SIMPLIFIED METHODS FOR EVALUATION OF REHABILITATED BUILDINGS

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ABSTRACT

Several approximate and simple methods of analysis are described which consider the nonlinear response of structures. These are being considered for guidelines for the retrofit of existing buildings but they might also be appropriate for codes for the design of new buildings. These methods are based on the static nonlinear analysis of the building. Several examples are given.

INTRODUCTION

Current seismic codes account for the reduction of response as a result of nonlinear action in a very approximate way. Response reduction factors (R in the USA, Q in Europe) are used to reduce the design force levels for various structural systems based on nonlinearity, excess capacity, and observed performance in past earthquakes. Although this approach is simple and works reasonable well for regular and uniform structures, it cannot account for irregularities which change the force distribution as nonlinearities develop.

Increased attention is being paid worldwide to the rehabilitation (retrofit) of existing structures because most buildings have not been properly designed for earthquake resistance and most of the risk to society comes from the deficiencies in the existing building inventory. The retrofit of existing buildings is much more complicated and more expensive than the seismic design of new buildings. Therefore, evolving guidelines for the rehabilitation of existing buildings, such as the FEMA project (1), are considering more realistic analysis methods. The additional engineering work required by these methods is justified by the much higher cost and greater complexity of the problem. However, it is expected that these methods will eventually be incorporated also into codes for the design of new buildings.

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Another reason for the need for more realistic analysis methods is a growing trend toward performance-based design. Current codes are based largely on the prevention of collapse for the design earthquake. However, owners, society, and municipalities may expect better performance in many cases. For example, for certain important facilities we may want to limit damage, avoid loss of life even if there is no collapse, or even try to assure continued operation of the facility. The design for these performance levels may depend on the seismic hazard. Current analysis methods are insufficient because they cannot realistically predict the forces and deformations in irregular structures, especially in systems where nonlinearities do not develop uniformly throughout the structure. Improved analysis methods should reflect the change in inertial force and element deformation distributions as nonlinearities occur.

IMPROVED ELASTIC ANALYSIS (IDR) METHOD

A significant improvement over the use of a single R (or Q) factor is afforded by the Inelastic Demand Ratio (IDR) approach. This procedure continues to rely on linear analysis and, therefore, does not account for the inelastic redistribution of forces and deformations. A regular lateral force analysis (or modal analysis) is performed, as in current codes, without the R factor. The elastic member forces are divided by the member capacities at the same member deformation (end rotation) levels and the resulting Inelastic Demand Ratios are compared to limiting values for various member types. Typical values are (11): Ductile steel and concrete beams 3.0, steel beams in braced frames 2.0, K-braces and connections 1.25, flexure of concrete walls 2.0 (single steel layer) or 3.0 (two layers). If the limiting values are exceeded, redesign is necessary or a nonlinear analysis approach must be used to more carefully examine demands.

STATIC NONLINEAR ANALYSIS METHODS

The various simple nonlinear analysis methods being considered for code use rely on static nonlinear analyses because nonlinear time-history analysis is too complex and unreliable for use in design offices, except by a few experienced engineers. On the other hand, the static nonlinear analysis of structures is relatively simple.

The Pushover Curve

The plot of the total lateral force (base shear) versus the deformation (usually the roof displacement) is called the "pushover curve". This analysis is relatively fool-proof since most engineers can easily estimate the story shear capacities and thus get an idea of the peak load level. It is likely that computer codes will be improved to include such static "pushover" analysis automatically. In the meantime, an easy way to perform pushover analysis is to reduce the stiffnesses of members whenever the elastic limit is reached and additional load is applied to the structure with the new stiffnesses. Often it is not necessary to modify the stiffness for every hinge formation, but only after several new hinges have formed. The forces and displacements are calculated incrementally. A typical pushover analysis result is shown in Fig. 1. The pushover analysis curves tend to be nearly elasto-plastic (bilinear) for frame structures because a story mechanism develops soon after the first flexural hinge forms. On the other hand, the force increases to much higher levels for structural systems with walls.

