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AMETRIC STUDIES ON INELASTIC
LLING OF STEEL MOMENT FRAMES

Report to the SAC Joint Venture
Task 3.5

FINAL REPORT

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ABSTRACT

This report summarizes the results of a brief study examining the effects of various modelling assumptions on the predicted inelastic response of a steel moment frame. This study was conducted as part of SAC Task 3.5.

The objective of the study was to determine how sensitive the predicted inelastic response of a moment frame may be to the assumptions used in developing the structural model. A variety of computer models were constructed for a six story single bay steel moment frame. Two basic classes of models were considered: "refined" models, and a "baseline" model. The baseline model used simple bilinear representations of inelastic response, and modelled bare steel behavior only (no composite floor). The refined model employed more realistic hardening rules that better represent experimentally observed inelastic behavior, and included composite floor slab effects. In addition to comparing the refined model to the baseline model, variations in other modelling parameters were also considered. The response of the models to four different loading cases were considered: a static pushover analysis and three strong ground motion records.

Some of the major conclusions of this study can be summarized as follows:

- The results of this study indicate that the predicted inelastic dynamic response of a steel moment frame is quite sensitive to modelling assumptions. Key inelastic response parameters, such as predicted beam plastic rotation demands, can change by more than 100% as model parameters are varied.

- The predicted response of a steel moment frame can be significantly affected by the presence of a composite floor slab. Neglecting composite floor slab effects can substantially underestimate the stiffness and strength of the frame, and alter the predicted plastic rotation demands.

- The predicted inelastic response of a steel moment frame is particularly sensitive to the assumed value of yield stress for the beams and columns of the model.

- Varying the model assumptions appears to have a very important effect on the predicted location of yielding within a joint. This is particularly true for frames in which the panel zone is relatively weak compared to the beam. Changing the modelling assumptions can cause the predicted location of yielding at a joint to shift almost completely between the beam and panel zone.

- In general, the results of this study indicate that varying modelling assumptions has a substantially greater impact on local response predictions such as beam plastic rotation, than on global response predictions such as interstory drift ratio. Thus, the accuracy of the model is much more important if local member and joint response predictions are of interest. The accuracy of the model is less important if only global response predictions, such as interstory drift or roof displacement, are of interest.

Inelastic dynamic analysis can be a valuable tool for developing an improved understanding of how a steel moment frame may respond to a strong earthquake. Such an analysis, guided by judgement, can be used to estimate approximate plastic rotation demands at critical locations in a steel moment frame. However, the analyst must recognize that large uncertainties exist in modelling inelastic behavior. Inelastic deformation demands cannot be predicted with great precision. Uncertainty in modelling inelastic behavior must be recognized when making design decisions.

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1.0 INTRODUCTION

This report documents the results of an analytical study on inelastic modelling of steel moment frames for earthquake analysis. This study was conducted as part of SAC Task 3.5.

The objective of this study was to investigate the sensitivity of the predicted inelastic response of a steel moment frame to some of the assumptions used in developing the models. Inelastic dynamic analysis is used as a tool to estimate the forces and inelastic deformation demands in the members and joints of a frame subject to strong ground motions. This study takes a brief look at how the predicted inelastic deformation demands, particularly the beam plastic rotation demands, change as the model parameters are changed. The results of this study are intended to provide an indication of the potential variability that might be expected in the predicted plastic rotation demands as a consequence of modelling uncertainties.

All of the analyses in this study were conducted for the same six story, single bay steel moment frame. The response of this frame was predicted using a variety of inelastic models. The major variables considered in constructing these models were as follows:

- the use of simple bilinear type inelastic models, versus the use of more refined models that employ more realistic hardening rules, for modelling the beams, columns, and panel zones;

- the use of bare steel models, versus the use of models that incorporate the effect of composite floor slabs in the response of the beams and panel zones;

- the effect of varying the value of yield stress assumed for the beams, columns, and doubler plates.

The influence of these model variables on the response of the frame was examined for four loading cases: a static pushover analysis and three strong ground motion records.

This study is not intended to be exhaustive. Rather, it is intended to provide a preliminary indication of some of the uncertainties involved with inelastic dynamic analysis of steel moment frames. This study was performed as a brief supplement to a SAC Task 3.1 study conducted by the writers on the response of a six story steel building in the Northridge Earthquake.

The remainder of this report is organized as follows: Section 2 describes the steel moment frame used in the analyses and the analytical models constructed of the frame; Section 3 summarizes the analytical results; and Section 4 provides conclusions.

2.0 DESCRIPTION OF FRAME AND MODELS

2.1 DESCRIPTION OF FRAME

A single moment frame was chosen for all of the analyses conducted in this study. An elevation of this frame is shown in Figure 1. This frame is part an actual six story steel building located in Santa Monica, California. The building was part of the SAC Task 3.1 Study, and was designated as Site 7 for that study. A large number of the moment frame connections were damaged in this building during the 1994 Northridge Earthquake. A complete description of the building and the damage are provided by Engelhardt, et al (1995). The frame chosen for the current study (Figure 1) was located along Grid Line G of the building. This frame was one of four single bay moment frames that provided the lateral resisting system for one direction of the building. The fact that the frame shown in Figure 1 is part of a building in Santa Monica is somewhat incidental for the purposes of this study. This frame was chosen simply as representing a realistic steel moment frame designed in accordance with the Uniform Building Code.

Some of the key features of the moment frame used in this study are as follows:

- The frame was designed in accordance with the 1985 Uniform Building Code.

- The frame has six stories above grade. The steel columns extend one level below grade, and are nominally pinned at their base. The ground level is considered the seismic base. At the ground level the columns achieve some degree of rotational restraint by the "backstay" effect achieved by their extension to the first basement level. The boundary conditions assumed in modelling the frame are as shown in Figure 1.

- All beams are specified as A36 steel, and all columns are specified as A572 Gr. 50 steel. The actual yield stress assumed in modelling the frame members was one of the variables considered in this study.

- All beams in the frame are composite beams with welded shear studs. The floor deck at the ground floor is 4 1/2" normal weight concrete on 3" deep composite metal deck (7 1/2" total thickness). The floor deck from the second floor to the sixth floor is 3 1/4" light weight concrete on 3" deep metal deck (6 1/4" total thickness). The roof is of insulating concrete (vermiculite) on 1/2" metal deck. The 28 day compressive strength for all concrete floors was specified to be 3000 psi. The ground floor deck was provided with #4 reinforcing bars spaced at 18 inches. The remaining floors were provided with a welded wire mesh with .192" diameter wires spaced at 6 inches (6x6 - W2.9xW2.9). These floor deck characteristics were used in developing the composite beam models for this frame.

- Beam-to-column moment connections were constructed using the conventional welded flange - bolted web detail. The beam flanges were welded to the column using complete penetration groove welds. The beam web was attached to a single plate shear tab using high strength bolts. No supplemental welds were provided between the shear tab and the beam web. Doubler plates were provided at the joints (see Figure 1 for doubler plate thicknesses). The doubler plates were specified as A572 Gr. 50 steel.

- To compute seismic masses, the total weight of the frame was taken as 1933 kips. This was taken as one-quarter of the estimated total dead load for the structure, including the facade and a 10 psf allowance for partitions. Approximately one-quarter of the total dead load was used for the frame, because the building had a total of four moment frames (with similar values of lateral stiffness) providing lateral resistance for the building. The distribution of the 1933 kips among the floors of the frame is listed in Table 1.

2.2 DESCRIPTION OF THE ANALYTICAL MODELS

In order to examine the effects of inelastic modelling assumptions on predicted structural response, two classes of models were constructed. In this study, these are referred to as a "baseline" model, and a "refined" model. For each class of model, several model parameters were varied as part of this study. All models used in this study are plastic hinge type models, in which inelastic effects are assigned to plastic hinges at the member ends, or to a rotational spring in the case of the panel zone element. All models were implemented in the ANSR-1 computer program (Mondkar and Powell 1975). Some of the key features of the models are described in the following sections.

2.2.1 BASELINE MODELS

The elements used to construct the baseline models are simple bilinear representations of inelastic response that have been widely used in the past, and are still widely used for modelling steel moment frames. Following are brief descriptions for the two element types used for the baseline models:

<u>Beam-Column Element</u>
Both the beams and the columns are modelled using a bare steel element. The end moment versus end rotation response is bilinear, with the strain hardening stiffness taken equal to 3% of the elastic stiffness. For modelling beams (no axial force), flexural yielding of the element occurs when the end moment reaches M_p. For modelling columns (with axial force), the moment at which yielding occurs depends on the axial force in the member, and is computed using a classical linear moment-axial force interaction formula. The relationship between axial force and axial deformations is assumed to remain linear elastic. That is, plastic axial deformations are not modelled. For both beams and columns, the model is based on the clear length of the member. The joints are modelled explicitly using a panel zone element.

Panel Zone Element

Joint flexibility and yielding are modelled using a panel zone element. This element consists of a rotational spring connecting the beam element to the column element. The spring is used to represent the relationship between unbalanced moment at the joint versus the relative rotation between the beam and column. The symbol γ is used to denote the relative rotation between the beam and column, since this rotation is largely the result of shear deformation of the panel zone. The panel zone element provides a bilinear representation of response. Yielding of the panel zone element occurs when the panel zone shear (developed as a result of unbalanced moment) reaches a value of $.55F_y t_{cw} d_c$, where t_{cw} is the thickness of the column web and doubler plate, d_c is the column depth, and $.55F_y$ is the yield stress in shear. The strain hardening stiffness of the element is taken as 3% of the elastic stiffness.

2.2.2 REFINED MODELS

The elements used to construct the "refined" inelastic models were developed in a recent study by Kim (1995). These elements were specifically developed to model steel moment frames under cyclic loading, and are believed to provide more realistic and accurate predictions of structural response, as compared to the conventional bilinear elements described above. These refined elements are briefly described below, along with some comparisons between model predictions and experimental results. Each of the elements were calibrated by comparison with experimental data, or by comparison with more sophisticated models. Detailed descriptions of the elements, and a description of the process used to calibrate and verify the elements, are provided by Kim (1995).

Composite Beam Element

A composite beam element was developed to provide an approximate representation of the cyclic inelastic response of composite beams with flexural yielding at the member ends. Figure 2 shows an outline of the hysteretic response of this member. The stiffness and strength of the element under positive moment are based on the properties of the composite section. The initial response in the first cycle of loading under positive moment is bilinear. For subsequent positive moment cycles, pinching effects due to crack closing in the concrete slab are included in the model. Strength and stiffness under negative moment are based on the properties of the bare steel beam combined with the reinforcement in the slab, if any. Hysteretic response under negative moment is curvalinear based on bounding surface modelling concepts. The model accounts for variations in the location of the inflection point when computing flexural stiffness.

Composite beam response is quite complex, and the composite beam element can, at best, be considered an approximate representation of this response. A comparison of model predictions with experimental results are shown in Figure 3. Comparisons are provided for an experiment by Lee (1987) and for an experiment by Tagawa (1986). These comparisons indicate that the composite beam element provides a reasonable representation of the experimental responses. Although there are numerous uncertainties in modelling composite beams, it is believed that the use of this composite beam element provides a significant improvement in response prediction compared to the use of conventional bare-steel elements.

Bare Steel Beam-Column Element

A bare steel element was implemented for the refined inelastic model for two purposes: (i) modelling of the columns; and (ii) modelling of bare steel beams, to compare with the response predictions using the composite beam element.

