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FEDEAS: NONLINEAR ANALYSIS FOR STRUCTURAL EVALUATION

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ABSTRACT

The ever increasing power of desktop computers, the ever increasing complexity of the design and analysis of new structures and the ever increasing need for the rational evaluation and strengthening of existing structures require a versatile, sophisticated and adaptable library of structural elements for use in a general purpose finite element analysis program. At the same time, guidelines for model selection are indispensable for the use of nonlinear analysis is practice. This paper addresses issues of element development and element implementation in the nonlinear dynamic analysis of structures.

KEYWORDS

Nonlinear analysis; seismic response; buildings; structural evaluation.

INTRODUCTION

The evaluation of the nonlinear behavior of structures depends on the development of advanced analytical models, which describe the time and load dependent behavior of the structural members. These models should satisfy two basic requirements: (a) they should be reliable, robust and computationally efficient, and, (b) they should be of variable complexity depending on the degree of detail required from the analysis: while individual critical members of the structure need to be evaluated with sophisticated finite element models, the overall behavior of multistory buildings and multiple span freeway structures can be described with sufficient accuracy with simpler member models. In fact, the ability to combine finite element models of critical regions of the structure with nonlinear or even linear member models of the rest of the structure should be an important consideration. Furthermore, the ability to refine a particular element model to the desired degree of detail is another important consideration in the development of such models.

An appropriate platform for the development of structural member models of variable complexity is a general purpose finite element analysis program that meets the following requirements: (a) it allows for an easy and transparent addition of elements to the program, (b) it provides utilities for input data generation, data storage and manipulation and output of the results, but allows the user to also easily incorporate custom-made utilities for new elements, (c) it provides several nonlinear solution strategies for static and dynamic analysis, but allows the user to also include custom-made solution strategies, (d) it provides utilities for, at least, rudimentary graphical pre- and post-processing, and, (e) it is so lean and efficient that it can run on a variety of platforms ranging from personal computers to workstations. Last, but most important, requirement is that the program be capable of accommodating three-dimensional as well as two-dimensional structural models.

The proposed structural element library FEDEAS (Finite Elements for Design, Evaluation and Analysis of Structures) is built around the finite element analysis program FEAP by Robert L. Taylor of the University of California, Berkeley. Salient features of FEAP are documented in Zienkiewicz and Taylor (1989), while a complete manual is in preparation.

This paper presents some salient features of the structural element library FEDEAS. The generality of FEAP makes a distinction between two-dimensional (2D) and three-dimensional (3D) elements unnecessary. Some of FEAP's capabilities are discussed in the examples. The report by Filippou (1996) contains a more comprehensive discussion.

FORMULATION OF STRUCTURAL ELEMENTS

The first release of the structural element library FEDEAS consists of a few elements and a collection of uniaxial hysteretic material models. The proposed elements are three-dimensional, but can be used equally well in a 2D analysis. The elements are: a linear and nonlinear truss element, a linear and nonlinear frame element, and a nonlinear hinge element. In the final development stages is a cable element with nonlinear geometry, a nonlinear frame element for prestressed concrete members with bonded or unbonded tendons, a frame element with relative slip between constituent components that models bond between reinforcing steel and concrete or partial composite action between concrete deck and steel girder.

The common characteristic of most of these elements is that they are based on the flexibility method of analysis. In this case the element stiffness matrix is derived by inversion of the flexibility matrix, which is formulated with the virtual force principle: the relation between internal and external work is based on force-interpolation functions that relate the internal forces at a cross section along the element axis to the end forces. This approach offers several advantages over commonly used stiffness-based elements: (a) the force-interpolation functions are exact solutions of equilibrium conditions for the frame element and can, thus, be readily established even in the presence of element loads; this is not the case with stiffness-based models, where the displacement interpolation functions are not exact for nonprismatic and/or nonlinear elements; (b) the strict satisfaction of equilibrium yields superior numerical robustness and accuracy in the presence of strength loss and softening, which can be expected in the evaluation of older concrete structures with poor detailing, but also in new steel structures with fracturing connection behavior; (c) the direct inclusion of element loads yields a significant reduction in the number of nodes and elements of the structural model.