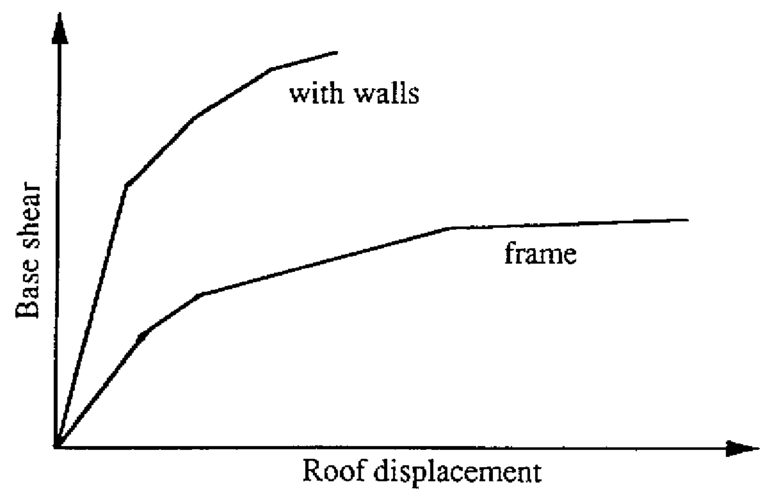

Fig. 1 Typical Pushover Curves

The pushover curve reveals significant information about the characteristics of the structure and is, therefore, important even if no further use is made of it. It shows the progress of damage in the structure, the concentration of hinges, and indicates whether there is a soft story (story mechanism developing before significant hinging occurs elsewhere in the structure). It also helps trace the load transfer among the various load-resisting structural systems in the nonlinear range. This is important because many failures have occurred in earthquakes as a result of poor load transfer.

In the simplest approach, the pushover analysis is performed for an elastic force distribution, for example for the code forces which are functions of the floor masses and the heights. Alternatively, the lateral force distribution is obtained from a modal analysis, though the combination of modal forces at each floor level cannot be done in a rational manner. The forces by either approach are still elastic and do not reflect the stiffness changes. For example, the force distribution for a building with a soft story is significantly different from that of a regular structure.

The reduced (tangent) member stiffness may be too low since the actual member response is cyclic. To account for the higher effective member stiffness, one may assume a secant stiffness for members with nonlinearities, corresponding to an assumed member deformation level (Fig. 2), and perform the next step on the stepwise force analysis. If the resulting member deformation is different from the assumed one, another secant stiffness is assumed. This iterative analysis converges rapidly.

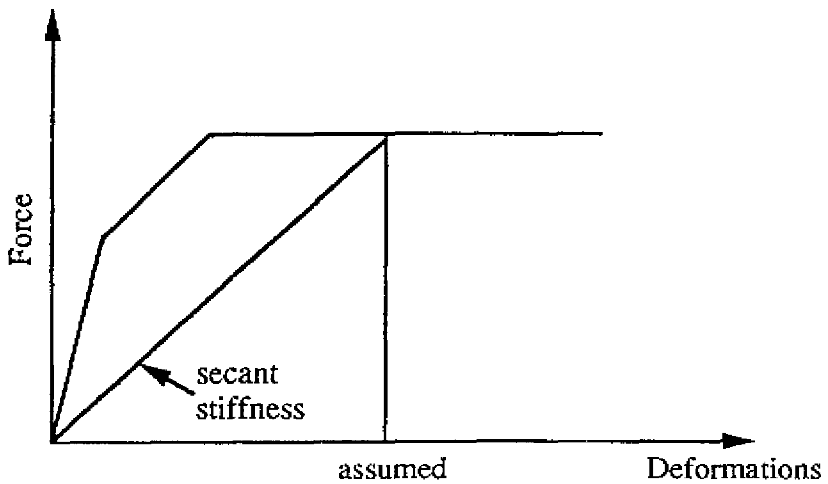

Fig. 2 Secant Member Stiffness for Iterative Approach

For irregular structures it may be necessary to improve the force distribution in the pushover analysis. One way is to generate a load vector as the product of the current displacements and the masses. The new displacements from this load application is an improved inertia force vector shape. If used iteratively, it would converge to the first mode shape for the current stiffness matrix and we could also get the natural frequency from the Rayleigh quotient. Again, this analysis can easily be performed by an ordinary static analysis program.

The pushover curve is a property of the structure; thus we need to estimate the seismic demand. Several approximate ways have been proposed for code use.

The Capacity Spectrum Method

An appealing way of estimating the demand on a system described by its pushover curve is the Capacity Spectrum (10). The base shear and the roof displacement values at each point on the pushover curve are transformed into spectral acceleration and spectral displacement values, usually for the first mode, using the basic equations of modal analysis. The resulting curve is the Capacity Spectrum, which is superposed on the response spectrum curve of the design earthquake, as shown in Fig. 3 for a ten-story steel frame with five bays, one of which has infills. The intersection of the capacity spectrum curve and the response spectrum curve for a selected value of damping gives an estimated response from which the damage state and displacements can be readily calculated.

