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APPROXIMATE INELASTIC PROCEDURES TO IDENTIFY FAILURE MECHANISMS FROM HIGHER MODE EFFECTS

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ABSTRACT

Two 17-story steel frame buildings were evaluated using approximate inelastic procedures which identify failure mechanisms caused by higher mode effects. The inelastic analysis procedures used were the Capacity Spectrum Method (CSM) and the modal pushover procedure. The results of the evaluations indicate that the failure mechanisms for these buildings for the studied earthquakes are primarily caused by the effects of higher modes. A Modal Criticality Index (MCI) was used to identify the vibration mode most likely to cause building failure.

KEYWORDS

Approximate inelastic procedure, Capacity Spectrum Method (CSM), modal pushover, steel frame building, higher mode effects, Modal Criticality Index (MCI), Acceleration-Displacement Response Spectrum (ADRS)

INTRODUCTION

Despite the numerous dramatic story-collapses in the mid or upper reaches of multistory buildings induced by earthquake ground motions in Mexico City in 1985 and in Kobe in 1995, realistic methodologies that would enable engineering design practitioners to anticipate, evaluate, and protect against localized story collapses due to higher mode building excitation are noticeably absent from codes and from evaluation handbooks.

Code-based static design methodologies all derive essentially from single-degree of freedom analytical methods and utilize a fudge factor (F_t) to simulate higher mode effects on the upper floors of a building. While the reasonableness of F_t with respect to higher-mode induced story shears in the upper stories of a long period structure can itself be debated, it is especially important to recognize that F_t necessarily results in an amplification of overturning moment in the lower stories of a building that is not consistent with higher mode excitation. If anything, higher mode excitation would tend to diminish, not amplify, the base-overturning moment resulting from first mode response. As a result, code-based static design methodologies for long-period structures tend to generate designs with relatively "over-strong" lower stories which, during a major earthquake, might be expected to promote the occurrence of localized story mechanisms on higher floors.

Commonly employed code-stipulated dynamic design methodologies represent some improvement over the static approach, but still miss the boat when it comes to clearly and accurately characterizing higher mode effects. For one thing, since the shape of the code-response spectrum often varies significantly from the shape of the site-specific design response spectrum postulated for a particular site, the relative magnitude of higher-mode-driven story forces in the upper floors of a long period structure compared to the first-mode-driven story forces may be vastly under-estimated or over-estimated. The relative magnitude of modal spectral ordinates is more important than absolute magnitude since for a postulated major earthquake the former will control the sequencing of plastic hinges and mechanism formation, not the latter.

It is also noted that even site-specific response spectrum analysis may not provide an accurate characterization of higher mode response. Because site-specific spectra typically represent an envelope of a family of different earthquakes occurring along different fault lines rather than one particular earthquake, these spectra will also tend to obscure the relative magnitude of higher mode versus lower mode response of a particular building. Designs generated by these composite spectra will always tend to have one mode "over-designed" with respect to another because any single earthquake in the family would not excite all modes to the degree predicted by the composite spectrum. Again, a concentration of plastic hinging culminating in the formation of a story mechanism consistent with the response of a particular mode is the likely result. Furthermore, because different modes may be excited during different time intervals in any particular earthquake and even site-specific response spectra cannot capture this time dependent effect, it is quite conceivable that a long-period structure will experience inelastic excursions due to higher modes before its first mode even gets going. Such behavior can dramatically alter anticipated global building response by concentrating energy-dissipating actions within two or three stories, thereby decreasing the likelihood of well-distributed plastic hinging. It may very well be inappropriate, therefore, to design long-period structures using an $R_w=12$ in anticipation of widespread hinging when standard code design techniques do little to promote distributed hinge sequencing and may actually have the opposite effect. Simply put, even the best code-based design procedures and commonly employed methods fall short by blurring the importance of and the distinction between the individual modal responses of a particular building. To wit, many standard analytical software programs include subroutines for modal combination that cannot be easily over-ridden via menu-driven options, which makes the task of studying the behavior of individual modes cumbersome.