The incorporation of flexibility based elements in a structural analysis program that centers on the direct stiffness method is not as straightforward as for stiffness based elements. This explains the limited number of such elements, even though a few investigators have recognized their superiority (Zeris and Mahin 1991). Even the few elements proposed to date have failed to give a consistent formulation of the element and its numerical implementation in a structural analysis program. Even though the solution to this problem was first sketched out in the paper of Ciampi and Carlesimo (1986), the study by Spacone resulted in the first consistent formulation and clear numerical implementation of a flexibility-based frame element in a general purpose finite element analysis program (Spacone *et al.* 1996a). The algorithm in the latter study proved to be computationally efficient and numerically robust, even in the presence of strength loss and material softening. This algorithm points out that the consistent state determination of flexibility based elements needs to be based on residual deformations, much like the structure state determination is based on residual forces in the well-known Newton-Raphson iteration and its variations.

The consistent theoretical framework of the study by Spacone *et al.* (1996a, b and c) allows the formulation of a class of flexibility based frame elements with either distributed or concentrated end inelastic deformations. In either case the hysteretic behavior of the section can be described by means of a force-deformation relation or can be derived from the hysteretic behavior of individual fibers into which the section is subdivided. In the former case it is really not possible to describe the interaction between internal forces in a rational way, as is the case in the latter at the expense of some complexity.

MATERIAL LIBRARY

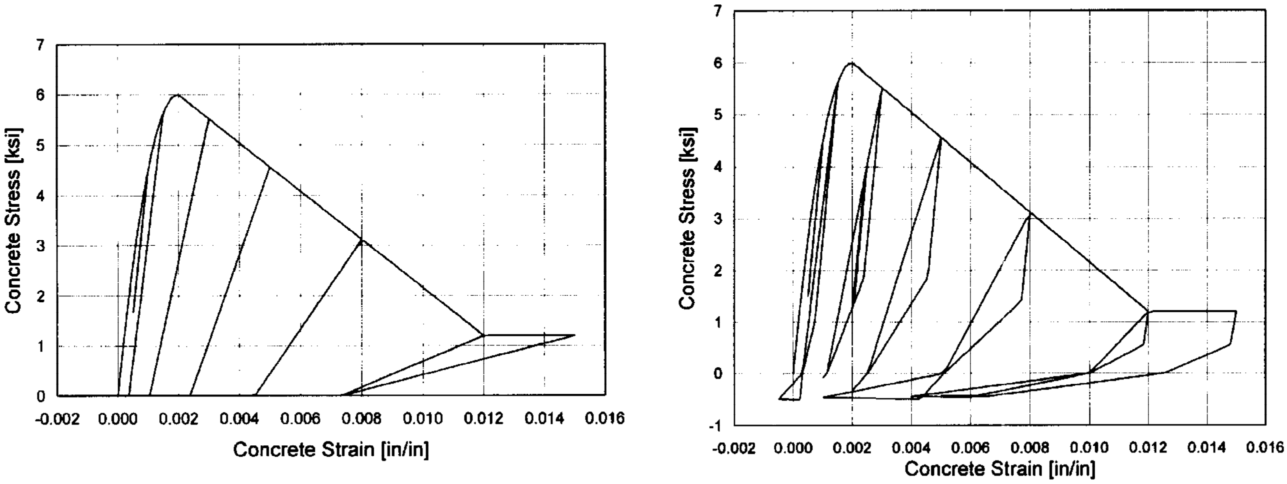

Figure 1-Hysteretic Stress-Strain Relation of Concrete Material Models

The material library of FEDEAS consists of uniaxial force-deformation relations that describe the hysteretic behavior of fibers or sections of the structural elements. Several models of the same material are available, allowing the user to select the desired level of complexity. There are, thus, three models for the hysteretic behavior of concrete: (a) a model with no tensile strength, (b) a model with tensile strength and a linear tensile strain softening branch, and (c) a model with tensile strength and a nonlinear tensile strain softening branch. All three models have the same behavior in compression. Model (a) has a simple rule for loading-unloading in compression, while models (b) and (c) follow a slightly more complex rule. Figure 1 shows a typical stress-strain history for concrete models (a) and (b).

The material library also contains several hysteretic steel models. Figure 2 shows the characteristic hysteretic behavior for two of these: a bilinear model with isotropic strain hardening in tension and compression and a nonlinear steel model according to Giuffré-Menegotto-Pinto modified to include the same isotropic strain hardening as the bilinear model.