The bare steel beam-column element models inelastic response using plastic hinges at the member ends. The element models changes both in flexural and axial stiffness, using several yield surfaces to account for interaction between bending moment and axial force. Initial yielding and subsequent hardening follow the multilinear force-deformation relationships shown qualitatively in Figure 4. Strain hardening results in translation and expansion of the yield surfaces, shown qualitatively in Figure 4(c). Mroz and isotropic hardening rules are combined to model hardening under monotonic, symmetric cyclic, or random loading. For modelling bare steel beams (with no axial force), model parameters have been developed for beams with all-welded connections, as well as for beams with welded flange-bolted web connections.

Figures 5 and 6 show comparisons between cyclic loading experiments on beams (no axial force) and model predictions for both the baseline bilinear element, as well as for the refined multi-linear element. Figure 5 shows results for a beam with an all-welded connection to a column. Note that the baseline model significantly underestimates the actual strength of the beam. Figure 6 shows results for a beam with a welded flange-bolted web connection to a column. In this case, the bilinear element overestimates the actual strength of the beam.

For steel elements under combined bending and axial load, experimental data is scarce. Consequently, predictions of the hinge type models are compared with predictions from a more accurate fiber element type model, using a bounding surface stress-strain model for steel. These comparisons are shown in Figure 7. For this analysis , the member is subject to a constant axial force and cyclic bending. Note that the baseline bilinear element significantly underestimates the strength of the member.

Panel Zone Element

Figure 8 shows an outline of the hysteretic response of the panel zone element used for the refined models. For the first half-cycle of loading, the response follows a trilinear monotonic model. After the first half cycle of inelastic loading, the model follows a curvalinear response based on bounding surface modelling concepts.

Figure 9 compares response predictions for the panel zone element with the response measured in an experiment by Krawinkler (1971). Note that the correlation is reasonably close. A similar comparison is made in Figure 10, using the bilinear panel zone element. The bilinear element provides a rather poor response prediction.

Modelling of panel zones in frames with composite beams requires further modification. The stiffness and strength of the panel zone (as represented by the relationship between unbalanced moment and panel rotation) are increased by the presence of the slab, because the depth of the panel zone is effectively increased. For the refined panel zone model, the depth of the panel zone for positive moment on an exterior joint is taken as the distance from mid-depth of the slab to the beam's bottom flange. For negative moment, the depth of the bare steel beam is used. Additional modelling parameters are also further modified based on the limited available experimental data. A comparison of model predictions with experimental data is shown in Figure 11. There is considerable uncertainty in modelling of panel zones in the presence of composite slabs.

A source of difficulty in modelling panel zone behavior is properly accounting for the contribution of doubler plates to the stiffness and strength of the panel zone. Experimental data suggest that the doubler plates are usually not fully effective, but the actual degree of participation is difficult to predict. For the refined panel zone element, doubler plates were assumed to be only 40% effective, That is, only 40% of the doubler plate thickness was included in the panel zone thickness. This is based on comparison of the model with panel zone experiments that include doubler plates (Kim 1995).

2.2.3 SUMMARY OF INELASTIC FRAME MODELS

As part of this study, eight different models were constructed for inelastic analysis of the frame shown in Figure 1. Six of these models used the refined elements, and two used the baseline elements, as described above. A summary of the eight different models is provided in Table 2. All of the models using refined elements are provided with a designation starting with "A." The models using baseline elements are provided with a designation starting with "B." Note that the models designated as "A" and "A-1" are identical.

In addition to the use of refined versus baseline models, two additional variables were considered in the study. The first is the yield stress of the steel members. For most of the models, F_y=47 ksi is used for the A36 beams, and F_y=57 ksi is used for the A572 Gr. 50 columns and doubler plates. These values are based on statistical averages for A36 and A572 Gr. 50 shapes from a recent steel industry sponsored study. For two of the models, however, different values are used. For Model A-2, minimum specified yield stress values are used (F_y=36 ksi for A36, and F_y=50 ksi for A572 Gr. 50). Model A-3 considers a case where the beam has a significantly higher yield stress than the column.

The other variable considered in the analyses is the degree of doubler plate participation in the panel zone model. This variable, ranging from 0% to 100%, represents the percent of the specified doubler plate thickness included in the thickness of the panel zone. For most of the refined models, 40% doubler plate participation is assumed, based on calibration of this model to available experimental data. However, for one case (Model A-1a), 100% participation of the doubler plate is assumed for the refined panel zone element, in order to evaluate the sensitivity of the analysis results to this variable. For the baseline model (Model B), 100% doubler plate participation is assumed, as is conventional with this model.

For both the refined and baseline models, an additional case is considered where the doubler plates have been eliminated from the model, i.e., 0% participation. This is represented by Models A-ndp and B-ndp (ndp=no doubler plate). These cases represent designs permitted since the 1988 Uniform Building Code, in which significant inelastic action is permitted in the panel zones.

Several features were common to all of the eight models considered in this study:

- The damping matrix is taken as a linear combination of the mass and tangent stiffness matrices. Mass and stiffness proportional damping coefficients were chosen to provide 2% of critical damping in the first and fourth modes.

- Gravity loads were not included in the analysis.
  Analysis of this frame as part of SAC Task 3.1 showed that gravity load effects were generally insignificant for this frame. Consequently, in order to simplify the analyses, gravity load effects were not included in this study.

- Second order effects were not included in the analyses.

- No contribution of gravity load framing was included in the lateral stiffness or strength of the model.

2.3 LOADING CASES

Several loading cases were considered for this study:

- Static pushover analysis, using a triangular load distribution over the height of the frame;

- 1994 Northridge Earthquake, Sylmar Olive View Hospital Record, North-South Component (designated "sylm.000");

- 1992 Landers Earthquake, Lucerne record, East-West Component (designated luc.270);

- 1985 Mexico City Earthquake, SCT-1 Record, East-West Component (designated "SCT-1 N90E").

These ground motion records were chosen somewhat arbitrarily, but are intended to represent very strong ground motions. These records are used only to provide a comparative assessment of the response predictions for the various inelastic models. These analyses are not intended to evaluate the safety of the frame or the adequacy of the design. Not all loading cases were run for all eight of the models.

3.0 ANALYTICAL RESULTS

3.1 OVERVIEW

This chapter summarizes and discusses the results of the inelastic frame analyses. The discussion will highlight the similarities and differences in the response predictions provided by the various analytical models for the single bay moment frame shown in Figure 1. As described in Chapter 2, eight different analytical models were constructed of this frame. Four different loading cases were considered: a static pushover analysis, and three strong ground motion records.

Table 3 lists which loading cases were run for each of the eight models. All four loading cases were run for Model A (also designated Model A-1) and for Model B. These represented the basic "refined" and "baseline" models. Note also that the sylm.000 record was run for all eight models, to provide a consistent basis for comparison.

For each analysis case run, certain common response measures are reported in this chapter:

- Roof displacement:
  For the static analysis, roof displacement versus total base shear is plotted. For the dynamic analyses, roof displacement versus time is reported.

- Envelope of maximum story drift ratios:
  For each analysis, this envelope indicates the maximum drift ratio developed at each story at any time during the analysis. The maximum values reported for each story do not necessarily occur simultaneously.

- Plastic rotations at selected joints:
  Plots are provided of beam moment versus beam plastic rotation and of panel zone moment versus panel zone rotation. For each analysis, these plots are provided at selected joints at the 2nd and 3rd floor levels. In the plots, a joint location designated as: "Joint at G3-2nd Floor" refers to the 2nd floor joint on grid line 3 (see Figure 1), etc. All plastic rotations listed in this report are measured from the initial undeformed position of the element (beam, panel zone, or column). For these analyses, generally little or no yielding developed in the columns, except at the ground floor. Consequently, this chapter only highlights the response of the beams and panel zones.

3.2 NATURAL PERIODS OF VIBRATION

Table 4 lists the natural period of vibration computed for each model, for the first four modes. The same mass was used for each model, so the differences in the natural period reflect the differences in the elastic stiffness of the models. Among the models that include doubler plates (all models except A-ndp and B-ndp), the first mode natural period varies from 1.82 seconds

(Model A-1a) up to 2.19 seconds (Model A-1b). The models with the highest stiffness (lowest periods) are those with composite beams. The higher frame stiffness is due primarily to the increased stiffness of both the beam and panel zone resulting from the presence of the floor deck.

For this frame, differences of up to 20% in the computed natural period are obtained due to differences in the elastic modelling assumptions. For dynamic analysis, this is large enough to expect differences in predicted structural demands on the frame for the same ground motion record. Thus, the differences in structural response of the various models for a particular ground motion record will be due both to differences in modelling, as well as to differences in structural demands.

3.3 RESPONSE OF MODEL A VERSUS MODEL B (REFINED VERSUS BASELINE MODEL)

All four loading cases were considered for both Model A and Model B. Model A is the refined model, using 40% doubler plate participation in the panel zone, and using statistical average yield stress values. In the opinion of the writers, Model A represents the most realistic model for this frame, of the various models considered in this study. Model B is the baseline model, and can be considered typical of the simplified inelastic models that are widely used for steel moment frame analysis. In the following sections, the response of these two models is compared.

3.3.1 STATIC PUSHOVER ANALYSIS

For the static pushover analysis, the same triangular distribution of lateral force was applied to Models A and B. The total lateral force was then increased until the roof displacement reached 50 inches. This corresponds to an average drift ratio for the entire frame (roof displacement divided by frame height) of about 5%. The plots in Figure 12 illustrate the results of the analysis.

Figure 12a plots base shear coefficient versus roof displacement. This plot indicates that Model A is about 20% stiffer and about 20% stronger than Model B. This can be attributed largely to the higher strength and stiffness of the composite beam and the panel zone in Model A. Both models were forced to the identical roof displacement value of 50 inches. Figure 12b indicates that the two models predict nearly identical drift ratios for all stories in the frame.

Beam plastic rotations are shown at two joints in Figure 12c. Based on these plots, it appears that the predicted beam plastic rotations vary up to about 50% between the two models. Note that at grid line 3, Model A predicts higher beam plastic rotations, whereas at grid line 4, Model B predicts larger beam plastic rotations. At a given floor level, Model B, in fact, predicts identical beam plastic rotations at each end of the beam. In contrast, Model A predicts significantly different plastic rotations at the two ends of a beam. This is a consequence of the composite beam model, which provides for different strength and stiffness for positive moment (at grid line 3) versus negative moment (at grid line 4).

Figure 12d shows panel zone rotations for the two joints at the 2nd floor. These plots show panel zone moment versus total (elastic + plastic) panel zone rotation, and clearly illustrate the differences in strength and stiffness for the two different panel zone models. Note that large differences in panel zone rotation are predicted by the two models. At grid line 3, Model B predicts much larger panel zone rotations than Model A. At grid line 4, the opposite is true. That is, Model A predicts larger panel zone rotations than Model B. This behavior is again due to the inclusion of composite floor slab effects in Model A. The composite slab not only results in different beam strengths for positive and negative moment, but also different panel zone strengths under positive and negative moment. For positive moment, the composite panel zone is significantly stronger because of the effective increase in panel zone depth. This effect is clearly evident in Figure 12d.

In Figure 12e and 12f, the variation of beam plastic rotation and panel zone rotation with story drift ratio are illustrated. These plots reinforce the previous observations. For the same story drift ratio, the two models predict significantly different beam and panel zone plastic rotations. Note that for a given story drift ratio, the total plastic rotation at a joint (beam + panel zone) is approximately the same for both models. However, the distribution of inelastic deformation between the beam and the panel zone can be significantly different between the two models.

