# AN EVALUATION OF RESPONSE SPECTRA DESIGN PROCEDURES THROUGH INELASTIC ANALYSIS

by

S. A. Anagnostopoulos[I], J. M. Biggs[II]

## SYNOPSIS

The use of modal analysis with inelastic spectra, for design and response prediction, is evaluated using a computer program that integrates numerically the nonlinear equations of motion. Two buildings, a 10-story reinforced concrete and a two-story steel, with moment resisting frames, designed using inelastic spectra, are analyzed for an artificial motion which matches approximately the design spectra. Comparisons are made and some tentative conclusions are finally drawn.

## INTRODUCTION

It has long been recognized that buildings should be designed with sufficient strength to avoid costly damage during a moderate but probable earthquake and also with sufficient ductility to avoid collapse during a major earthquake that will probably occur once in their lifetime.

An attempt to utilize this idea in a possible code format was made by the Applied Technology Council of California and is described in a report submitted to the U.S. National Bureau of Standards (1). A set of design and analysis procedures, described briefly in the next paragraph, were formulated and applied to 11 existing buildings, representing a wide variety of structural types and materials of construction. The major assumption underlying the ATC-2 procedures is the validity of the use of inelastic spectra, as derived by Newmark & Hall (3), to predict inelastic structural response through modal analysis.

This paper contains an evaluation of this approach for 2 of the 11 buildings, by comparing their reported responses to those obtained using a numerical integration of the nonlinear equations of motion.

## TWO LEVEL INELASTIC SPECTRUM DESIGN

The ATC-2 design procedures can be summarized as follows.

1. For the particular area of interest and based on probabilities of earthquake occurrence, select two appropriate Elastic Design Spectra, one for Damage and the other for Collapse.

2. Following the rules in (3), and using appropriate values of damping and ductility factors as suggested in (1), derive inelastic acceleration and displacement spectra. Call the first DTSS (Damage Threshold Spectrum for Strength), and the second CTSD (Collapse Threshold Spectrum for Deformation).

3. Perform a modal analysis of the structure for the DTSS and compute earthquake design forces Q using the SRSS modal responses.

4. Design members for the following combinations of nonfactored loads:

---

I Research Engineer, Shell Development Co., Houston, Texas; formerly Research Associate, Dept. of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts.

II Professor of Civil Engineering, Massachusetts Institute of Technology.

a. D + L + Q
b. 2/3D - Q
c. Other combinations not involving earthquake load as per U.B.C. Ultimate Strength Design according to the ACI code or AISC specifications, with minor modifications, is to be used for concrete or steel respectively.

5. Ductility requirements for moment resisting frames should be computed using the CTSD and approximate formulas suggested in (1).

6. Check stability of the frames due to P-$\delta$ effect at the deformations produced by the CTSD.

## COMPARISON WITH INELASTIC TIME HISTORY ANALYSIS

Two of the buildings, that were redesigned to evaluate the ATC-2 procedures, were analyzed using a computer program called "FRIEDA," that has been developed at MIT (4). FRIEDA (FRame Inelastic Earthquake Dynamic Analysis), as the name suggests, can perform an inelastic dynamic analysis of plane frames by integrating numerically the differential equations of motion. It assumes plastic hinges forming at the member ends when the moment exceeds the capacity and can use either the dual or the single component representation of inelastic member behavior. It can account for gravity loads through actions at the ends of the members and for bending-axial interaction in the columns by appropriate, user-defined interaction diagrams. It also has the capability for modelling haunched beams, rigid zones at the joints, beams with different moment capacities for top and bottom, etc. As output it gives elastic modal shapes and natural periods, maximum inelastic displacements, interstory drifts, shears, overturning moments, member end forces and ductility factors defined in terms of moments and plastic hinge rotation.