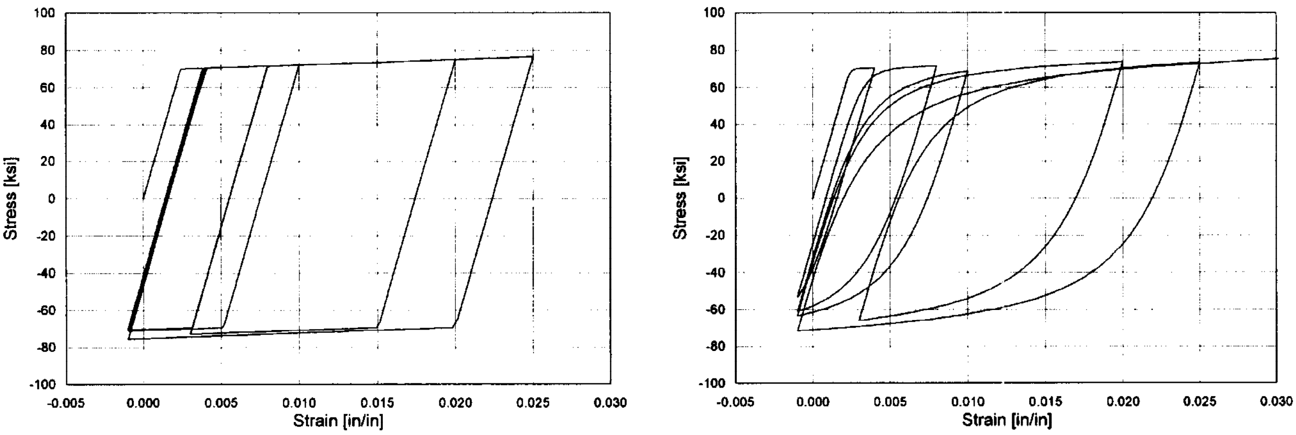

Figure 2-Hysteretic Stress-Strain Relation of Steel Material Models

Finally, the library includes several generic hysteretic force-deformation models that can be either used in the modeling of individual fibers or in the modeling of the force-deformation behavior of plastic hinges and the section force-deformation behavior of distributed inelasticity frame elements. Two examples, one from a model with bilinear envelope and one from a model with trilinear envelope are shown in Figure 3. The model with the trilinear envelope is shown with a negative (softening) second slope in the negative force-deformation quadrant.

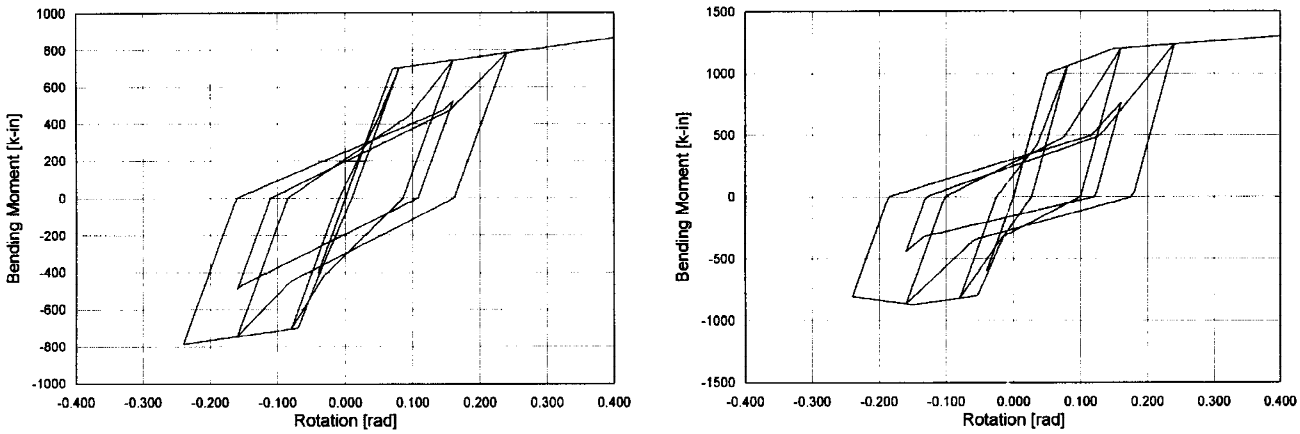

Figure 3- Generic Hysteretic Models for Fibers or Cross Sections

MODEL SELECTION IN NONLINEAR ANALYSIS

Guidelines for model selection are indispensable for the use of nonlinear analysis methods in practice. This problem has received scant attention in the literature to date, and much remains to be done. The following observations and questions serve as a guidepost for further investigations:

- A widely held view is that the subdivision of a structural element in several control sections and the subdivision of each control section into fibers results in a computationally expensive model. The following objections may be advanced against this view: (a) the comparison involves models of very dissimilar capabilities and it is not clear whether the limitations of simpler models are important in the assessment of structural behavior or not; (b) in the nonlinear analysis of a large structure most time is spent in the nonlinear solution algorithm of the global equilibrium equations and not in the element state determination.