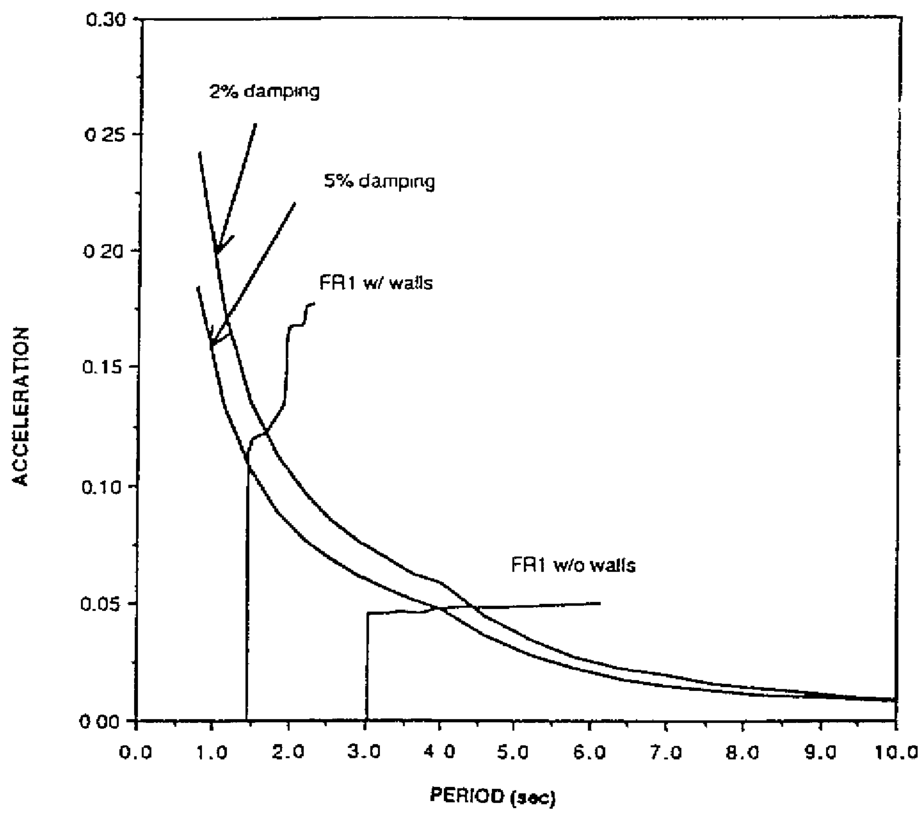

Fig. 3 The Capacity Spectrum Method for a Ten-story Infilled Steel Frame

It is necessary to employ an interpolation scheme between the spectral curves for various amounts of damping. The reduction in response caused by period lengthening is included in the method. However, the hysteretic energy absorption is not. Although it is not possible to replace ductility effects by equivalent viscous damping for the entire frequency range, using an increased damping approximately accounts for the hysteretic effects; this may be inaccurate for short-period structures. As the intersection is further to the right on the capacity spectrum curve, higher damping is appropriate. Relationships have been suggested by Iwan (8) and others for estimating the effective viscous damping as a function of global ductility. On the other hand, the lower spectral curves are for higher damping. Thus the interpolation needs to balance this opposing trends on the curve.

An interesting and more revealing way to plot response spectra is to plot the spectral acceleration S_a versus the spectral displacement S_d (rather than versus the period). Such a plot is indicated in Fig. 4. Constant period lines are rays from the origin. This plot shows the relationship of acceleration and displacement and indicates the effects of changing period. A hypothetical capacity spectrum curve is shown in Fig. 5, together with an interpolation method (4) in which the elastic response (initial period and low level of damping) is identified as point A on a code-type design spectrum. The collapse stage (point B) defines the corresponding period line, which intersects the spectrum with high damping at point C. The line connecting points A and C intersects the capacity spectrum at point D, which is defines the actual response.

