DESIGN OF REINFORCED CONCRETE FRAMES USING INELASTIC RESPONSE SPECTRA

Edwin G. Burdette (I)
David W. Goodpasture (II)
Dennis Bernal (III)
Presenting Author: Edwin G. Burdette

SUMMARY

Two reinforced concrete rigid frame structures were designed using the Inelastic Response Spectrum Approach (IRSA), with and without considering moment redistribution as permitted in the ACI Building Code. The N-S component of the 1940 El Centro earthquake was used in the determination of spectra for various ductility ratios. The completed structures were then subjected to this earthquake motion and a time-history analysis performed using DRAIN-2D (Ref. 1). The resulting ductility demands were compared with predicted and allowable values. The results were inconsistent: for the six story frame the designs were generally adequate; for the ten story frame, the inelastic rotations were excessive.

INTRODUCTION

Most structures in seismic regions are designed for seismic effects using code-specified equivalent forces to account for the inertial forces generated in the event of an earthquake. These design forces are much smaller than those which would actually be generated if the structure were to remain elastic even in an earthquake of moderate intensity. Thus, the assumption that inelastic action will occur is inherent in the use of current building codes such as the Structural Engineers Association of California (SEAOC) Code (Ref. 2) or the Uniform Building Code (UBC) (Ref. 3).

To design a structure to remain elastic under earthquake loading is impractical for most structures. Similarly, to account for inelastic behavior through the use of a rigorous, time history analysis is expensive and time consuming; and such an analysis requires both a level of expertise and the existence of computer capabilities not typically present in a structural engineering design office. A widely recognized need exists for a design procedure or a design code which provides for inelastic behavior in a more rational way than do current codes. The Inelastic Response Spectrum Approach (IRSA) combines the simplicity

I Professor of Civil Engineering, The University of Tennessee, Knoxville, Tennessee

II Professor of Civil Engineering, The University of Tennessee, Knoxville, Tennessee

III Consulting Structural Engineer, Santo Domingo, Dominican Republic

required for eventual incorporation into a code approach with the advantage that inelastic demands are explicitly considered. Unfortunately, the IRSA is not rigorously applicable to multidegree of freedom systems, and its applicability to multi-story rigid frames must be investigated before the method can be used with confidence.

Objective

The primary objective of the work reported in this paper was to investigate the applicability of the IRSA to the design of reinforced concrete rigid frames for earthquake loading. The applicability was assessed by comparing the inelastic demands predicted in the design using IRSA with actual inelastic demands obtained from a rigorous, step-by-step, inelastic dynamic analysis.

Scope

The scope of the research reported here was limited as follows:

(1) The one earthquake motion considered was the N-S component of the El Centro earthquake.

(2) Two rigid frames were considered, a six-story, single bay frame and a ten-story, three bay frame.

(3) Gravity loads in combination with seismic loads were considered, and the effects of moment redistribution were investigated. The effect of degrading stiffness was not considered.

OUTLINE OF DESIGN USING IRSA

Before the IRSA can be employed to design a structure, an earthquake ground motion must be selected and inelastic response spectra obtained. These spectra consist of plots of spectral acceleration versus period for selected values of ductility ratio, where spectral acceleration is defined as that value of the acceleration which, when multiplied by the mass of the oscillator, yields the maximum force applied to the structure. Once these inelastic response spectra are available, design using the IRSA may proceed in the manner outlined as follows:

(1) Select preliminary member sizes. Consideration of drift limitations and depth-to-span ratios will influence choice of column and beam sizes, respectively.

(2) Determine design live load. These loads depend on the function the structure will serve and will be chosen from an appropriate code such as the Uniform Building Code (UBC) or Standard Building Code (Ref. 4).

(3) Obtain forces and moments from dead and live loads and from earthquake loads as obtained from a UBC analysis.

(4) Resize members as necessary based on the forces and and moments in Step 3. Repeat Step 3. Continue until an acceptable design is obtained.

Steps 1-4 constitute a preliminary design of the structure, a necessary preamble to the application of the IRSA.

(5) From a dynamic computer analysis (using gross moments of inertia, I gross, and including member end joint sizes) obtain the modal shapes and periods for the frame.

(6) From an IRSA plot obtain the spectral accelerations for the desired design ductility at the periods obtained in Step 5.

(7) Apply these spectral accelerations plus dead, live and P-Δ loads to the frame. Make a computer analysis (using I gross and member end joint sizes) to obtain forces and moments at the faces of supports.

(8) Design reinforcement using the forces and moments from Step 7. Note: The IRSA moments for the columns are to be factored by a column amplification factor, α, before designing the columns.