- What is the necessary number of control sections in distributed inelasticity frame models for accurate global and local measures of response? Many studies to date fail to address the fact that the model selection is very much dependent on the objective of the analysis.

- What is the necessary number of fibers in a cross section discretization? This again depends on the objective of the analysis, but also on the complexity of the material and the loading history of the structural member.

The following examples represent a very small sample of a larger study that is under way at the University of California, Berkeley. Even though much remains to be done before offering definite conclusions on model selection, these examples help to outline possible answers to the questions above.

EXAMPLES

Steel Column under General Load History

The first example refers to a cantilever steel column of 30' height with a W14x730 cross section (Fig.4). The column is subjected to zero axial load and a general displacement history at the free end, as shown in Fig. 5a. The column cross section is divided into fibers with a nonlinear hysteretic stress-strain law similar to that in Fig. 2b. Figs. 5b, 5c and 5d show the bending moments at the base of the column for the following fiber discretizations: the web and each flange are divided into 3, 4 and 5 fibers along the width and one fiber across the thickness. These results are compared with the reference solution obtained with 20 fibers across the width and three fibers across the thickness. The results show that even as few as three fibers permit an adequate representation of the force-deformation relation, even though the yield envelope is clearly underestimated in the weak bending direction. The response in the strong bending direction is excellent, even for three fibers. The increase of the number of fibers to 5 yields results very close to the reference solution, even in the weak bending direction. Thus, a 5x1 fiber discretization scheme is quite adequate for the representation of the biaxial section response, while a 3x1 fiber discretization might still yield satisfactory results for the dynamic load-displacement history of a multistory-structure.

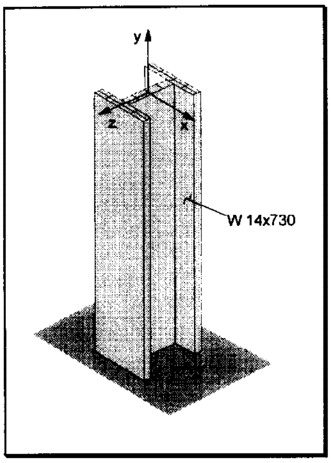

Figure 4- Cantilever Steel Column

The previous conclusion needs revision, if it is important to track the stress-strain history of individual fibers, and, particularly, if damage of the material results upon attainment of a critical strain value, as might be the case for steel under local instability or fracture and for poorly confined concrete. In this case, a larger number of fibers might be necessary for the accurate assessment of the ultimate member deformability.