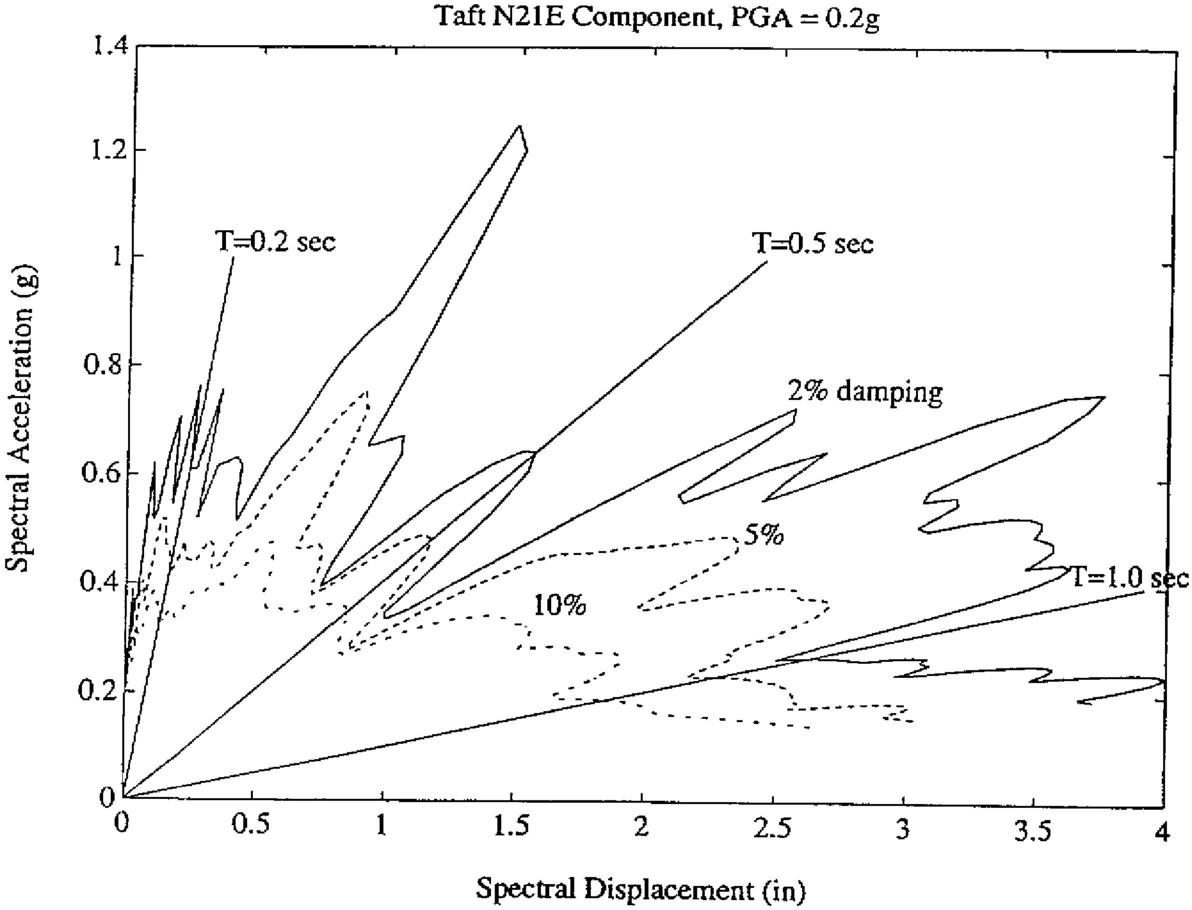

Fig. 4 Response Spectrum Curve Plotted in a New Way

INFILLED FRAMES

A relatively simple and a sophisticated analysis method has been tried to predict the static and dynamic response of infilled frames. Many buildings worldwide consist of light steel or concrete frames that were designed only for gravity loads and unreinforced concrete infills. The response of such structures is being studied analytically and experimentally at Cornell University and elsewhere.