The purpose of the column amplification factor, α, noted in Step 8, is to insure strong column-weak beam behavior, that is, to assure that hinging will take place in the beams rather than in the columns. The values of α required to assure such behavior were found to depend to some degree on the design ductility ratio. The values used for α were 1.35, 1.70, 1.85, and 2.00 for design ductility ratios of 2, 4, 6, and 8, respectively (Ref. 5). The design moments for the columns were obtained by multiplying the column earthquake moments by the appropriate factor and adding these moments to the moments from the dead and live loads. The design moments for the beams were determined from the larger of (a) 1.4 x dead load + 1.7 x live load or (b) dead load + live load + earthquake load. Typically, the latter controlled the beam design.

DESCRIPTION OF RESEARCH PROJECT

Two reinforced concrete rigid frames were designed using the IRSA. One frame was a six-story, one bay structure with overall height of 75 ft.-10 in. and bay width of 35 ft. All beams were 14 x 26-in. with a 5-in. slab. The columns varied from 24 x 16-in. in the lower stories to 16 x 16-in. in the top stories. A second frame was a ten-story, three bay structure with overall height of 124 ft. and bay widths of 25, 30, and 25 ft. All beams were 12 x 22-in., and the columns varied from 24 x 24-in. to 14 x 14-in. The inelastic response spectra used in the designs were obtained for the N-S component of the El Centro earthquake using 5 percent of critical damping in the first two modes. Designs were made for ductility ratios of 2, 4, 6, and 8. These designs were made according to the procedure described in Section 2, utilizing the strong column-weak beam concept.

Each structure which had been designed using the IRSA with a particular ductility ratio was analyzed through the use of a computer program, DRAIN-2D, which is capable of performing an inelastic time history analysis. The input seismic loading consisted of the accelerations from the N-S component of the El Centro earthquake, consistent with the IRSA. From the computer program the peak response of each structure was determined, including the magnitude of inelastic rotations. The inelastic responses of the structures obtained from DRAIN-2D were compared to the inelastic responses consistent with the choice of ductility ratios for the IRSA. The results of these comparisons permit an evaluation of the applicability of the IRSA and are discussed later herein. These results are presented for cases with and without redistribution of moments in the design. When moments were redistributed, the requirements of Section 8.4 of the ACI Building Code (Ref. 6) were followed. More thorough descriptions of the research project and complete listing of results are given in Refs. 7 through 9.

RESULTS

The computer program, DRAIN-2D, gave a complete load-deformation time history of each structure, including the magnitudes of the inelastic rotations. Two quantities obtained from DRAIN-2D were particularly useful in assessing the adequacy of design by IRSA. The values of the story ductilities were compared to the IRSA design ductilities, and the magnitudes of the calculated inelastic rotation capacities were compared to allowable inelastic rotations.

Story Ductilities

The story ductility ratio was calculated as the difference in horizontal drifts at two consecutive floor levels divided by the difference in the yield drifts. Ideally, the story ductility ratio for each story of a rigid frame would turn out to be the same value as the design ductility ratio used in the IRSA design. Thus, a comparison of story ductilities obtained from DRAIN-2D with the IRSA design ductilities provides at least a crude measure of the accuracy and applicability of the IRSA.

The results of the comparisons of story ductilities were variable for the several design ductilities of 2, 4, 6, and 8 and even for the different stories of a frame for a particular design ductility. Two trends were consistent for both frames: (1) there was no significant difference between the results with and without moment redistribution; (2) the value of story ductility was typically much higher at the top story than at any of the others. The range of story ductilities for both the six and ten story frames is given in Table 1.

Inelastic Rotations

Although calculation of the story drift ductility ratio provides an interesting comparison to the design ductility ratio, it is not critical to the performance of the structure. The critical comparison in determining the adequacy of a frame designed by the IRSA is the comparison of actual plastic hinge rotations to the allowable rotations. Also, a method was developed in Reference 10 to predict plastic hinge rotations for a particular ductility ratio; the actual rotations were compared to these predicted values.

In order to make the critical comparison just described, the values of allowable inelastic rotation for the positive and negative moment regions of each beam had to be determined. An extensive literature search revealed little data directly applicable to rotation capacity of beam-column joints. The best information obtainable for rotation capacity prediction was published in References 11 and 12. Based on these data, obtained for simple beams, the rotation capacity of a beam cross-section was calculated as the product of the difference between the ultimate and yield curvatures and one-half the effective depth of the beam.

A comparison of actual and allowable rotations is given in Table 2. Values in the table greater than unity indicate that the beam has exceeded the allowable rotation capacity and has therefore failed. Results of the comparison of actual and predicted rotations were as scattered as those in Table 1b. The actual rotations varied from zero to more than three times the predicted rotations.

DISCUSSION AND CONCLUSIONS

The results presented in Tables 1 and 2 indicate the inconsistency of the results obtained for the frames designed with the IRSA. The story ductilities varied significantly from the design ductilities for both the six and ten story frames and were in all cases greater than the design ductilities. For each frame and each design ductility ratio, the value of story ductility was larger for the top stories.