To summarize, the results of the static pushover analysis indicates the following observations:

- For the same story drift ratio, the two models predicted approximately the same total plastic rotation (beam + panel zone) at most of the frame joints.
- The distribution of total plastic rotation at a joint between the beam and the panel zone is significantly different for the two models. The difference between the two models is greater for beams under positive moment than for beams under negative moment, due to composite floor slab effects.
- Because the distribution of yielding between the beam and panel zone are different between the models, the two models can predict significantly different beam plastic rotations for the same story drift ratio.

3.3.2 DYNAMIC ANALYSIS

A dynamic analysis was conducted for Models A and B for the Sylmar, Lucerne, and Mexico City records. The three ground motion records are shown in Figure 13. Highlights of the analytical results are shown in Figure 14 for the Sylmar record, in Figure 15 for the Lucerne record, and in Figure 16 for the Mexico City record.

For the Sylmar record, Models A and B show somewhat different roof displacement records, but predict similar values for maximum story drift ratios (Figure 14b). Maximum beam plastic rotations predicted by the two models are similar at most joints. Model B predicts slightly larger beam plastic rotations than Model A. The plots in Figure 14c show the similarity between the two models. Note that about three large inelastic loading cycles are predicted for this record. Model B predicts no yielding in the panel zones, whereas Model A predicts a small amount of inelastic activity in the panel zones (Figure 14d).

For the Lucerne record, Models A and B predict significantly different roof displacement records and significantly different maximum story drift ratios (Figures 15a and 15b). There are also large differences in the predicted beam plastic rotations. At many joints, Model B predicts maximum beam plastic rotations on the order of 50% to 85% greater than for Model A. Both models predict that little or no yielding will occur in the panel zones. Note that this frame sees only one very large cycle of inelastic loading for the Lucerne record.

For the Mexico City record, both models predict very large story drift ratios, on the order of 4%. This soft soil ground motion record has a large peak in its acceleration response spectrum at about a 2 second period, and therefore generates a large response from this frame. As illustrated by Figure 16c, a large number of inelastic loading cycles occurs for this record. The maximum beam plastic rotations predicted by the two models are quite similar for most joints, and are generally within about 20% of each other. The basic character of the beam hysteretic response, however, is quite different for the two models, as illustrated by Figure 16c. Model B predicts no panel zone yielding. Model A, on the other hand, predicts rather substantial panel zone yielding (Figure 16d).

To summarize, the following observations can be made from the dynamic analyses:

- The differences found in structural response predictions for the two models vary among the three records. There appear to be few, if any, consistent trends in how Model A compares with Model B among the three records. This suggests that the significance of varying modelling parameters is earthquake dependent. The simpler baseline model agrees reasonably well with the refined model for the Sylmar and Mexico City records, but showed large differences for the Lucerne record.

- For the Sylmar and Mexico City records, the maximum beam plastic rotations predicted by the two models are quite similar, generally within 25% of each other. For the Lucerne record, there are large differences in the predicted beam plastic rotations, with Model B showing beam plastic rotations up to 85% greater than for Model B.

- Model B showed no panel zone yielding for any of these ground motion records. Model A, on the other hand, predicted some panel zone yielding for each of the records. This yielding was rather significant for the Mexico City record.

3.4 ADDITIONAL COMPARATIVE ANALYSES

In this section, the effect of varying additional model parameters on the predicted structural response is briefly investigated. All analyses reported in this section are for the Sylmar (sylm.000) record.

3.4.1 EFFECT OF VARYING YIELD STRESS IN COLUMNS AND BEAMS

In this section, three variations of the refined inelastic model are considered. The only difference between these three cases is the value of yield stress assumed for the beam, column, and doubler plate. Other than variations in yield stress, all other model parameters are identical for the three cases. The three models therefore have identical elastic stiffness, and identical natural periods of vibration.

The three models compared in this section are A-1, A-2, and A-3 (see Table 2). Model A-1 (same as Model A in Section 3.3) uses a beam yield stress of 47 ksi, and a column (and doubler plate) yield stress of 57 ksi. As noted earlier, these values are based on statistical averages for A36 and A572 Gr. 50 yield stress values from a recent study. Model A-2 uses minimum specified yield stress values, i.e., 36 ksi for A36 beams and 50 ksi for A572 Gr. 50 columns and doubler plates. Finally, Model A-3 considers the case where the beam has a considerably higher yield strength than the column. For this case, a yield stress of 60 ksi was used for the beams, and 50 ksi was used for the columns and doubler plates. Note that 60 ksi is close to the mean yield stress value for ASTM Group 2 shapes of Dual Grade steel. Most of the beams of this frame are classified as Group 2 shapes. The various yield stress values assumed for Models A-1, A-2, and A-3 can be considered plausible for an actual frame.

The response of the three models under the Sylmar record is shown in Figure 17. The roof displacement histories (Figure 17a), and the maximum story drift ratios are similar for the three models. Very large differences, however, are predicted in beam plastic rotations (Figure 17c). As might be expected, Model A-2 (lowest beam yield stress) predicts the highest beam plastic rotations. Similarly, Model A-3 (highest beam yield stress) predicts the lowest beam plastic rotations. The maximum beam plastic rotations vary by more than 200% among the three models at a number of joints. There are even larger differences in the predicted panel zone plastic rotations. (Figure 17d). Note that Model A-2 shows that most of the inelastic deformation at a joint occurs in the beam. Model A-3, on the other hand, shows most of the inelastic action in the panel zone, although beam plastic rotations are still substantial.

This analysis shows that yield stress values assumed in modelling the frame have a very large effect on the predicted plastic rotations. Changing the yield stress values can change the overall inelastic deformation demands at a joint, and can also shift the location of inelastic deformation between the beam, panel zone and column. Note that for the Sylmar record, differences in predicted inelastic response were more significant than the differences between the baseline and refined models (Section 3.3.2).

The comparison of these three models also indicates that the accuracy of an analytical model cannot necessarily be judged by its ability to predict global response parameters such as roof displacement. These three models provided quite similar predictions of roof displacement time history and maximum story drift ratios. Yet, they provide drastically different predictions of local response parameters, such as beam plastic rotations. This observation is significant when comparing the results of model predictions with the measured response of an instrumented building. Just because a model can reasonably match measured global response parameters such as roof displacement or acceleration, provides no assurance that the model is accurately predicts local response.

3.4.2 COMPARISON OF REFINED VERSUS BASELINE BARE STEEL BEAMS

In Section 3.3, the baseline model with a bilinear bare steel beam (Model B) was compared to the refined model with a composite beam (Model A-1). In this section, the baseline model with a bilinear bare steel beam (Model B) is compared with the refined model with a multilinear bare steel beam (Model A-1b). The multilinear bare steel beam element was described in Section 2.2.2. This comparison is intended to highlight the different response predictions that are obtained, even when bare steel beam elements are used for both the refined and baseline models. Such a comparison is of interest when modelling steel moment frames that are constructed without composite slabs.

The response of Models B and A-1b for the Sylmar record is shown in Figure 18. The response of the refined model with composite beams (Model A-1) is also shown for reference. This figure indicates that there are significant differences in the predicted response for the two bare steel beam models. The differences are particularly large in the roof displacement histories (Figure 18a) and in the beam plastic rotation plots (Figure 18c). This observation suggests that when modelling moment frames that actually do not have composite floor slabs, the accuracy of the bare steel beam model has a very significant effect on the predicted response.

3.4.3 EFFECT OF ASSUMED DOUBLER PLATE PARTICIPATION

As described in Section 2, one of the uncertainties in modelling panel zone behavior is the effectiveness of the doubler plate in contributing to the shear stiffness and strength of the panel zone. Based on model calibrations to limited experimental data, the panel zone element used in the refined model assumed 40% doubler plate participation. That is, only 40% of the actual doubler plate thickness was included in the total panel zone thickness. In this section, the sensitivity of the predicted model response to the assumed doubler plate participation is briefly examined.

For Model A-1a, doubler plates were assumed to be 100% effective. That is, the full thickness of the doubler plate was used in computing panel zone stiffness and strength. The response of Model A-1a is compared to Model A-1 (40% doubler plate participation) for the Sylmar record in Figure 19. When considering roof displacement history, maximum story drift ratios, and beam plastic rotations, the response of the two models is nearly identical. There is some difference in the predicted panel zone response (Figure 19d). Overall, however, the response of this model does not appear to be particularly sensitive to the assumed percentage of doubler plate participation in the panel zone. This observation can likely be attributed to the fact that this frame, even with only 40% doubler plate participation, has relatively strong panel zones that forces most of the inelastic deformation into the beam. Thus, making the panel zone even stronger by increasing the assumed doubler plate participation has little influence on the frame behavior. The assumed percentage of doubler plate participation may be considerably more important in frames with relatively weak panel zones.

3.4.4 COMPARISON OF REFINED AND BASELINE MODELS WITHOUT DOUBLER PLATES

In this section, the refined inelastic model is compared with the baseline inelastic model. In both models, however, the doubler plates have been removed. That is, in constructing the panel zone elements, it was assumed that no doubler plates are present. These models are referred to as: "A-ndp" and "B-ndp" (npd = no doubler plate). This analysis is intended to evaluate the difference between the refined and baseline models for frames in which the panel zone plays a more prominent role in the inelastic response of the frame.

The response of Models A-ndp and B-ndp under the Sylmar record in shown in Figure 20. Rather large differences in the predicted response are apparent. Model B-npd predicts that no yielding will occur in the beams. All yielding is predicted to occur in the panel zones. The refined model (A-ndp), on the other hand, gives a substantially different picture of the response. Model A-ndp predicts considerable yielding in the beams. From this comparison, it appears that the difference in predicted response between the refined and baseline models are considerably more significant in frames with somewhat weaker panel zones.

3.5 SUMMARY OF BEAM PLASTIC ROTATIONS AT SELECTED JOINTS

In order to provide an additional perspective on the results of the dynamic analyses, Tables 5 to 7 list the maximum beam plastic rotations developed at selected joints for each analysis case. Beam plastic rotations are reported for the joint on the 2nd floor at Grid Line 3 (Table 5), for the joint on the 3rd floor at Grid Line 4 (Table 6), and for the joint on the 6th floor at Grid Line 4 (Table 7). In each case, the maximum positive and negative plastic rotations are reported. The results from Models A-ndp and B-npd are considered separately from the others. The "ndp" models do not have doubler plates, and so these models essentially represent a different frame than the other six models.

Consider first the models that include doubler plates, i.e., all models except A-ndp and B-ndp. The data in the Tables 5 to 7 support the observations made above. That is, the predicted beam plastic rotations can be quite sensitive to the modelling assumptions. The difference in predicted maximum plastic rotation between models can sometimes exceed 100%. In many cases, however, the agreement between the various models is quite good, particularly when considering the absolute (positive or negative) maximum plastic rotation. The absolute maximum plastic rotation at a joint for the various models is often within .005 rad. The largest difference among the models in absolute maximum plastic rotation at any given joint is on the order of .01 rad. When considering the complete span of plastic rotation at a joint, i.e., the amount of plastic rotation between the maximum negative and maximum positive values, the discrepancy between the models is considerably greater.

Consider next the models for the frame without doubler plates. That is, Models A-ndp and B-ndp. These models were intended to represent a frame in which the panel zones are relatively weaker with respect to the beams. The results in Tables 5 to 7 show some very significant discrepancies between the baseline model (B-npd) and the refined model (A-npd). The baseline model predicted that the yielding at a joint will occur entirely in the panel zone, with the beams remaining essentially elastic. That is, Model B-npd predicts no plastic rotation demands in the beams. Model A-ndp, on the other hand, predicts rather substantial plastic rotation demands on the beams, up to .015 rad at the 6th floor (Table 7). The baseline model appears to substantially underestimate the strength of the panel zone (see Figure 10), and therefore overestimates its participation in developing inelastic deformations at a joint. Thus, for frames with relatively weak panel zones, the use of the simpler baseline model can be quite misleading.