BUILDING DESCRIPTIONS

Two sample buildings (see Fig. 1 and Fig. 2) located in different geographic regions and of different ages and construction were studied. The first building is a 17-story office building, constructed in the early 1960's, and located in central California. It is 231 ft tall, with plan dimensions of 130 ft by 300 ft. The building geometry does not contain any significant structural irregularities. The typical story heights are 13'-4" with a 14 ft ground floor height. The lateral force resisting system consists of steel moment resisting frames in both principle directions. There are two 15-bay frames in the longitudinal direction and six 5-bay frames in the transverse direction. The columns are 14-inch wide flange sections ranging in size from W14x48 to W14x370. At the frame intersections the columns are orientated with the strong axis in the transverse direction. The "beams" for all frames consist of 44-inch deep, open-web, double angle steel trusses. The 20 ft longitudinal truss "girders" support the floor framing steel joists at the 3rd points. The 26 ft transverse truss "beams" are parallel to the floor joists and have nominal gravity loading. At the column connections the truss members are welded to 5/8-inch full depth gusset plates continuously welded to the column. All steel is grade A36. The building is supported on a pile foundation with interconnecting concrete grade beams. The deep trusses have a higher moment capacity than the columns and therefore this building is referred to as the strong beam building.

The second building is also a 17-story office building but was constructed in the mid 1980's and is located in southern California. This building was previously studied for SAC Task 3.1 (Paret, et al., 1995) and ATC33 Task 12 (Freeman, et al., 1995). The building is 248 ft tall with plan dimensions of 120 ft by 158 ft. The typical story heights are 13 ft with a 20 ft ground floor and one subgrade level. There are no significant structural irregularities. The gravity system consists of steel wide flange beams and columns supporting a lightweight concrete and metal deck floor system. The lateral system consists of two 2-bay steel moment resisting frames in each principle direction. The transverse moment frames have 28 ft bays, and the frames step out one bay below the 3rd floor. The longitudinal frames have 31 ft bays. The north longitudinal frame is set-in one bay from the exterior. The moment frame beams are A36 steel wide flange sections ranging in size from W30x99 to W36x300. The columns are A572 grade 50 steel, ranging in size from W14x300 to W14x730. The beam