The inelastic rotations required were also somewhat inconsistent, although the rotations for the six story frame in no case exceeded the calculated rotation capacity. On the other hand the original designs of the ten story frame experienced excessive rotations for all design ductilities. A redesign of these frames taking advantage of moment redistribution resulted in inelastic rotations which were acceptably low except for the design ductility of 2. Moment redistribution resulted in designs with reduced negative moment steel and increased positive moment steel. The resulting increase in rotation capacity in the negative moment regions coupled with the change in inelastic behavior of the frames due to the revised capacities apparently produced more favorable conditions in the ten story frame. This situation did not occur in the six story frame. Thus, a general statement defining the effects of moment redistribution cannot be made on the basis of the study.

A study similar to the one just described was reported in Reference 13. For a ten story frame designed by IRSA, the ductility demands were excessive in the top stories, and the conclusion was drawn that designs made using the IRSA, at the stage of development of the IRSA at that time, were not significantly better than those made using the UBC. While no attempt was made in the study reported herein to compare the IRSA designs with UBC designs, the high degree of inconsistency of results from the IRSA points out that further development of the method will be required before it can be considered a useful, practical, and reliable design tool.

TABLE 1

RANGE OF STORY DUCTILITIES

Six Story Frame

Design Ductility Ratio	Story Ductility Ratio	Story Ductility Ratio With Moment Redistribution
2	2.6-5.8	2.6-5.2
4	6.2-9.0	6.1-8.8
6	7.1-10.0	7.6-9.9
8	9.6-13.5	9.1-13.0

Ten Story Frame

Design Ductility Ratio	Story Ductility Ratio	Story Ductility Ratio With Moment Redistribution
2	2.76-7.01	2.78-7.09
4	6.61-17.34	6.52-16.79
6	8.99-23.37	8.63-22.22
8	8.53-17.93	8.51-20.73

TABLE 2

COMPARISON OF ROTATION CAPACITIES - NEGATIVE MOMENT IN BEAMS

Six Story Frame

Design Ductility Ratio	Ratio of Actual to Allowable Rotation Without Redistribution	With Redistribution
2	0-0.54	0-0.69
4	0-0.85	0-0.73
6	0-0.92	0-0.73
8	0-0.82	0-0.70

Ten Story Frame

Design Ductility Ratio	Ratio of Actual to Allowable Rotation Without Redistribution	With Redistribution
2	0-3.47	0-3.45
4	0-1.84	0-0.99
6	0-1.38	0-0.72
8	0-1.29	0-0.72

REFERENCES

1. Powell, G.H. and A. Kanaan, "A General Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures," A computer Program Distributed by NISEE/Computer Applications, April 1973.

2. "Recommended Lateral Force Requirements and Commentary", Seismology Committee of the Structural Engineers Association of California (SEAOC), San Francisco, 1975.

3. Uniform Building Code, 1979 Edition, International Conference of Building Officials, Whittier, California, 1979.

4. Standard Building Code, 1982 Edition, Southern Building Code Congress International, Inc., Birmingham, Alabama, 1982.

5. Lucas, Steven D., Edwin G. Burdette, and David W. Goodpasture, "Application of the Inelastic Response Spectrum Approach to the Design of Reinforced Concrete Frames: Six Story Frame-Phase I NSF", The University of Tennessee, Knoxville, 1981.

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

7. Large, Philip E., Edwin G. Burdette, and David W. Goodpasture, "Procedure for Design and Evaluation of Frame 1, Gravity and Lateral Loads," The University of Tennessee, Knoxville, July 1982.

8. Lucas, Steven D., Edwin G. Burdette, and David W. Goodpasture, "Design and Evaluation of a 10-Story, 3-Bay Reinforced Concrete Frame by the Inelastic Response Spectrum Approach", The University of Tennessee, Knoxville, May 1983.

9. Marsh, M. Lee and Edwin G. Burdette, "Evaluation of Two Reinforced Concrete Frames Designed by the Inelastic Response Spectrum Approach Including Moment Redistribution as Allowed by the ACI 318-77 Design Code," The University of Tennessee, Knoxville, July 1983.

10. Bernal, Prospero Dionisio (Dennis), "Seismic Design of Reinforced Concrete Buildings: An Inelastic Response Spectrum Approach", Ph.D. Dissertation, The University of Tennessee, Knoxville, August 1979.

11. Corley, W.G., "Rotational Capacity of Reinforced Concrete Beams", Journal of the Structural Divison, ASCE, Vol. 92, ST5, October 1966.

12. Mattock, A.H., Discussion of "Rotational Capacity of Reinforced Concrete Beams," by W.G. Corley, Journal of the Structural Divison, ASCE, Vol. 93, ST2, April 1967.

13. Biggs, John M., Wai K. Lau, and Drew Persinko, Seismic Behavior and Design of Buildings, Report No. 1, "Aseismic Design Procedures for Reinforced Concrete Frames," Publication No. R79-21, M.I.T., July 1979.