4.0 SUMMARY AND CONCLUSIONS

4.1 SUMMARY

This report has summarized the results of a brief study examining the effects of various modelling assumptions on the predicted inelastic response of a steel moment frame. The objective of the study was to determine how sensitive the predicted inelastic response of a moment frame may be to the assumptions used in developing the structural model. A variety of computer models were constructed for a six story single bay steel moment frame. Two basic classes of models were considered: "refined" models, and a "baseline" model. The baseline model used simple bilinear representations of inelastic response, and modeled bare steel behavior only (no composite floor). The refined model employed more realistic hardening rules that better represent experimentally observed inelastic behavior, and included composite floor slab effects in modelling both the beams and panel zones. In addition to comparing the refined model to the baseline model, variations in other modelling parameters were also considered, most notably, the value of yield stress assumed for the beams, columns, and doubler plates. The response of the models to four different loading cases were considered: a static pushover analysis and three strong ground motion records.

Key findings of this study are summarized below:

- Differences up to 20% in the predicted first natural period of the frame were obtained due to the different elastic stiffness of the models. A primary factor affecting the elastic stiffness of the models was the inclusion or exclusion of composite floor slab effects. Thus, the differences in the predicted response of the various models for the same ground motion is due both to differences in modelling inelastic behavior, as well as to differences in structural demands arising from the different natural periods of the models.

- The refined model was compared to the baseline model in a static pushover analysis. For the same story drift ratio, the two models predicted approximately the same total rotation (beam + panel zone) at most of the frame joints. However, the distribution of total plastic rotation at a joint between the beam and panel zone was significantly different for the two models. Consequently, for the same story drift ratio, the two models predicted significantly different beam plastic rotation demands.

- The refined model was compared to the baseline model for three strong motion records: Sylmar, Lucerne, and Mexico City. For the Sylmar and Mexico City records, the maximum beam plastic rotations were generally quite similar, typically within 25% of one another. For the Lucerne record, the differences between the models were more substantial. Thus, the sensitivity of the predicted response to variations in modelling assumptions appears to be ground motion dependent. The response of the baseline model may be quite similar to the refined model for one record, but may be very different for another record.

- The effect of varying the assumed value for yield stress in the beams and columns of the refined model was examined for the Sylmar record. The predicted inelastic response was very sensitive to this variable. Changing the assumed yield stress values changed the overall inelastic deformation demands at a joint, and also shifted the location of inelastic deformation between the beam, panel zone, and column.

- The comparison of the three models with varying yield stress values indicates that the accuracy of an analytical model cannot necessarily be judged by its ability to predict global response parameters such as roof displacement. These three models provided quite similar predictions of roof displacement time history and maximum story drift ratios. Yet, they provided drastically different predictions of local response parameters, such as beam plastic rotations. This observation is significant when comparing the results of model predictions with the measured response of an instrumented building. Just because a model can reasonably match measured global response parameters such as roof displacement or acceleration, provides no assurance that the model is accurately predicts local response.

- The refined model was compared to the baseline model for the frame with the doubler plates eliminated. This analysis was intended to evaluate the effect of modelling assumptions for frames where the panel zone plays a more prominent role in developing inelastic deformation at a joint. The difference in predicted response of the two models was much larger than for the frame with doubler plates. For the frame without doubler plates, the baseline model predicted that the beams would remain elastic. The refined model, on the other hand, predicted rather substantial beam plastic rotation demands. Thus, the predicted beam plastic rotation demands are particularly sensitive to modelling assumptions for frames with relatively weak panel zones. Such frames have been permitted since the 1988 UBC. Note that this analysis does not indicate that frames with weak panel zone designs are necessarily better or worse than frames with very strong panel zones.

4.2 CONCLUSIONS

Some of the major conclusions of this study can be summarized as follows:

- The results of this study indicate that the predicted inelastic dynamic response of a steel moment frame is quite sensitive to modelling assumptions. Key inelastic response parameters, such as predicted beam plastic rotation demands, can change by more than 100% as model parameters are varied.

- The predicted response of a steel moment frame can be significantly affected by the presence of a composite floor slab. Neglecting composite floor slab effects can substantially underestimate the stiffness and strength of the frame, and alter the predicted plastic rotation demands.

- The predicted inelastic response of a steel moment frame is particularly sensitive to the assumed value of yield stress for the beams and columns of the model.

- Varying the model assumptions appears to have a very important effect on the predicted location of yielding within a joint. This is particularly true for frames in which the panel zone is relatively weak compared to the beam. Changing the modelling assumptions can cause the predicted location of yielding at a joint to shift almost completely between the beam and panel zone.

- In general, the results of this study indicate that varying modelling assumptions has a substantially greater impact on local response predictions such as beam plastic rotation, than on global response predictions such as interstory drift ratio. Thus, the accuracy of the model is much more important if local member and joint response predictions are of interest. The accuracy of the model is less important if only global response predictions, such as interstory drift or roof displacement, are of interest.

Inelastic dynamic analysis can be a valuable tool for developing an improved understanding of how a steel moment frame may respond to a strong earthquake. Such an analysis, guided by considerable judgement, can be used to estimate approximate plastic rotation demands at critical locations in a steel moment frame. However, the analyst must recognize that significant uncertainties exist in modelling inelastic behavior. Plastic rotation demands cannot be predicted with great precision and the potential for considerable variability in plastic rotation demands must be recognized when making design decisions. Recognizing this uncertainty is particularly important when making decisions regarding the acceptability of particular beam-to-column connection details based on predicted beam plastic rotation demands. A decision to use a connection detail with very limited plastic rotation capacity, based on the fact that an inelastic dynamic analysis showed low beam plastic rotation demands, should be approached with great caution. Whereas one model for a frame may show that the beams have zero plastic rotation demand (because all of the yielding is predicted to occur in the panel zone), another model of the same frame may show substantial plastic rotation demands in the beams. Greater confidence in the predicted plastic rotation demands may be possible by the use of more accurate analytical models. However, even with the most sophisticated and accurate analytical models, unpredictable variations in the yield stress of beams and columns can still cause large variations in predicted plastic rotation demands.

Finally, it is noted that the above conclusions are based on a very limited number of analyses on a single moment frame. This study was not intended to be exhaustive nor thorough. Rather, this was intended to be a brief, preliminary study on some of the uncertainties involved with inelastic analysis of steel moment frames. The results of this study also suggest possible topics for further study:

- The ability to model the effects of a composite concrete floor slab on the response of beam and panel zone elements is still quite limited. Further work is needed on developing accurate analytical models that include composite slab effects. Additional experimental data are needed on subassemblages with composite slabs in order to calibrate and verify such models.

- Experiments indicate that at larger levels of plastic rotation, beam strength and stiffness degrades due to local buckling. This degradation was not included in the models considered in this study. Analytical models are needed to simulate this effect, particularly for composite beams.

- The models considered in this study did not include the contribution of gravity load framing (framing with "simple" beam-column connections) towards the stiffness and strength of the system. Developing realistic assessments of building response in earthquakes requires the development of such models.

- Models that mimic the response of damaged moment frame joints are needed to evaluate the consequences of such damage.

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TABLE 1 - WEIGHTS USED TO COMPUTE SEISMIC MASS

Floor Level	Weight (kips)
Roof	273
6th	296
5th	318
4th	350
3rd	355
2nd	341
TOTAL	1933

TABLE 2 - SUMMARY OF ANALYTICAL MODELS

Model	Element Types	Beam : Bare steel VS. Composite	Beam Yield Stress (ksi)	Column Yield Stress (ksi)	% Doubler plate participation
A or A-1	refined	composite	47	57	40
A-2	refined	composite	36	50	40
A-3	refined	composite	60	50	40
A-1a	refined	composite	47	57	100
A-1b	refined	bare steel	47	57	40
A-ndp	refined	composite	47	57	0
B	baseline	bare steel	47	57	100
B-ndp	baseline	bare steel	47	57	0

TABLE 3 - SUMMARY OF ANALYSES

MODEL	LOADING CASE			
	Static Pushover	SYLM.000	LUC.270	SCT-1 N90E
A or A-1	X	X	X	X
A-2		X		
A-3		X		
A-1a		X		
A-1b		X		
A-ndp		X		
B	X	X	X	X
B-npd		X		

TABLE 4 - NATURAL PERIODS OF VIBRATION

MODEL	NATURAL PERIOD OF VIBRATION (SEC.)			
	T_1	T_2	T_3	T_4
A or A-1	1.876	.688	.379	.251
A-2	1.876	.688	.379	.251
A-3	1.876	.688	.379	.251
A-1a	1.817	.668	.370	.247
A-1b	2.186	.811	.435	.280
A-ndp	1.955	.715	.391	.256
B	2.139	.797	.430	.277
B-ndp	2.323	.855	.456	.289

TABLE 5 - MAXIMUM BEAM PLASTIC ROTATIONS
JOINT AT 2nd FLOOR - GRID LINE 3

MODEL	Ground Motion Record		
	SYLM.000	LUC.270	SCT-1 N90E
A or A-1	+ .015 - .007	+ .011 - 0	+ .032 - .010
A-2	+ .020 - .006		
A-3	+ .008 - .002		
A-1a	+ .016 - .009		
A-1b	+ .019 - 0		
B	+ .018 - .009	+ .016 - 0	+ .028 - .007
A-ndp	+ .009 - .001		
B-npd	+ 0 - 0		

TABLE 6 - MAXIMUM BEAM PLASTIC ROTATIONS
JOINT AT 3rd FLOOR - GRID LINE 4

MODEL	Ground Motion Record		
	SYLM.000	LUC.270	SCT-1 N90E
A or A-1	+ .012 - .011	+ 0 - .010	+ .028 - .018
A-2	+ .010 - .016		
A-3	+ .010 - 0		
A-1a	+ .012 - .012		
A-1b	+ 0 - .018		
B	+ .008 - .016	+ 0 - .019	+ .010 - .030
A-ndp	+ .009 - 0		
B-npd	+ 0 - 0		

TABLE 7 - MAXIMUM BEAM PLASTIC ROTATIONS
JOINT AT 6th FLOOR - GRID LINE 4

MODEL	Ground Motion Record		
	SYLM.000	LUC.270	SCT-1 N90E
A or A-1	+ .017 - .009	+ 0 - .010	+ 0 - .011
A-2	+ .018 - .010		
A-3	+ .015 - 0		
A-1a	+ .017 - .009		
A-1b	+ .016 - .020		
B	+ .013 - .014	+ 0 - .010	+ .006 - .014
A-ndp	+ .015 - 0		
B-npd	+ 0 - 0		