The first building shown in Figure 1 (#3 in reference 1) is 10 stories of Reinforced Concrete with moment resisting frames in the longitudinal direction and a combination of such frames and shear walls in the transverse. The two interior frames have haunched beams, while the two exterior have prismatic members. The first three natural periods, shown in Figure 1, were computed using full design loads and uncracked sections following the ATC-2 suggestions. Inelastic analysis, with a time step of 0.005 sec., was performed for the longitudinal direction only. The second building shown in Figure 2 (#10 in reference 1) is a two-story steel structure, almost symmetric, with moment resisting frames on the perimeter. Its first natural period of 1 second is unusually high, resulting in lower earthquake forces. Figure 3 contains the two inelastic spectra DTSS, CTSD, the elastic spectrum corresponding to the CTSD, and the spectrum of the artificial motion that was generated to match the elastic spectrum corresponding to the inelastic CTSD. The inelastic time history analysis was performed using this artificial motion. The two peak ground accelerations of the basic elastic Damage and Collapse spectra were 0.24g and 0.28g respectively. Results and comparisons for building 1 are shown in Figure 4. The ATC-2 modal analysis tends to predict rather well the interstory drifts and total displacement at the top. Because of the elastic nature of the ATC-2 analysis, the predicted response varies more or less uniformly, failing to predict possible changes due to variations in the pattern of yielding. This can be observed from the ductility plots of Figure 4. While the building remains elastic above the 7$^{th}$ floor, the ATC-2 approximation predicts yielding there with ductility factors up to 1.75. Most of the yielding takes place at the girders of the lower floors of the exterior frame, with a maximum ductility factor 2.56 compared to the ATC-2 prediction of 1.40. The interior frames have a maximum ductility factor of 1.1 in the second floor and remain elastic above the 3rd. Again ATC-2 predicts almost uniform yielding over all stories. The columns have remained elastic except at the base of the exterior frames where ductilities up to 2.0 were developed. An undesirable situation occurs in the two exterior columns of the exterior frame, which despite the gravity load they carry, develop tension approximately half their capacity (in tension). This reduces significantly the plastic moment capacity and can lead to large ductility factors. Results and comparisons for the 2-story building are summarized in Table 1. This is a weak-column design with excessively high 1st natural period. Here the ATC procedure underpredicted the top floor displacement by 20%. It also predicts yielding for the second floor where it actually does not occur. The ATC-2 ductility factors in this case were estimated roughly as the ratio of the member forces due to the CTSD and DTSS, which is approximately equal to the ratio of the corresponding spectral accelerations.

## CONCLUSIONS

Before any conclusions are drawn, two points should be emphasized. First the level of the CTSD was quite low in comparison to the DTSS, causing little yielding, not enough to produce the mechanisms assumed in the procedures. Second, the time history analysis was performed with only one motion (because of excessive costs), so the conclusions to follow are only tentative. With these in mind, the following observations can be made.

1. The produced designs seem to be satisfactory for the particular level of motions that were chosen. This might not be the case for more extensive yielding.

2. Use of a modal analysis to predict inelastic response tends to predict uniform yielding conditions and fails to reflect either variation of yielding patterns in a floor or with height, or local concentration of yielding.

3. The approximate formulas to predict local values of ductility factors cannot be applied unless mechanisms are formed. It is possible, however, to have high local values of ductility without reaching the story mechanism level.

4. The exterior columns of reinforced concrete frames should be designed with increased strength, to avoid significant reduction of their plastic moment capacity due to earthquake-induced tensions.

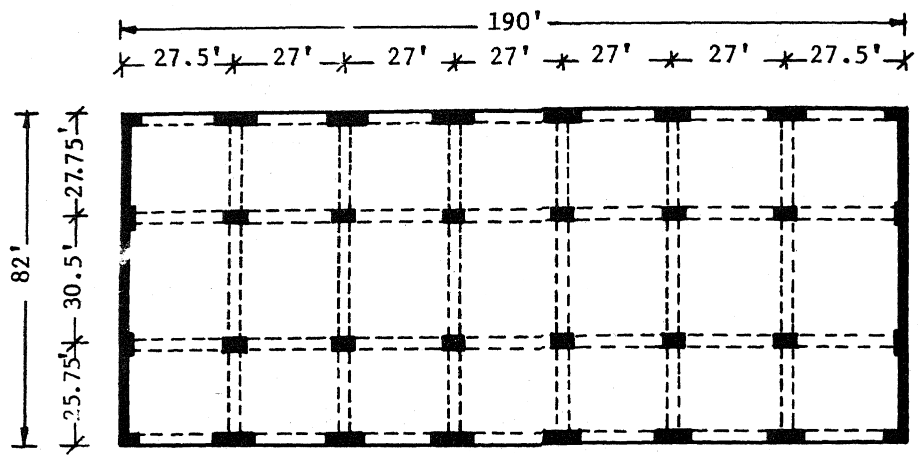
## REFERENCES


1. Applied Technology Council, "An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings," A Study Report for the National Bureau of Standards, Washington D.C., September 1974.