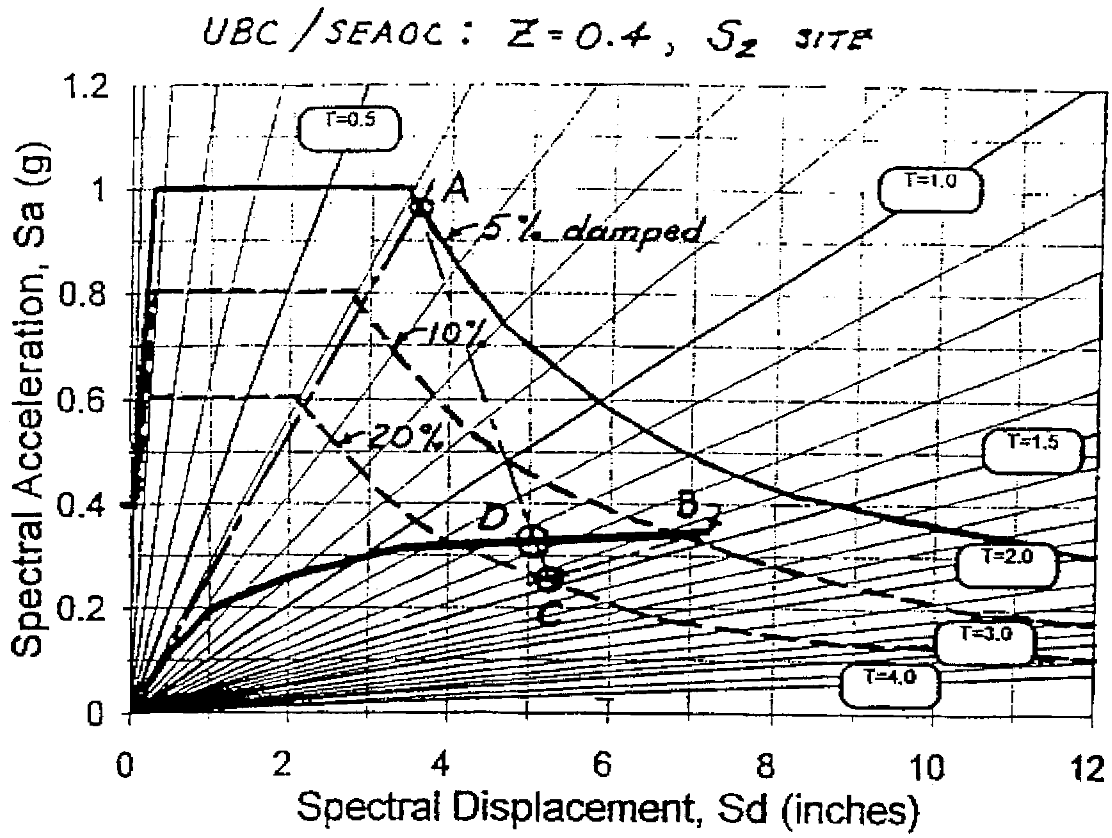

Fig. 5 Capacity Spectrum Method Interpolation

In the simpler model the infill is modelled by three parallel struts in each direction (2): the central strut is diagonal, the other two intersect the beams or columns at a distance from the joint. The three-strut model is able to account for hinges forming in the beams or columns after the infill crushes at the corners and thus the central strut loses its stiffness. The model allows efficient dynamic analysis of multistory infilled frames and gives a reasonably accurate picture of the overall response. The three-strut model was used in calculating the pushover curve in Fig. 3.

The detailed analysis (5) used the finite element program DIANA (3) which can model cracking, plastification, initial gaps, and sliding. Bilinear relationships were used for normal and tangential forces at the wall-frame interface. Good agreement with experiments was obtained for the load-displacement relationship (Fig. 6). Current research concentrates on infills with window and door openings.

EXAMPLE APPLICATIONS

The use of these simple inelastic evaluations methods can permit rapid, and more importantly, realistic evaluations of the demands and capacities of complex structures. To illustrate this, an example analysis of a ten-story steel frame building with infill unreinforced brick masonry is presented. The building, located in San Francisco, California, has a rectangular footprint, with side dimensions 37 by 45 meters. Infill wall panels are typically 43 cm thick, and have regularly distributed penetrations for windows on all sides. Fig. 7 shows a partial elevation of a typical wall. Stories are 3.8 meters tall and bays 4.1 meters wide. Bays are penetrated by pairs of centrally located window openings, 2.1 meters high by 1.1 meters wide.