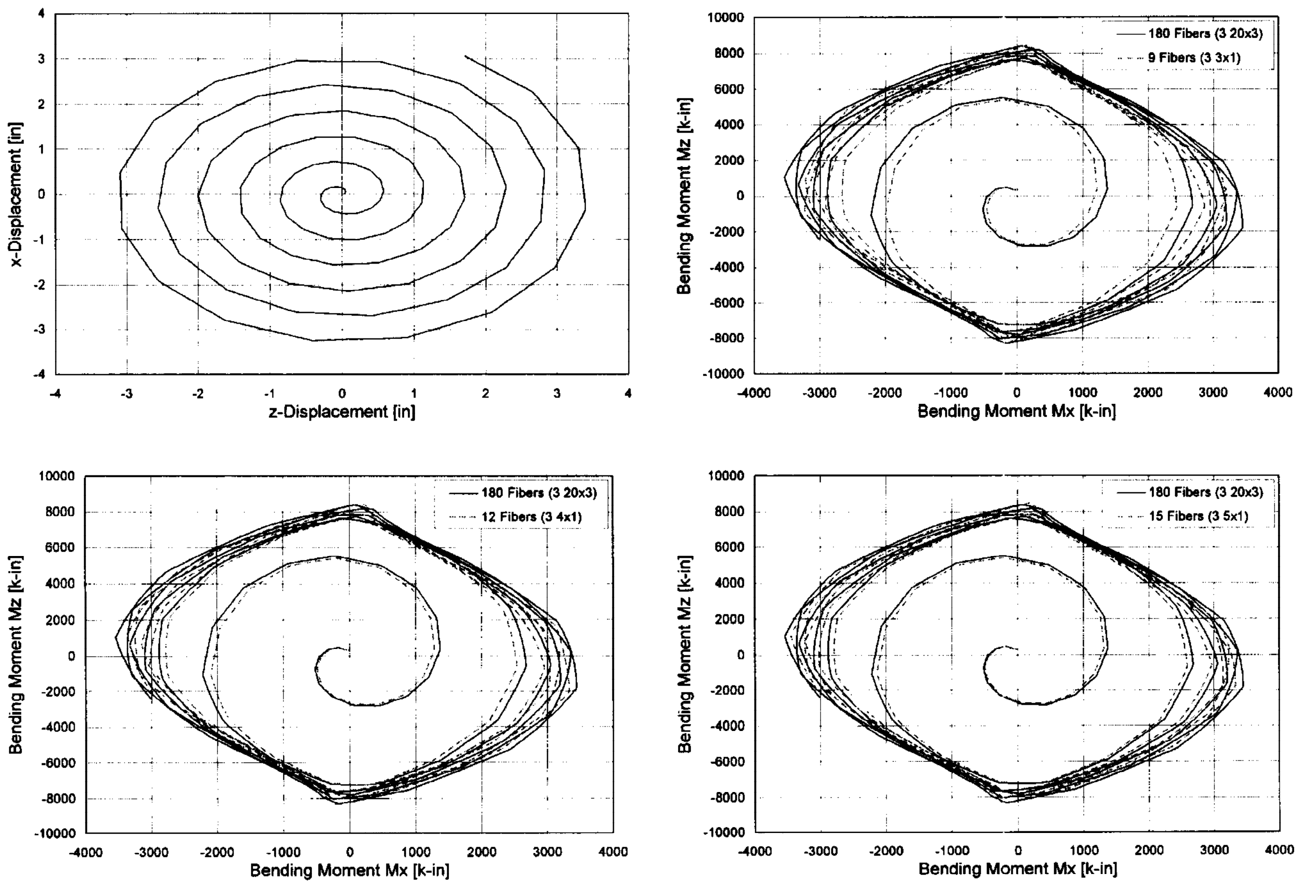

Figure 5- Column Response with Different Fiber Section Discretizations

18-Story Steel Frame with Fracturing Connections

The last example refers to the nonlinear static and dynamic analysis of an 18-story steel moment resisting perimeter frame structure. Fig. 7 shows the three dimensional model of the structure. In the first phase of the study a typical plane frame in each principal building axis was investigated. These studies form part of the evaluation of this structure which suffered widespread damage in the welded beam-to-column connections by fracture during the 1994 Northridge earthquake. More details can be found in Anderson and Filippou (1996).

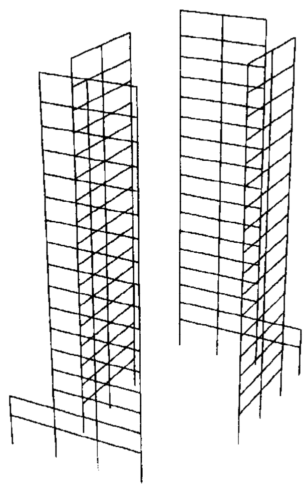

Figure 7- 18-Story Steel Frame Model

By subdividing the girder cross into fibers and assigning a fracturing steel stress-strain relation to the fibers of the bottom flange it was possible to simulate realistically the moment-rotation behavior of the girders with fracturing bottom flanges, as shown in Fig. 8.

The first step was the study of the nonlinear static load to collapse response of the plane frames with different models and modeling assumptions.