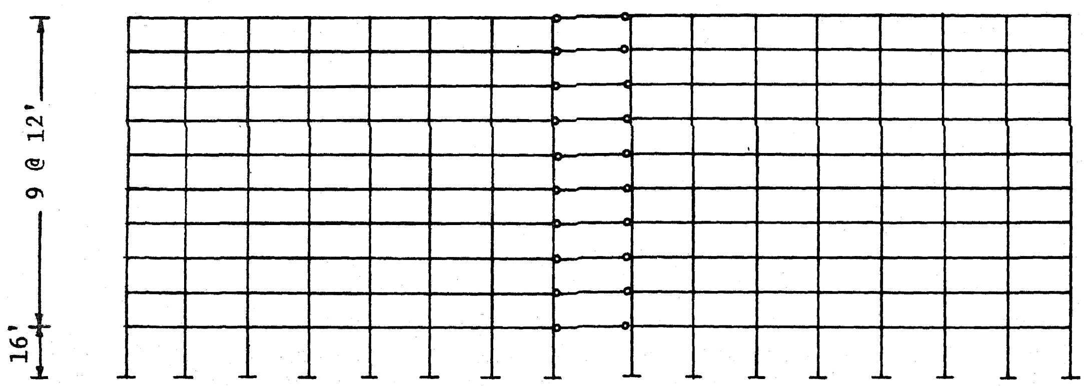
2. Luyties, W.H. III, Anagnostopoulos, S.A., Biggs, J.M., "Studies on the Inelastic Dynamic Analysis and Design of Multistory Frames," M.I.T. Dept. of Civil Engineering Research Report R76-29, July 1976.

3. Newmark, N.M., Hall, W.J., "Procedures and Criteria for Earthquake-Resistant Design," Building Practices for Disaster Mitigation, Building Science Series 4C, National Bureau of Standards, February 1973, pp. 209-237.

4. Luyties, W.H. III, Anagnostopoulos, S.A., Roesset, J.M., "FRIEDA - A Computer Program for Inelastic Dynamic Analysis of Frames," to be printed as an MIT Research Report.