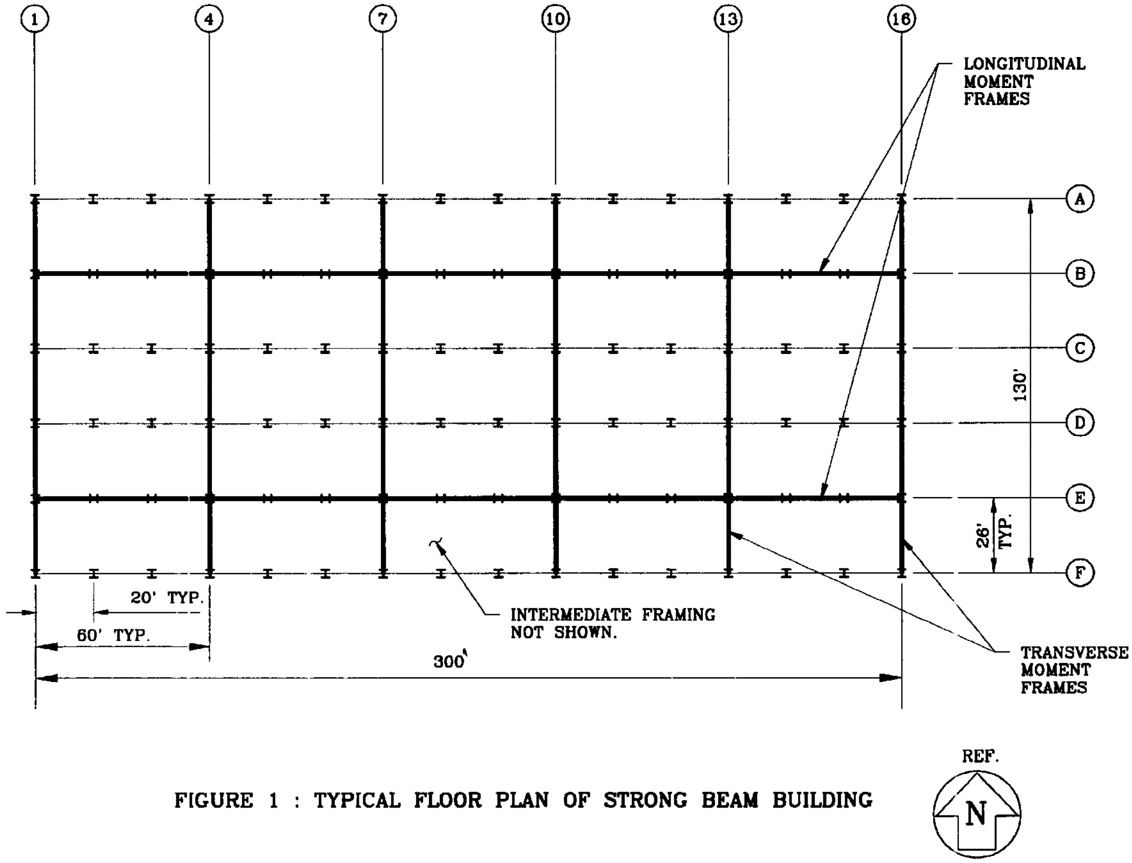


FIGURE 1 : TYPICAL FLOOR PLAN OF STRONG BEAM BUILDING

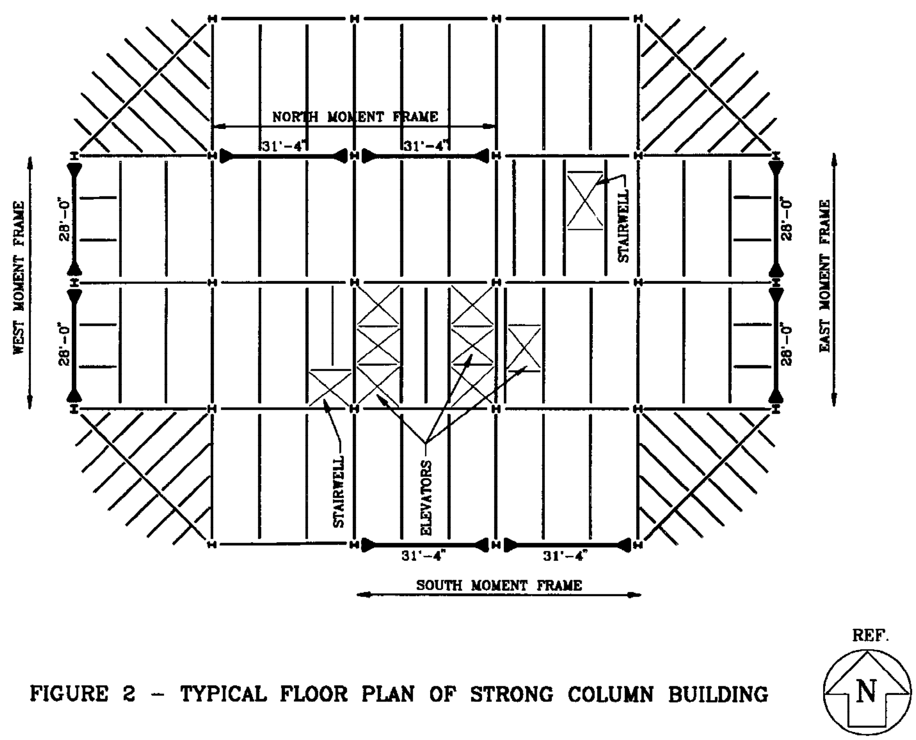


FIGURE 2 – TYPICAL FLOOR PLAN OF STRONG COLUMN BUILDING

connections consist of full penetration field welds at the top and bottom flanges, and a single shear plate web connection. The building is supported by concrete pile caps bearing on precast, prestressed piles. The moment capacity of the columns exceeds the beam moment and therefore this building is referred to as the strong column building.