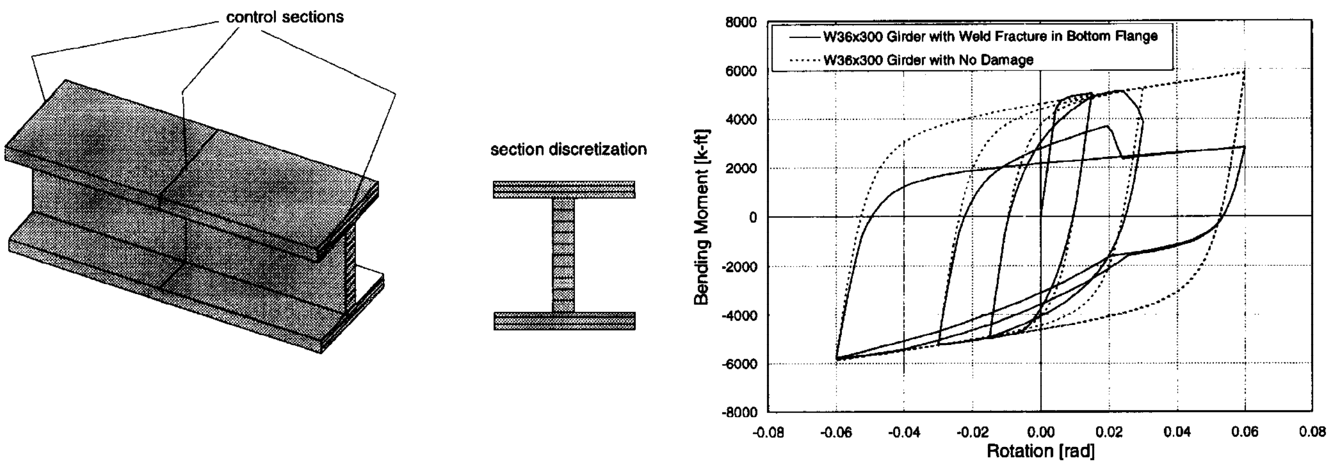

Figure 8- Moment-Rotation Response of Girder with Fracturing Connection

Fig. 9 shows the results for frame A. The effect of the rigid panel zones and the second order effects is clearly visible. A simple moment-curvature girder hysteretic model yields virtually identical results to the more sophisticated fiber cross section model for girders with no fracture and without the effect of distributed gravity loads.

Figs. 10-12 show the results of the dynamic analyses of Frame B to the recorded Canoga Park and Sylmar records. The maximum interstory drift in Fig. 10 shows that most of the earthquake damage concentrates in floors 10 through 16 in good agreement with the observed connection damage. This damage concentration is even more pronounced with the use of a fracturing connection model in the girders of the model of Frame B. Such a connection model amplifies the interstory drift in the upper floors and reduces it in the lower floor of the frame (Fig. 10). It is interesting to observe in Fig. 11 that the top story displacement response does not increase with the fracturing connection model: the elongation of the frame period after the connections fracture following the first inelastic excursion at 4.72 sec has little effect on the response for the Canoga Park record. The concentration of connection fractures in floors 10 through 16 is also very clear in the displacement distributions at the first three top displacement peaks of the response in Fig. 12.

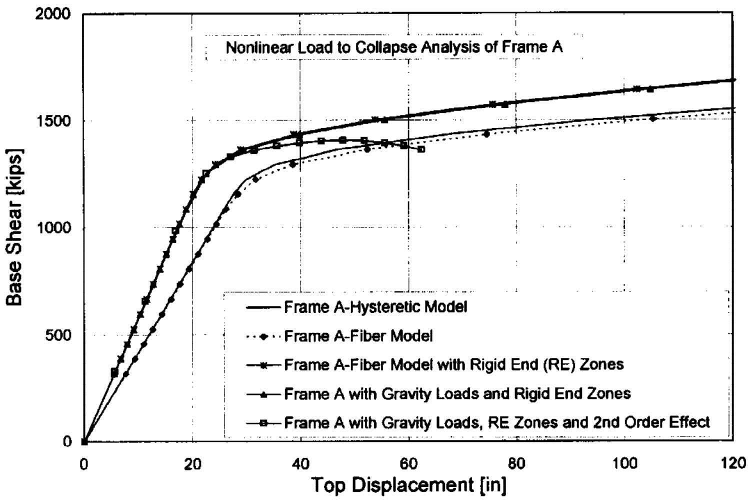

Figure 9- Effect of Gravity Loads and Second Order Effect on Load-Displacement Response of Frame A