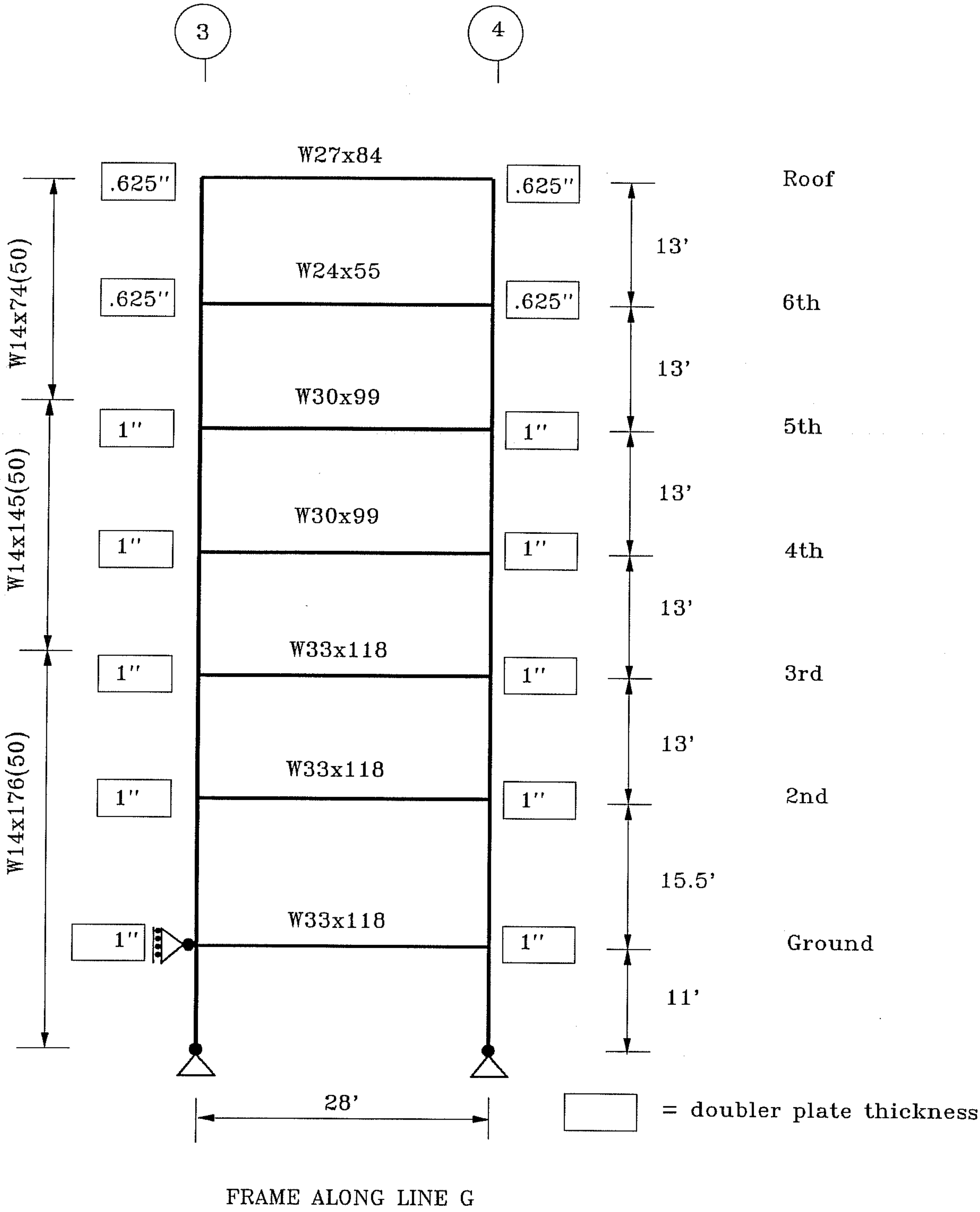
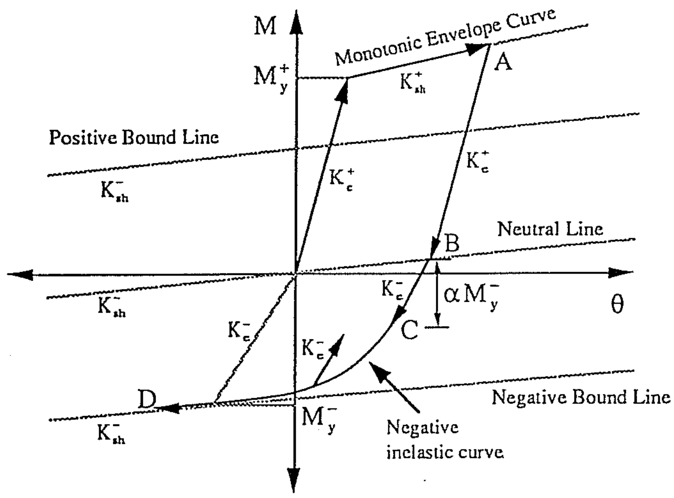
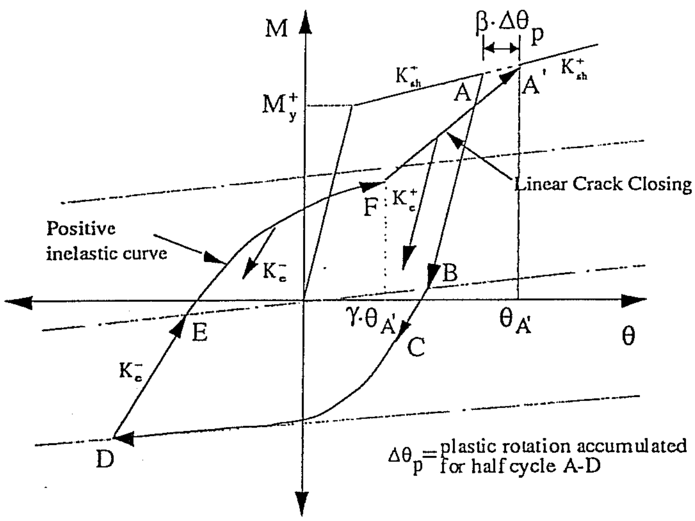


FIGURE 1 - ELEVATION OF FRAME USED FOR PARAMETRIC STUDIES



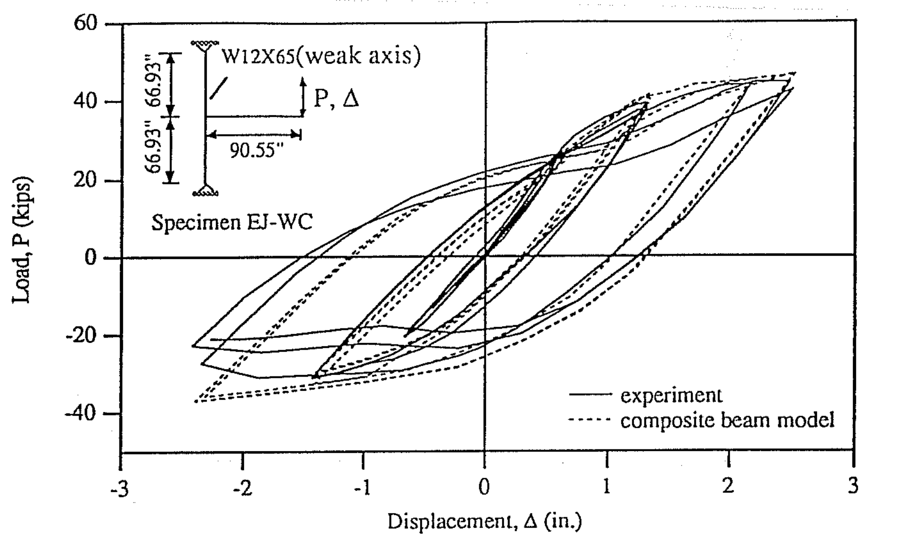
a) Stiffness Degradation for Negative Moment

$\alpha = 0.5$
$\gamma = 0.2$
$\beta = 0.05$
$K_{SH}^+ = .025\,K_E^+$
$K_{SH}^- = .05\,K_E^-$

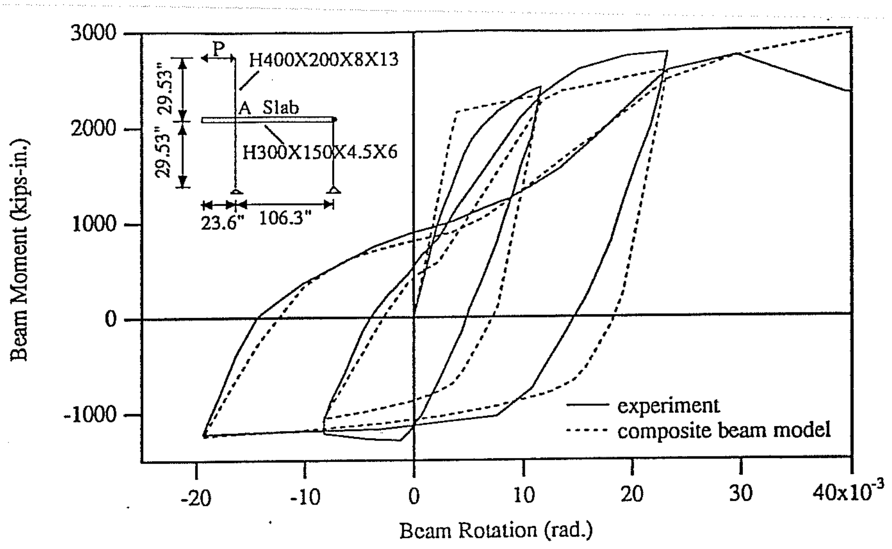


b) Stiffness Degradation and Pinching for Positive Moment

FIGURE 2 - HYSTERETIC RESPONSE MODEL FOR COMPOSITE BEAM ELEMENT

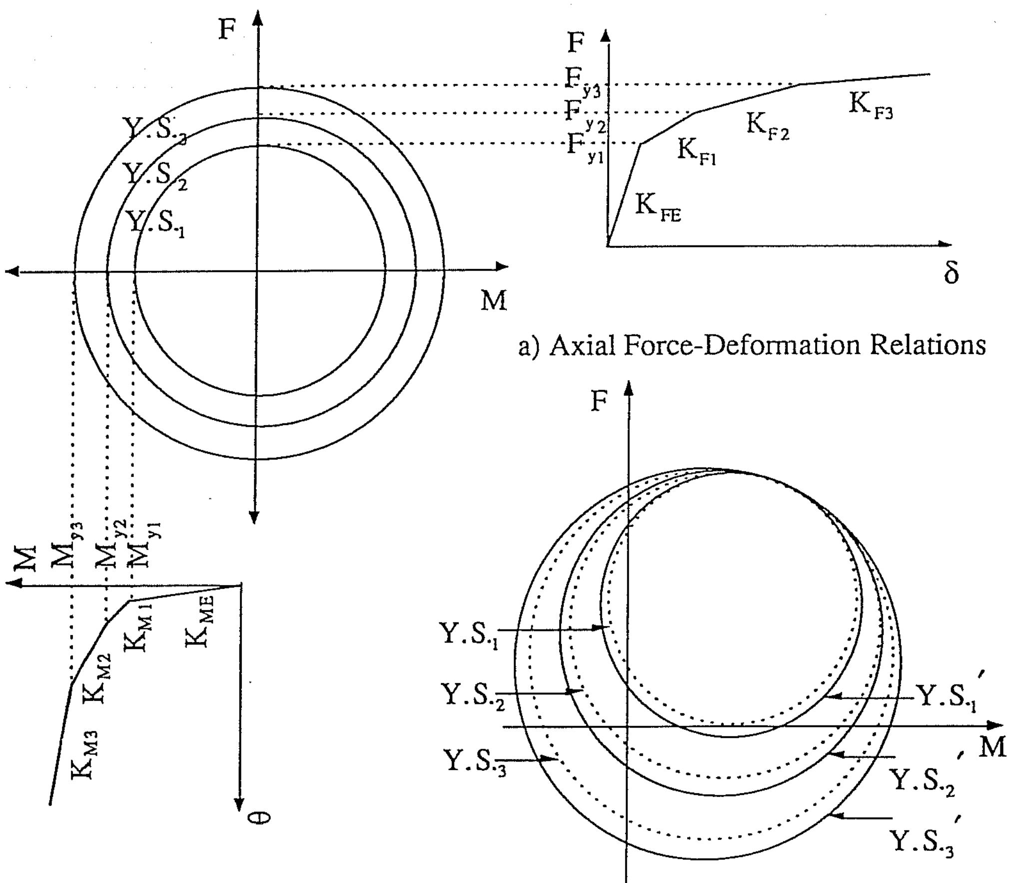


(a) Experiment by Lee (1987)