CAPACITY SPECTRUM METHOD

The Capacity Spectrum Method (CSM) (Freeman 1994) using the modal pushover procedure and the Acceleration-Displacement Response Spectrum format (ADRS) (Mahaney et. al. 1993) was used to evaluate the response of these buildings to different earthquake ground motions. CSM is a method which allows a graphical comparison of building capacity to the earthquake demand. The lateral force resisting capacity of the building is represented by force-displacement curves obtained from pushover analyses. The demands of the earthquake are represented by a response spectrum curve. When both sets of curves are plotted on one graph using the same set of coordinates (e.g., S_a v. S_d as in the ADRS format), the relationship between capacity and demand become readily apparent. The intersection of the capacity (pushover) curves with the demand (response spectrum) curve approximates the response and performance of the structure for that particular earthquake.

The capacity curves are determined by statically loading the structure with lateral forces to calculate base shear values and roof displacements that define the global force-displacement characteristics of the structure. The procedure to develop these curves has been referred to as the pushover analysis and is generally applied to 1st modes only. The base shear values and roof displacements are converted to S_a and S_d values, respectively, by use of effective modal weights and modal participation factors as determined from the dynamic characteristics of the structure. These values of S_a and S_d are plotted in the ADRS format.

The demand curves are generally represented by earthquake response spectra in the S_a v. T format. The response spectra are presented at various levels of damping. For example, the 5 percent damped curve may be used to represent the demand when the structure is responding elastically. The 10 percent and 20 percent damped curves may be used to represent the reduced demand in the inelastic range to account for hysteretic damping and nonlinear effects. The T axis is converted to S_d to form response spectra in the ADRS format. In this format, the periods are represented by radial lines.

The Capacity Spectrum Method can be summarized as follows:

> *If the capacity curve can extend through the envelope of the demand curve, the building survives the earthquake. The intersection of the capacity and demand curve represents the force and displacement of the structure for that earthquake.*

Modal Pushover Procedure

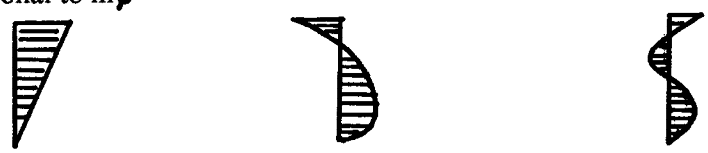
The modal pushover procedure involves performing several pushover analyses, each using a lateral load pattern based on a different elastic mode shape. The purpose of the modal pushover procedure is to generate capacity curves that represent the response of the building for particular modes of vibration. The CSM using the modal pushover procedure is outlined below.

1. Develop a model of the building.
2. Determine the mode shapes and periods of the building
3. Determine the lateral load patterns for the modes of interest.
4. Perform pushover analyses for the modes of interest.
5. Create capacity curves for each pushover analyses by plotting base shear versus roof displacement.
6. Convert capacity curves into ADRS (S_a v. S_d) format.
7. Plot response spectra into ADRS (S_a v. S_d) format.
8. Compare capacity and demand curves.

For the two buildings analyzed, pushover analyses were performed for the first three translation modes of each building in the direction of interest. The lateral load pattern for the first mode is determined by multiplying the first mode shape by the weight at each floor. The lateral load pattern for the second mode is determined by multiplying the second mode shape by the weight at each floor. Similar calculations are made for the third mode. The diagrams in Fig. 3 schematically illustrate the CSM using the modal pushover procedure for these buildings. Note that the force-displacement plot, V v. Δ_R, masks the significance of the higher modes.