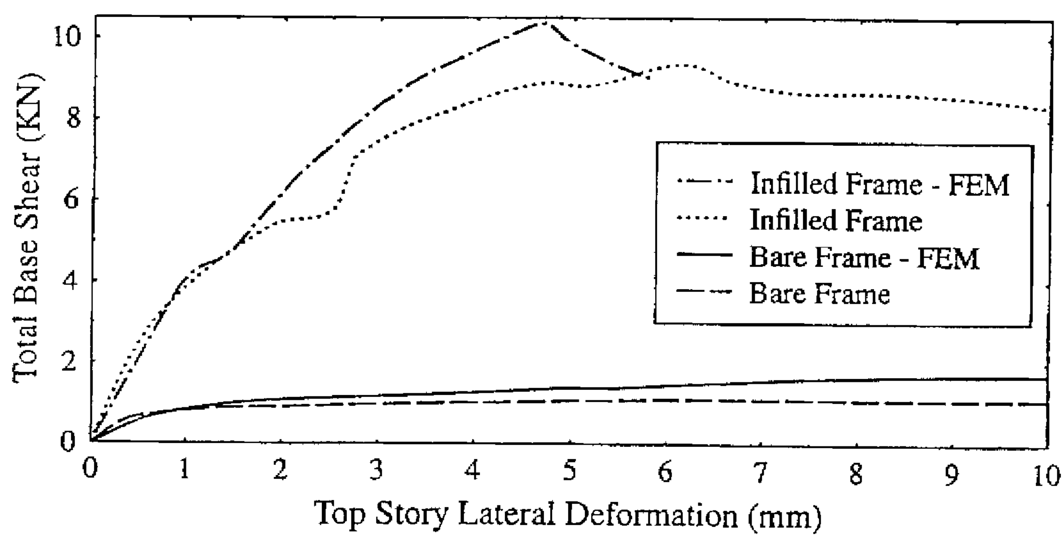

Fig. 6 Comparison of DIANA Analysis of an Infilled Frame with Experiment

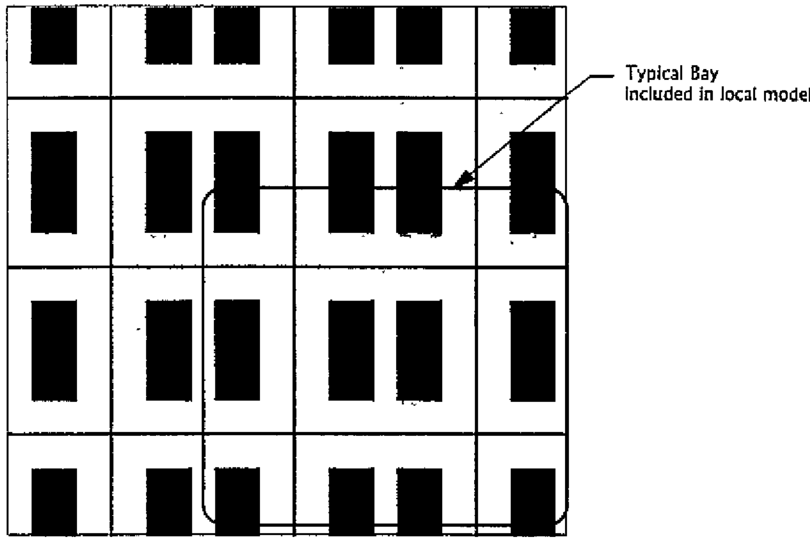

Fig. 7 Partial Elevation of Infill Masonry Wall

At low levels of lateral loading, masonry walls infilled within frame structures behave as shear diaphragms. However, at relatively moderate load levels, principal tensile stresses induced in the masonry by shearing forces result in cracking of the masonry. Following such behavior, the masonry tends to behave primarily as a compressive material to resist lateral deformations induced in the frame. A number of models have been proposed by researchers for physical representation of these phenomena. Solid infilled panels are commonly represented by a series of diagonal struts which span between opposing corners of the frame panels. Following initiation of cracking in such panels, secondary failure modes are typified by compressive crushing of the end bearing zones of masonry against the frame, followed by either plastic hinging or shear failure of frame elements. A three-strut model (2) has recently been demonstrated to be capable of capturing these progressive failure modes.

Alternative models have been proposed for infill panels with large penetrations, such as those in the subject building. Finite element modeling (6) indicates that such infill panels tend to from a system of skewed fields, tangent to the openings in the panels, similar to those indicated in Fig. 8. For masonry of average strength, failure sequences for these compressive fields, classically described as diagonal shear cracking of the piers and spandrels. Following initiation of such cracking, a series of plastic hinges form in the framing elements, until a complete mechanism can form. The ultimate strength of such panels tends to substantially exceed the first formation of shear cracking and occurs at much larger lateral deformation levels.

