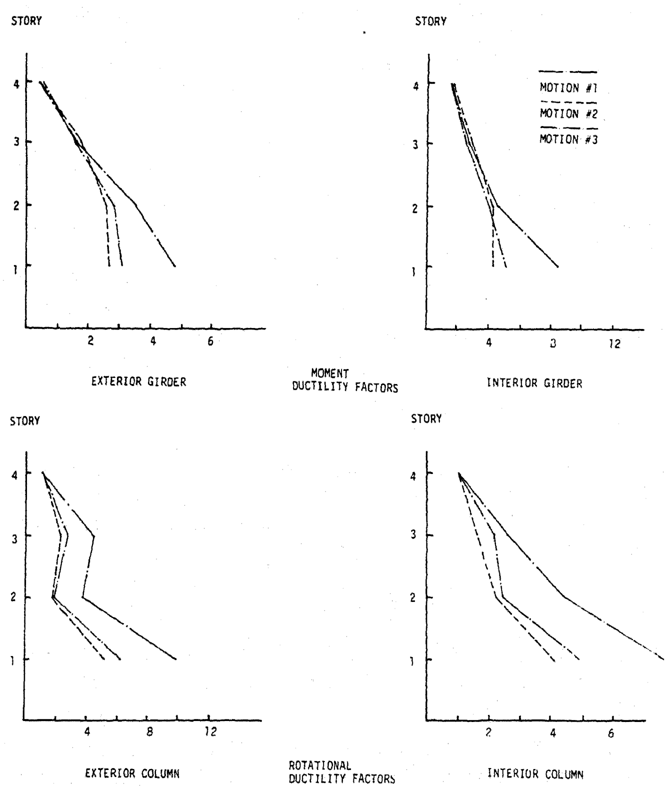

Fig. 2. Computed Ductility Demands. UBC/ACI Design

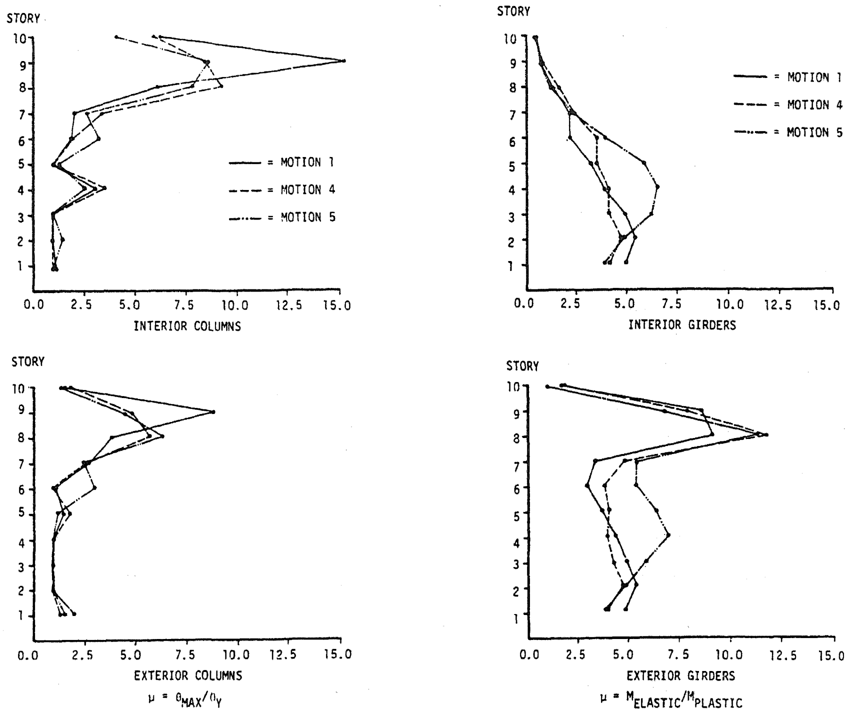

Fig. 3. Computed Ductility Demands. UBC/ACI Design

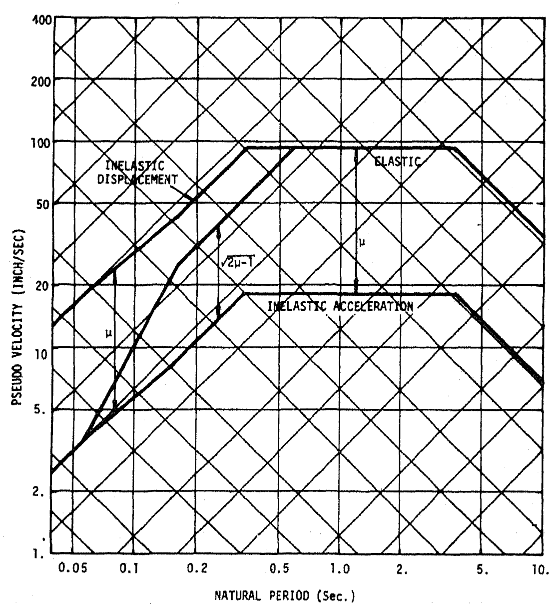