Determine Lateral Load Patterns for Modal Pushover Analyses

Shapes are proportional to mϕ



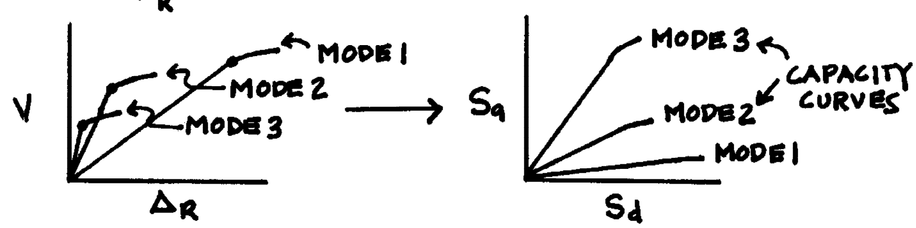
1st Mode   2nd Mode   3rd Mode

Determine Capacity (Pushover) Curves for Each Mode

For each load pattern, conduct a pushover analysis.

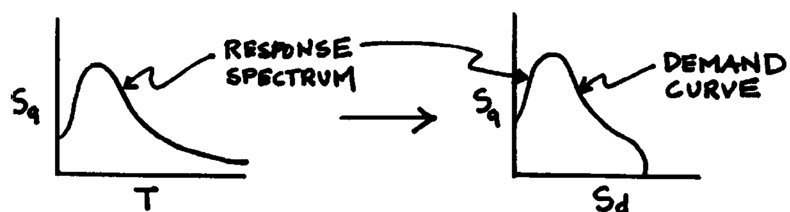
Plot capacity curves as baseshear (V) v. roof displacement (Δ_R)

Convert V v. Δ_R to Sa v. Sd



Determine Demand Curve

Plot response spectrum in ADRS format (Sa v. Sd)



Compare Capacity v. Demand

Overlay capacity curves on demand curve on Sa v. Sd plot

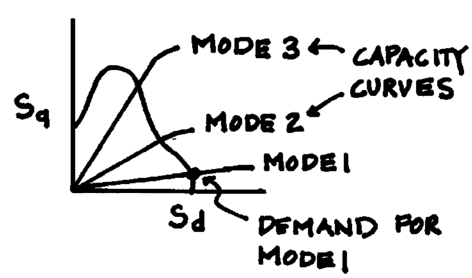


Fig. 3 CSM Using the Modal Pushover Procedure

COMPARISON OF CAPACITIES V. DEMANDS USING ADRS FORMAT

The capacity and demand curves for the strong beam building and strong column building were plotted in the ADRS format. Figure 4 shows the ADRS plot for the strong beam building. Capacity curves were generated individually for mode 1, mode 2, mode 3, and neglect mode combination effects. Orthogonal effects on columns at frame intersections were evaluated separately and are also neglected in these curves. The pushover analyses for the strong beam building used an elastic model that was modified to account for member hinging. The demand curves shown are the site-specific response spectra that have return periods of 475 years and 2000 years. From this plot, it is immediately apparent that all of the modal capacity curves escape both response spectra. Not only do the capacity curves escape, but they appear to escape while the structure still remains elastic. For instance for mode 1, the demand from the 2000-year earthquake (which occurs at the intersection of the mode 1 capacity curve with the 2000-year response spectra curve) is much lower than the demand needed to significantly yield the structure. With consideration of orthogonal effects and modal superposition, the capacity curves would indicate yield at a significantly lower S_a. It is apparent from this plot that the response of mode 2 is critical for this building, i.e., significant yielding of this structure would most likely occur due to the predominant response of the second mode. The building can be characterized as Mode 2 Critical because the earthquake demand is much closer to the mode 2 yield capacity than to the mode 1 or the mode 3 yield capacities. Another way to look at this is that if the structure was designed or evaluated using only a 1st mode analysis, the failure of the structure caused by 2nd mode response would be completely overlooked. Analysis of demand/capacity ratios for each mode verify the significance of the second mode.