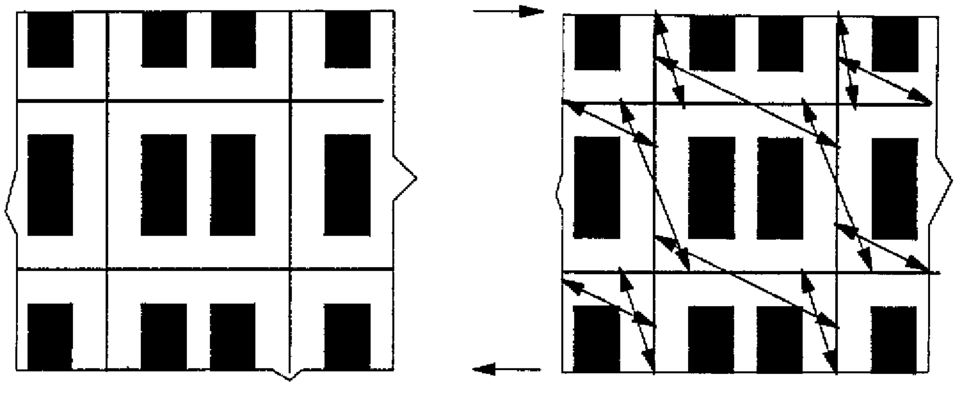

Fig. 8 Typical Compressive Fields in Laterally Loaded Penetrated Wall Panel

Such behavior for a typical bay can be modeled fairly simply, using pairs of inclined braces, located on either side of the central pressure lines indicated in Fig. 8b. Such a model was developed for the typical bay of the example building, and is shown in Fig. 9. Based on previous studies, struts in this model were assigned axial areas equal to a rectangular section with thickness matching that of the wall and width equal to twice that of the wall. Steel column elements were modeled conventionally. Each beam was broken into a series of segments, this allows the relatively low plastic moment capacity of the semi-rigid framing connections to be modeled separately from the larger plastic moment capacity of the steel section.

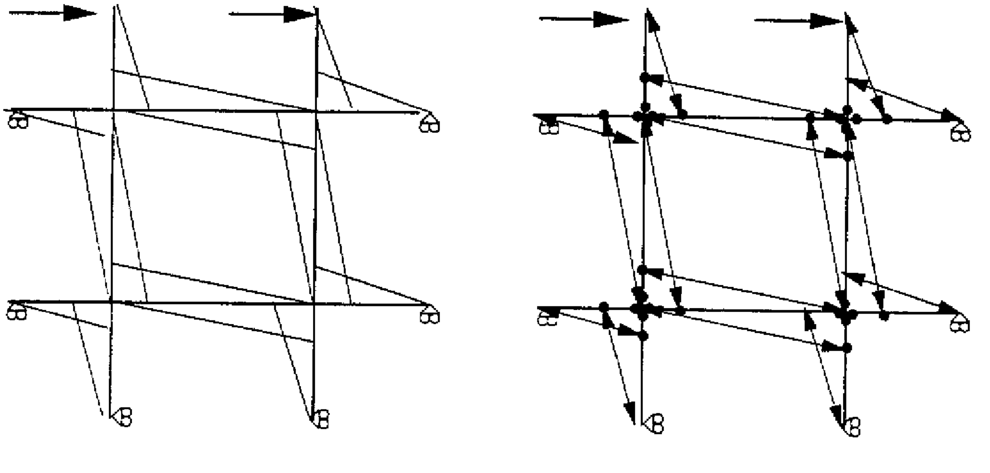

Fig. 9. Simple Model of Infill Strut Behavior

DRAIN 2D (12) software was used to perform a static pushover analysis of the model representing the typical bay. This curve is presented in Fig. 10. At any lateral deformation level, the total story stiffness may be taken as the stiffness of the representative panel, factored by N/2, where N is the total number of columns in the frame at the particular story. An iterative solution technique was then utilized to determine the demands induced on the structure by the design earthquake, following an approach previously suggested in (9). In this approach, a three dimensional analytical model of the structure was created. The steel frame was modeled using the elastic properties of the beams and columns. The stiffening effect of the masonry infill was modeled by a pair of diagonal struts across each bay of the frame. The stiffness of the struts is selected to match the secant stiffness of the typical bay model presented above, at a given deformation level.

Initially, a demand deformation resulting from the design earthquake is estimated using judgment and an appropriate secant modulus determined for the infill masonry. Then, the global diagonally braced frame model is subjected to a response spectrum analysis, using the stiffness obtained from the first estimate, and a response spectrum adjusted for an appropriate amount of effective damping, based on the expected extent of hysteretic response. The analysis will typically indicate a somewhat different pattern of story deformations than originally estimated. Using these new estimates of the deformation demand on the structure, a new series of secant modulus stiffnesses is selected from the local pushover model, and the global three dimensional model is revised using these properties. The response spectrum analysis is repeated and a new pattern of deformations derived. Typically, three or four iterations will result in convergence of the predicted deformation pattern. When this convergence occurs, the displacement demand on the structure will have been determined and the stress state on masonry and framing elements can be obtained directly from the nonlinear model and DRAIN analysis. For the example structure, interstory drifts of approximately 6.3 cm, uniformly distributed throughout the structure, were determined as the demand produced by the design response spectrum. This corresponded to achievement of nearly a full mechanism in the frame, as indicated by the location of the final secant in Fig. 10. Evaluation of compressive demands in the various masonry struts for this demand level were acceptable. However, axial loads in the end columns of frames resulting from structure overturning were excessive, indicating a potentially hazardous condition. Strengthening of the structure, through local reinforcement of the corner columns, could permit the building to withstand the expected demands.