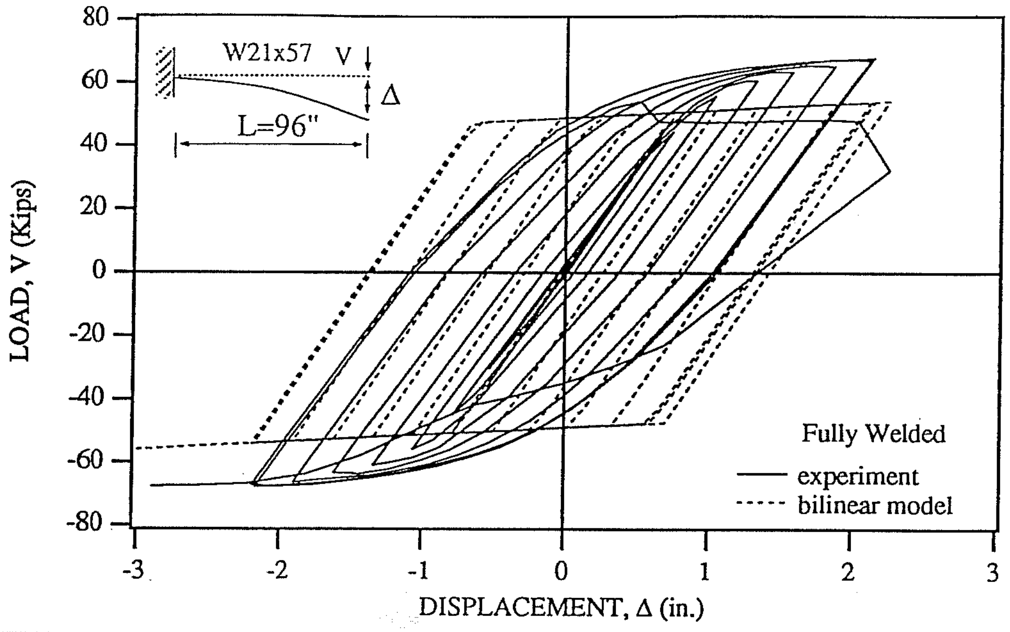
(b) Experiment by Tagawa (1986)

FIGURE 3 - COMPARISON OF COMPOSITE BEAM ELEMENT PREDICTIONS WITH EXPERIMENTAL RESULTS

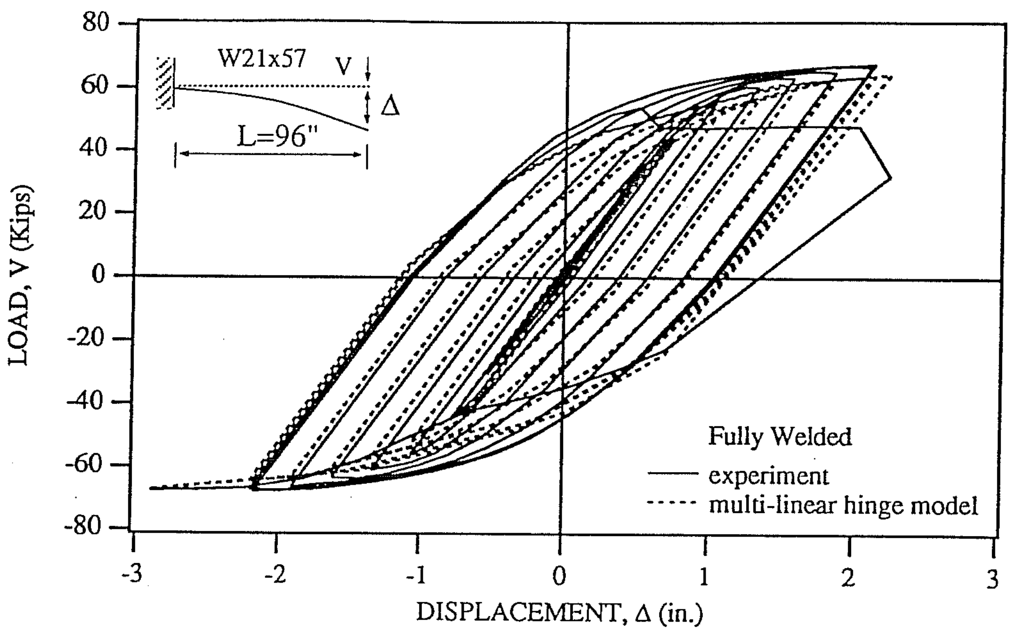


b) Moment-Rotation Relations c) Translation and Expansion of Yield Surfaces

FIGURE 4 - QUALITATIVE RESPONSE OF BARE STEEL BEAM-COLUMN ELEMENT

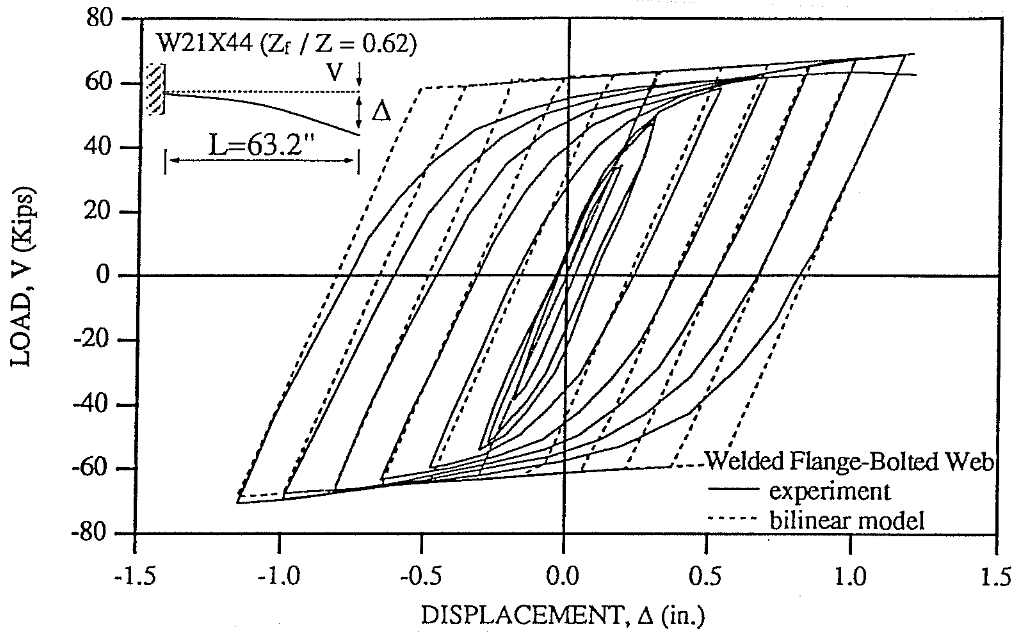


(a) Baseline Bilinear Element

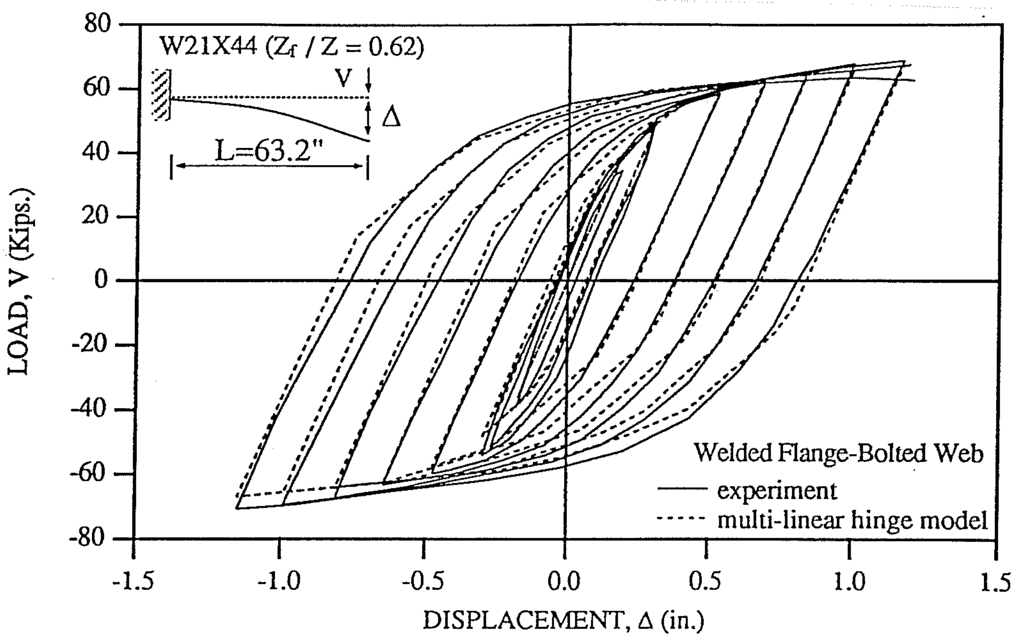


(b) Refined Multi-Linear Element

FIGURE 5 - COMPARISON OF BARE STEEL MODEL PREDICTIONS WITH EXPERIMENTAL RESULTS BY ENGELHARDT AND HUSAIN (1992) - BEAM WITH AN ALL-WELDED CONNECTION