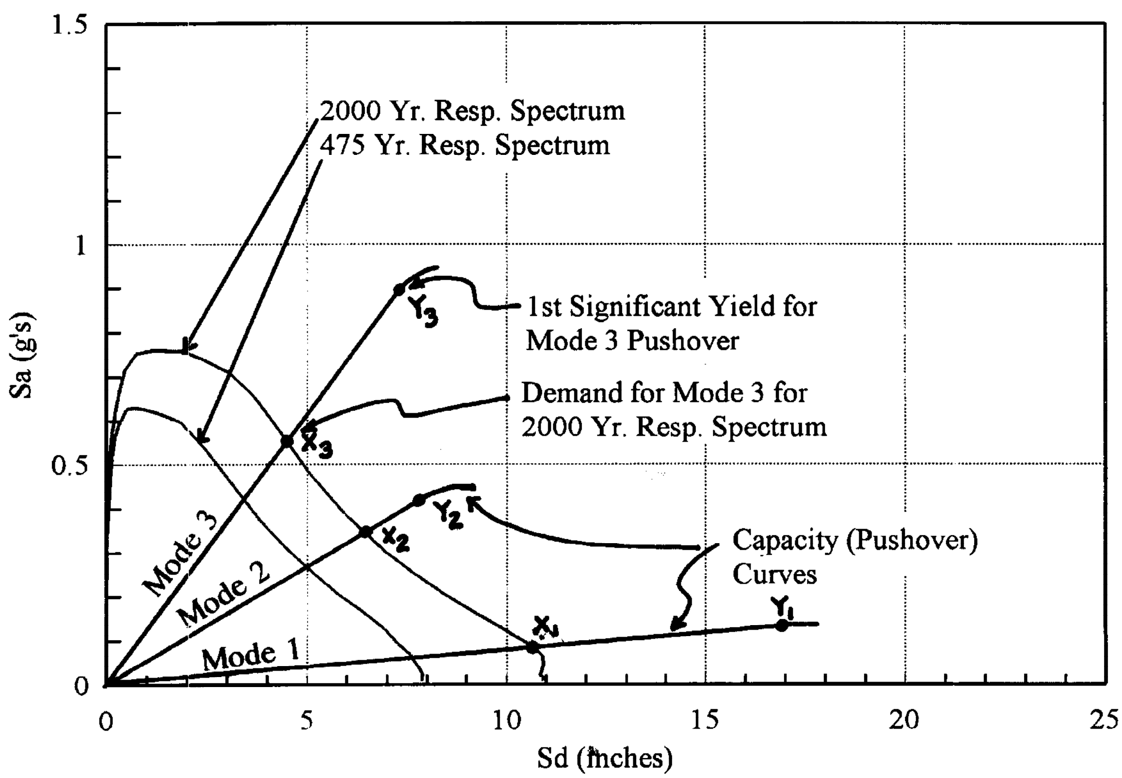


Fig. 4 Capacity and Demand Curves for the Strong Beam Building - ADRS Format

One method to determine the critical mode is to divide the earthquake demand by the yield capacity of the building for each mode. In Fig. 4, earthquake demands for each mode are identified by "X_n" where the subscript denotes a particular mode. The yield capacities for each mode are identified by "Y_n" where the subscript denotes a particular mode. The S_a values associated with each of these locations can be found. For a particular mode, dividing the S_a value for demand by the S_a value for capacity gives what will be termed as the Modal Criticality Index (MCI). For example, considering the 2000-year spectrum, the MCI for mode 1 is equal to 0.6. Similarly, the MCI for mode 2 is equal to 0.9, and the MCI for mode 3 is equal to 0.6. The largest MCI value identifies the critical mode. For this building, mode 2 with a MCI of 0.9 is clearly the most critical mode.

A similar study was performed on the strong column building. Figure 5 shows the capacity and demand curves for the strong column building plotted in the ADRS format. The capacity curves were generated individually for mode 1, mode 2, and mode 3. To perform the pushover analyses for the strong column building, a two-dimensional inelastic model was developed using the computer program DRAIN2D+ (Tsai and Li 1994). The demand curve shown is a 5% damped response spectrum developed from earthquake ground motions recorded near the building site. Unlike the results of strong beam building described above, not all capacity curves escape the demand curve while the structure remains elastic. For the mode 2 pushover, this earthquake causes the structure to experience significant yielding. The yielding of the structure for mode 2 is concentrated in the upper floor beams of the building. Note that this earthquake causes no significant yielding of the structure in either mode 1 or mode 3. Again, MCI values can be computed. The MCI values were 0.9, 1.3, and 0.6 for modes 1, 2, and 3, respectively. Mode 2 is critical with a MCI value of 1.3. From these values alone, the response of a building to a particular earthquake can be quickly determined. The strong column building is close to yielding for mode 1, is yielding in mode 2, and is far from yielding in mode 3.