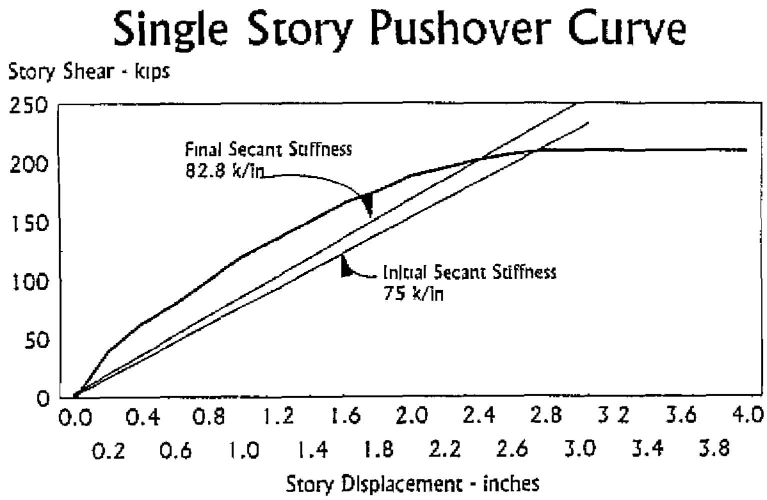

Fig. 10 Pushover Curve for Story, showing iterative deflection estimation approach

A capacity spectrum method analysis of the building was also performed, as a check on the iterative approach. To perform this analysis, a stick model was constructed consisting of a series of shear beams, each representing the stiffness of an individual story, stacked one upon the other. The stick model was subjected to an incrementally increased pattern of lateral loads, with a vertical distribution equal to that specified by the Uniform Building Code (7) for the static analysis procedure. At a given level of lateral loading, the stiffness of each story was taken as that predicted by the individual story pushover analysis previously described. The total pushover curve for the structure, consisting of a plot of the lateral deflection at the center of mass against the total applied base shear was then plotted against the design response spectrum, as shown in Fig. 11. The intersection of the two curves represents the total demand base shear and lateral deflection of the structure.

The intersection of the two curves occurs at a lateral deflection of the center of mass of the structure of approximately 51 cm. Note that this corresponds to a roof deflection of approximately 64 cm, which corresponds well with the 6.3 cm average interstory drift predicted by the iterative approach. Therefore, the two approaches to estimating the earthquake induced demand on the structure produced similar results.

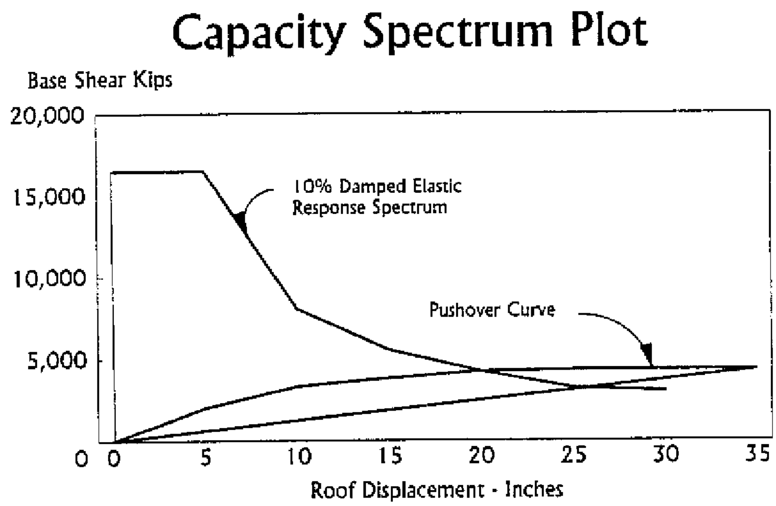

Fig. 11 Capacity Spectrum Plot for Example Structure

CONCLUSIONS

Current elastic seismic analysis methods are inadequate for the estimation of the internal force and displacement distributions. Codes and guidelines under development are introducing relatively simple nonlinear approaches. Several variations of this approach are described, among them the Capacity Spectrum Methods, which relies on a nonlinear static pushover analysis of the structure. Examples are given for this approach, including infilled frames.

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