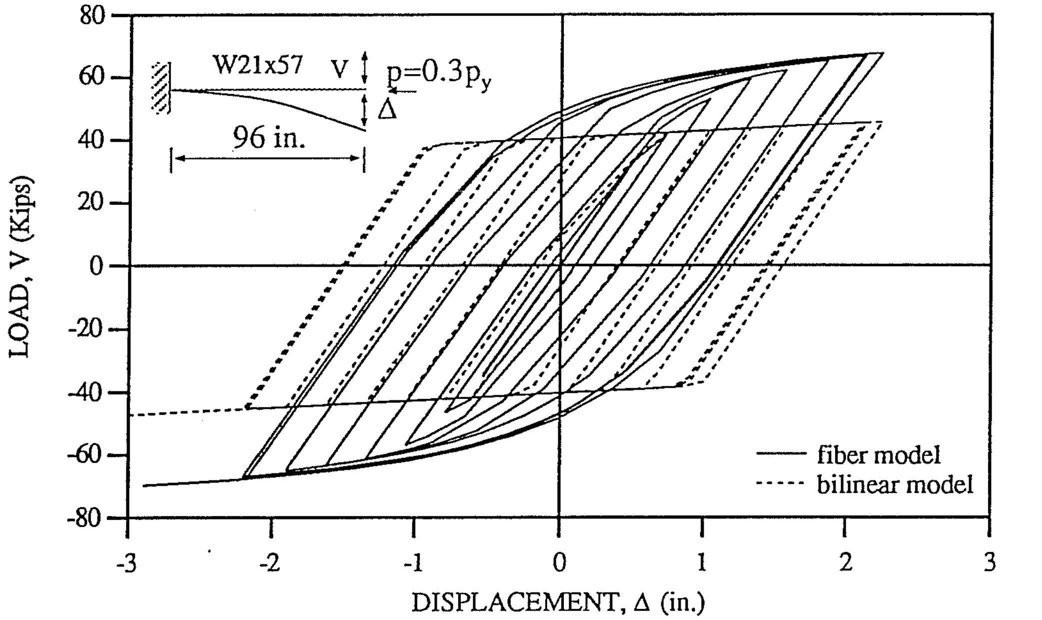


(a) Baseline Bilinear Element

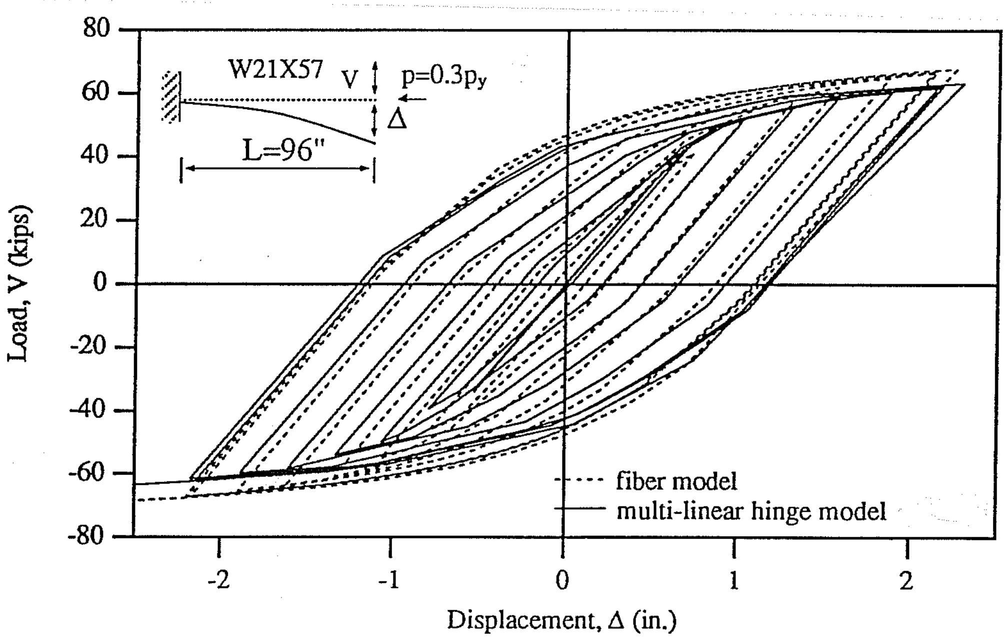


(b) Refined Multi-Linear Element

FIGURE 6 - COMPARISON OF BARE STEEL MODEL PREDICTIONS WITH EXPERIMENTAL RESULTS BY TSAI AND POPOV (1988) - BEAM WITH WELDED FLANGE-BOLTED WEB CONNECTION



(a) Baseline Bilinear Element



(b) Refined Multi-Linear Element

FIGURE 7 - COMPARISON OF BARE STEEL MODEL PREDICTIONS WITH FIBER ELEMENT PREDICTIONS - MEMBER SUBJECT TO CONSTANT AXIAL FORCE AND CYCLIC BENDING

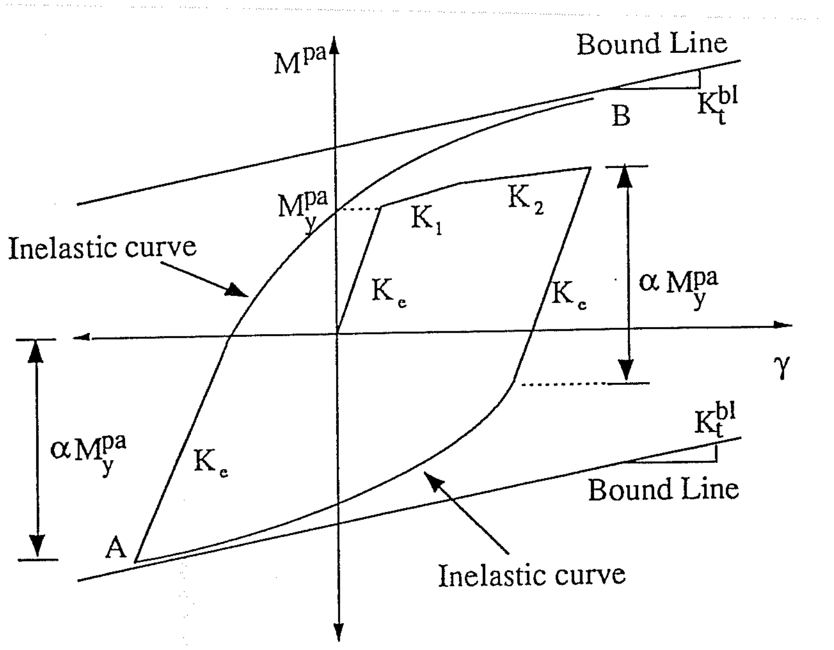


FIGURE 8 - HYSTERETIC RESPONSE MODEL FOR REFINED PANEL ZONE ELEMENT

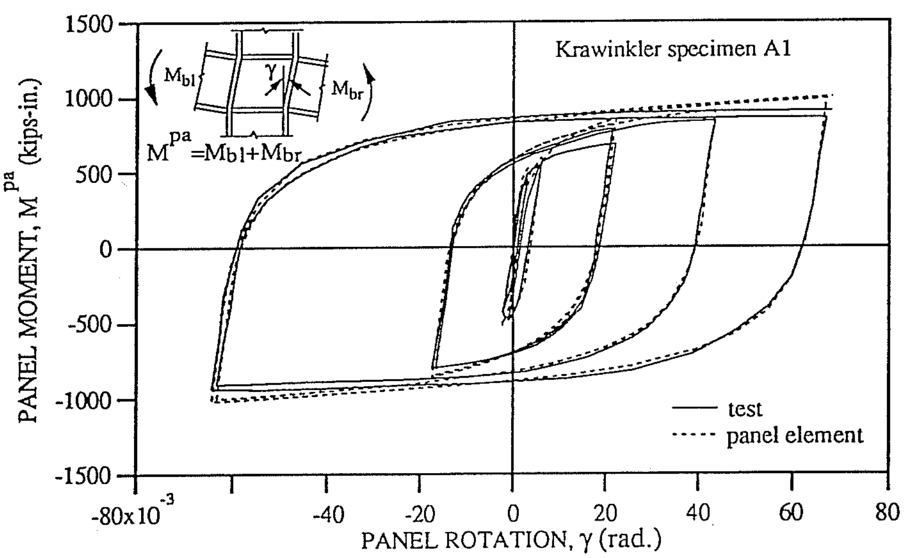


FIGURE 9 - COMPARISON OF REFINED PANEL ZONE MODEL PREDICTIONS WITH EXPERIMENT BY KRAWINKLER (1971)

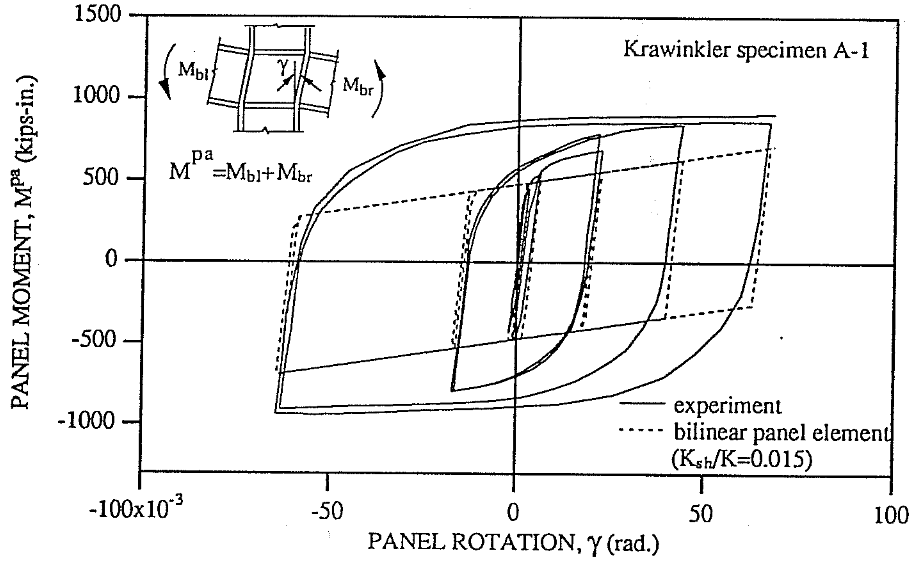


FIGURE 10 - COMPARISON OF BILINEAR PANEL ZONE MODEL PREDICTIONS WITH EXPERIMENT BY KRAWINKLER (1971)

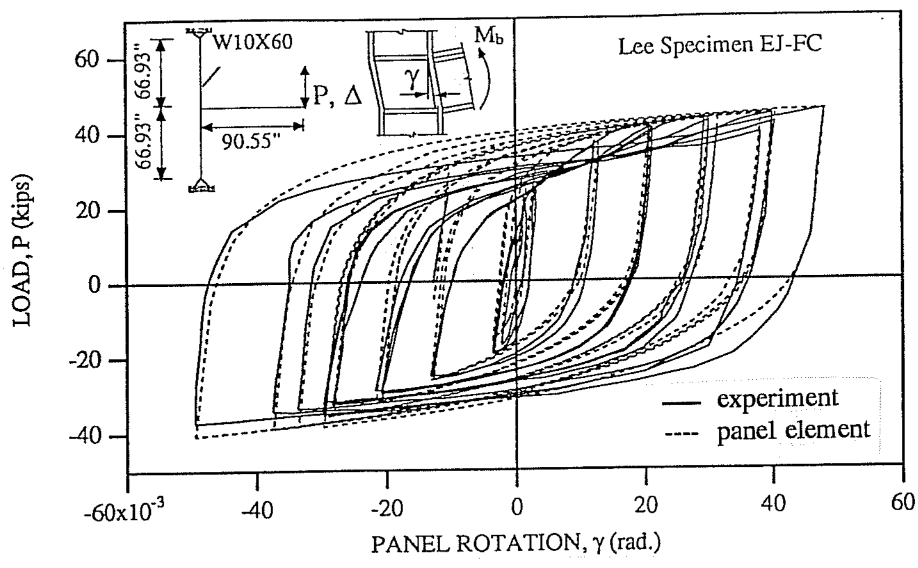
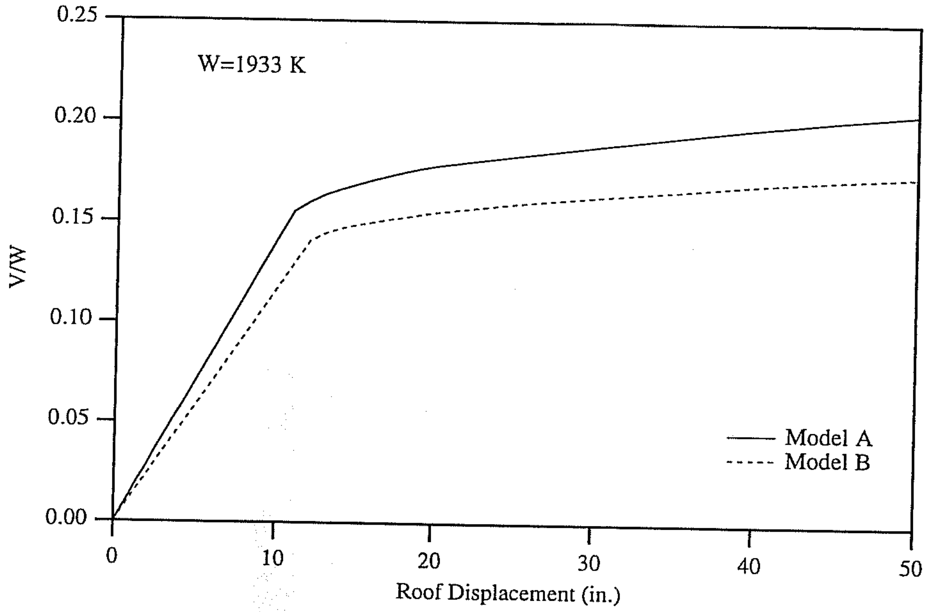
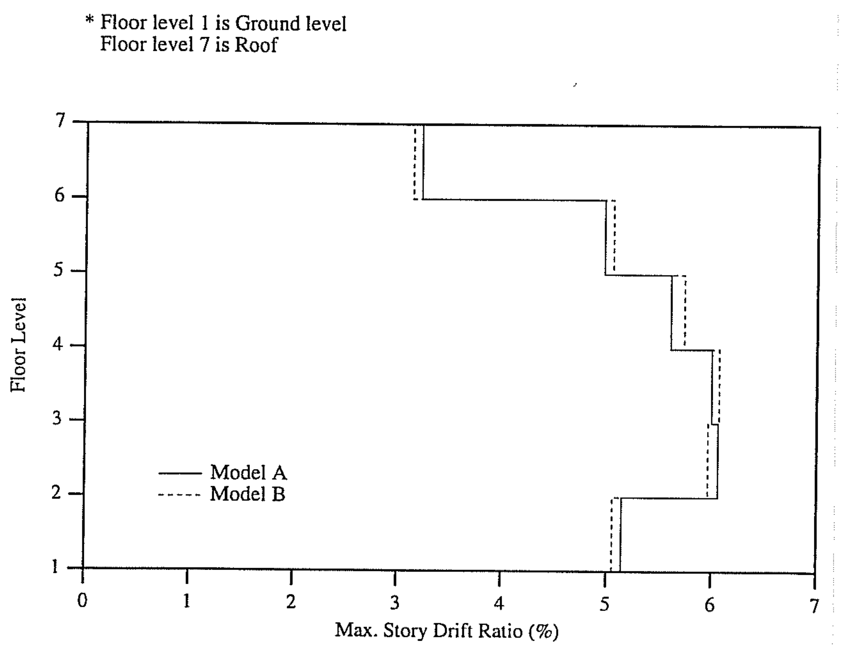