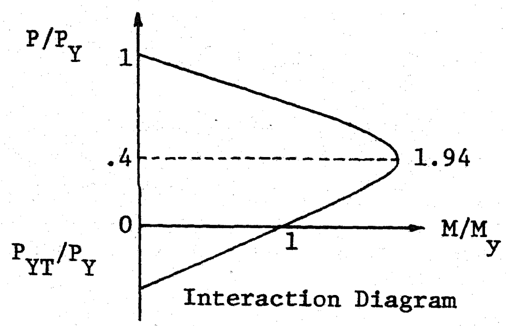



PLAN VIEW - TYPICAL FLOOR




EXTERIOR FRAME

INTERIOR FRAME

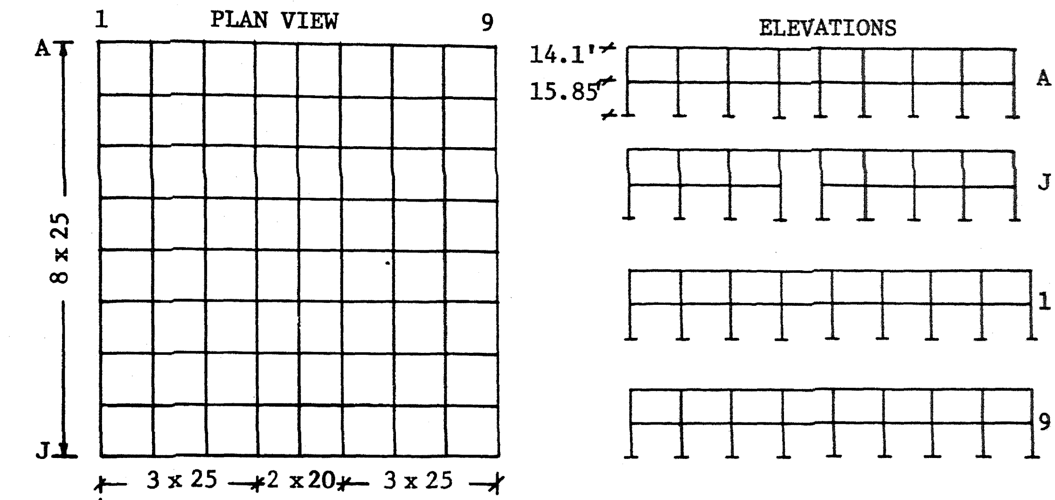



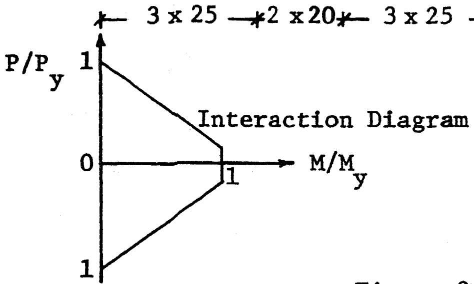

Interaction Diagram

| MODE | NATURAL PERIODS CASE 1 | CASE 2 | ATC |
|---|---|---|---|
| 1 | .66 | .72 | .74 |
| 2 | .21 | .24 | .24 |
| 3 | .12 | .14 | .13 |

CASE 1 - With Girder Depth
CASE 2 - Without Girder Depth

Figure 1 - 10-Story Reinforced Concrete Building




| MODE | NATURAL PERIODS | | | |
|---|---|---|---|---|
| | FRIEDA | | ATC | |
| | A,J | 1,9 | A,J | 1,9 |
| 1 | .99 | 1.02 | .97 | 1.00 |
| 2 | .36 | .37 | .35 | .35 |