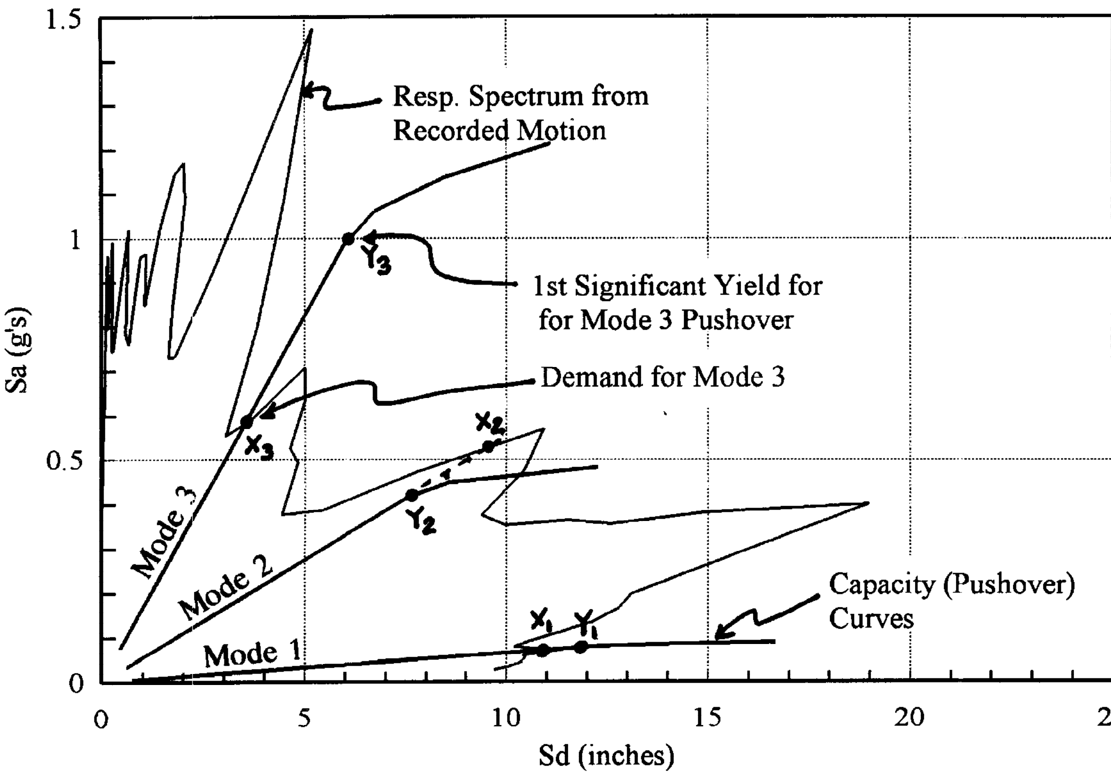


Fig. 5 Capacity and Demand Curves for the Strong Column Building - ADRS Format

CONCLUSIONS

The following conclusions were reached from this study:

- Code design methodologies do not adequately address, and appear to exacerbate, the potential for higher mode induced story collapses in long-period structures.
- Modal Criticality Indices (MCI) can be computed using modal pushover procedures or approximated using elastic modal analysis of individual modes.
- $V - \Delta_R$ plots of modal pushover curves mask the significance of modal criticality. Modal criticality is best expressed in the ADRS format.
- Post-earthquake evaluation procedures as well as seismic evaluation procedures in general must account for the shape of the site-specific response spectrum to correctly characterize the behavior of higher modes, which can vary from earthquake to earthquake.

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