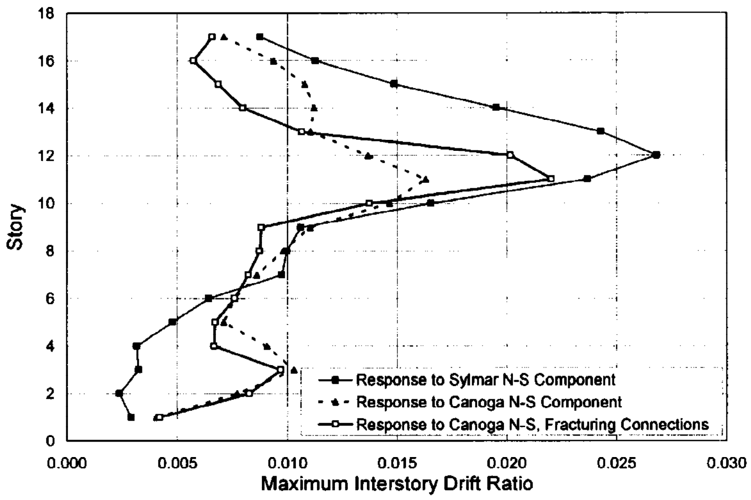

Figure 10- Maximum Interstory Drift Ratio for Frame B under N-S Component of Canoga Park and Sylmar Records; Effect of Connection Fracture

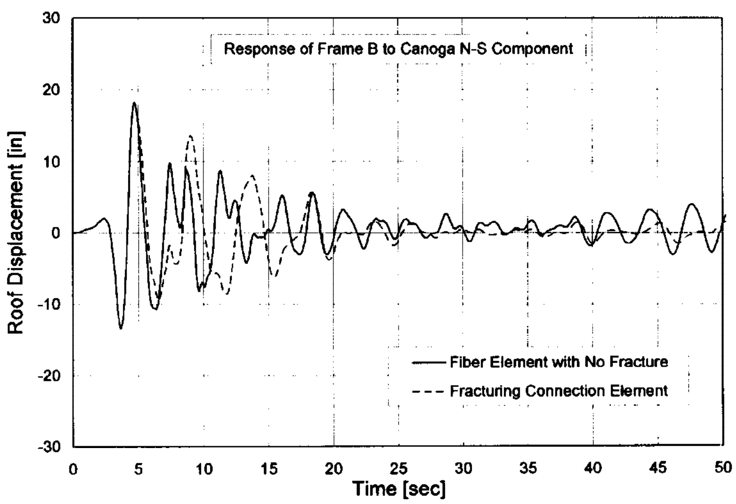

Figure 11- Effect of Connection Fracture On Roof Displacement Time History Of Frame B Under N-S Component Of Canoga Park Record

CONCLUSIONS

The rational evaluation and strengthening of existing structures require a versatile, sophisticated and adaptable library of structural elements for use in a general purpose finite element analysis program. At the same time, guidelines for model selection are indispensable for the use of nonlinear analysis is practice. The paper illustrates with three examples, how model selection is intimately connected with the objective of the analysis: while a relatively simple model might yield accurate global response results, local ductility estimates and damage predictions require more refined analytical models that account for internal force interactions, the presence of distributed gravity loads and the mechanical characteristics of the constituent materials. The paper introduces a library of such elements that are flexibility based and offer several advantages over commonly used stiffness based elements.

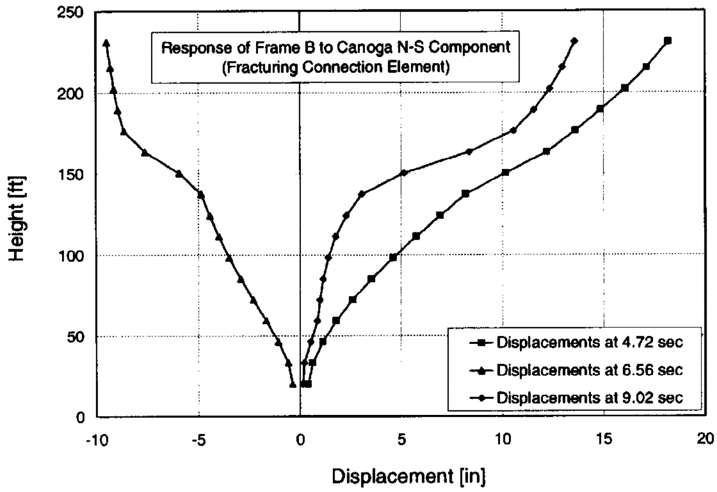

Figure 12- Displacement Distribution Snapshots of Frame B at First Three Displacement Peaks of Earthquake Response in Fig. 14 with Connection Fractures

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