Figure 2 - 2-Story Steel Building

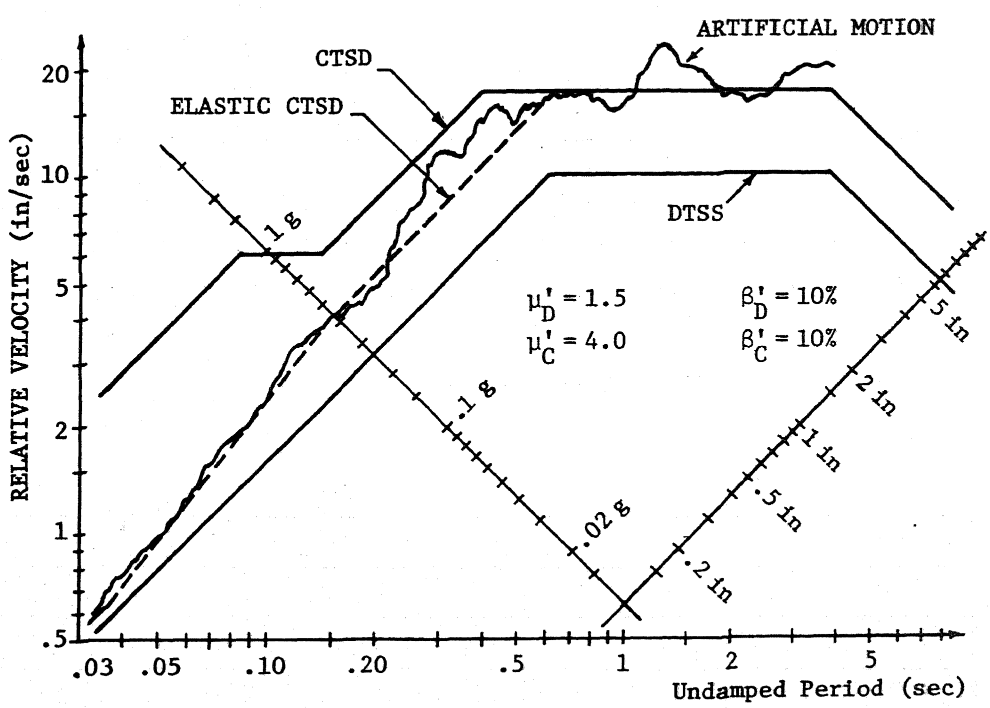
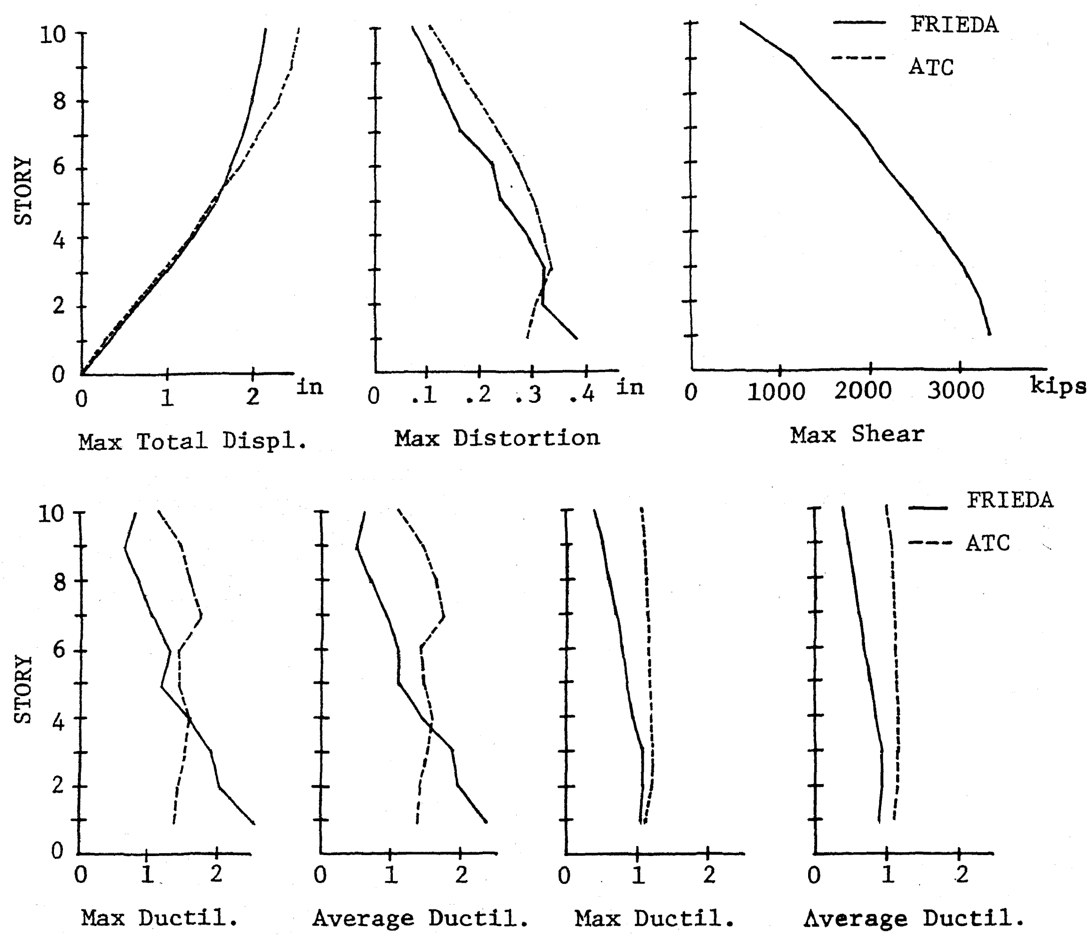



Figure 3 - Response Spectra




EXTERIOR FRAME GIRDERS INTERIOR FRAME GIRDERS

COLUMNS: Elastic everywhere except bottom of 1$^{st}$ Story
Where Max. Ductility = 1.4 ÷ 2.0

Figure 4 - Results for 10-Story R.C. Building

| STORY | DISPLAC. | | DISTORT. | | SHEAR | | COL. DUCTILITY | | | GIR. DUCTILITY | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | INEL. | ATC | INEL. | ATC | INEL. | ATC | INELASTIC | | ATC | INELASTIC | | ATC |
| 1 | 2.70 | 2.20 | 2.70 | 2.20 | 1106 | 1460 | 2.26* | 2.00** | 1.75 | 1.76* | 1.40** | 1.60 |
| 2 | 4.00 | 3.22 | 1.30 | 1.17 | 824 | 660 | NO YIELD | | 1.75 | NO YIELD | | 1.55 |

*Definition by plastic hinge rotation
**Definition by moment (curvature)

Table 1 - Results for 2-Story Steel Building

## DISCUSSION

### B.R. Seth (India)

By the method adopted for design the failure may occur by collapse of the building by the fatigue failure of the column at one section only as a very few cycles of reversal at yield lead to failure, instead by instability including P-$\delta$ effect.

### Author's Closure

Not received.