Fig. 4. Newmark-Hall Inelastic Response Spectra

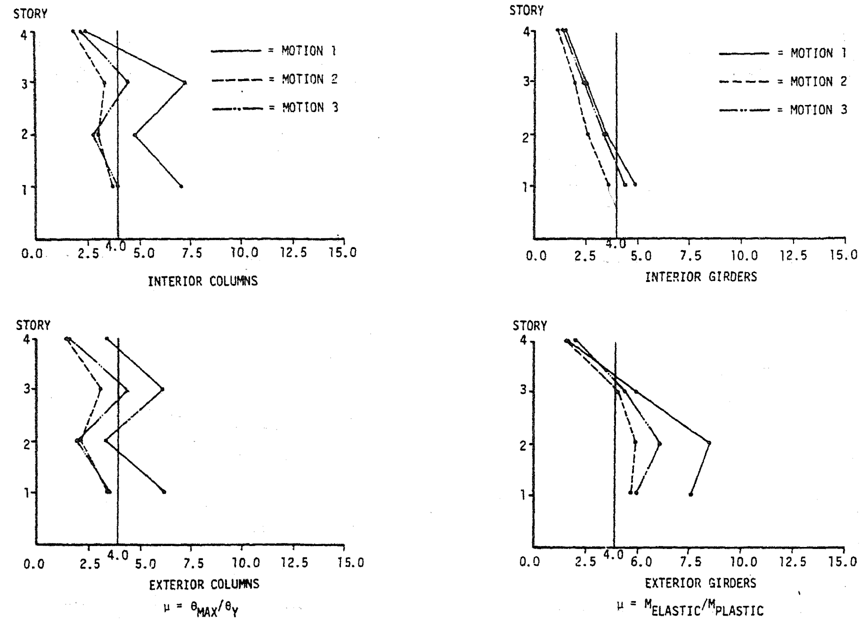

Fig. 5. Computed Ductility Demands. Response Spectrum/ACI Design

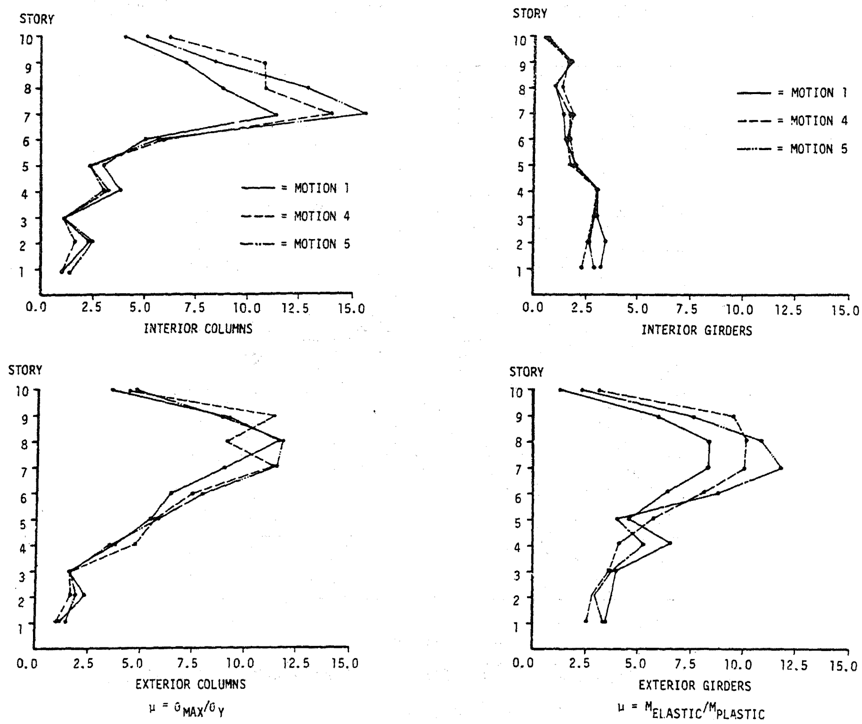

Fig. 6. Computed Ductility Demands. Response Spectrum/ACI Design