FIGURE 11 - COMPARISON OF REFINED PANEL ZONE MODEL PREDICTIONS WITH EXPERIMENT BY LEE (1987) - SPECIMEN WITH COMPOSITE SLAB

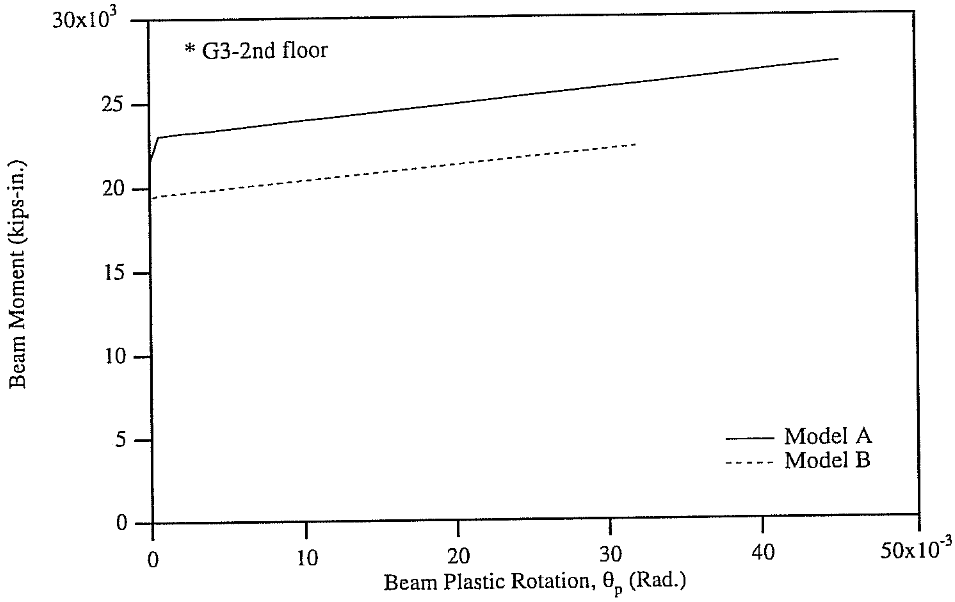


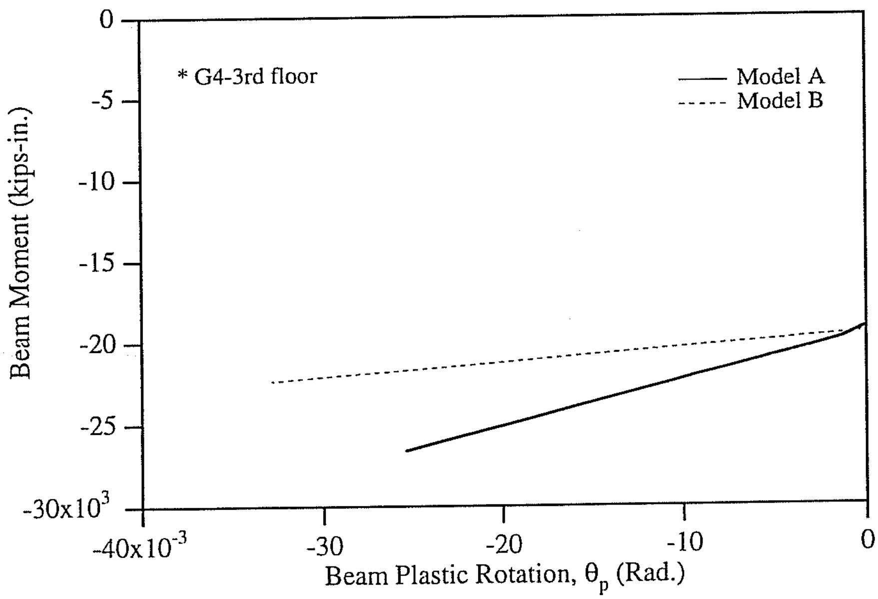
(a) Roof Displacement Versus Base Shear Coefficient



(b) Envelope of Maximum Story Drift Ratios

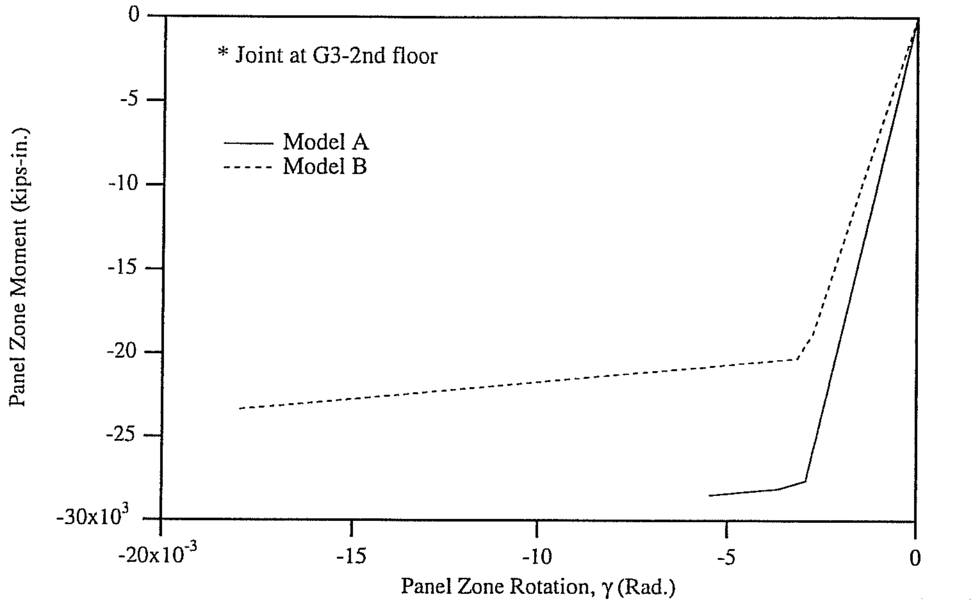
FIGURE 12 - MODEL A VERSUS MODEL B: STATIC PUSHOVER ANALYSIS

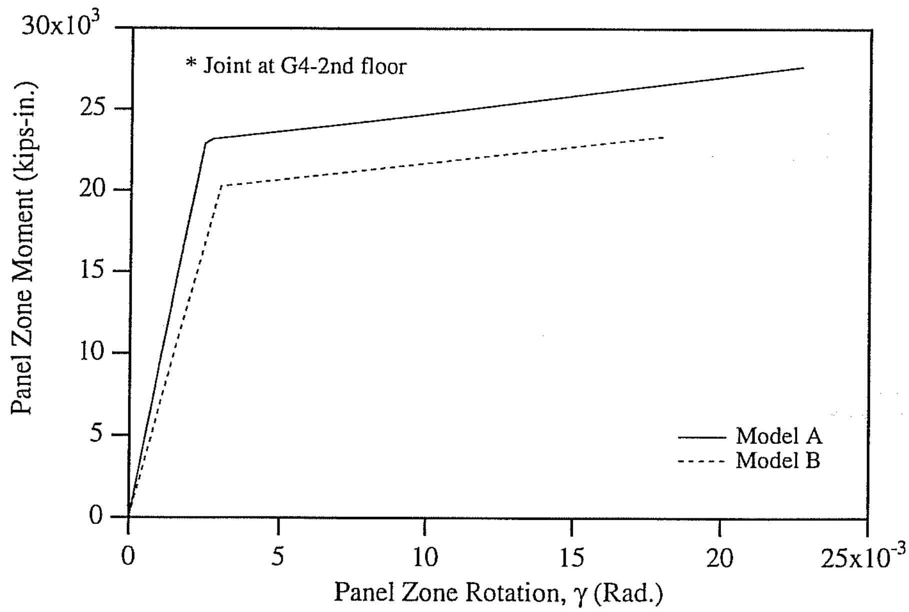




(c) Beam Moment versus Plastic Rotation at Selected Joints

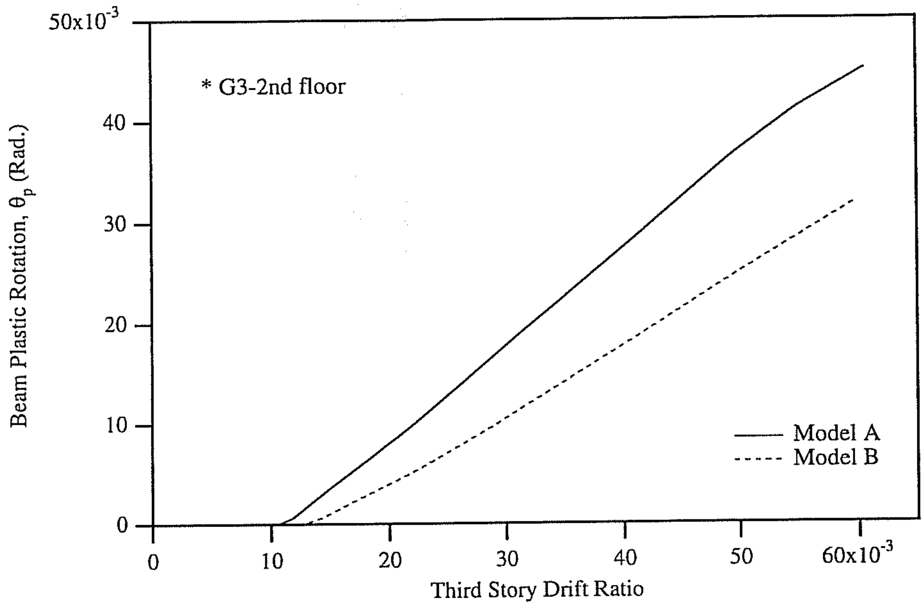
FIGURE 12 - MODEL A VERSUS MODEL B: STATIC PUSHOVER ANALYSIS (CONT.)



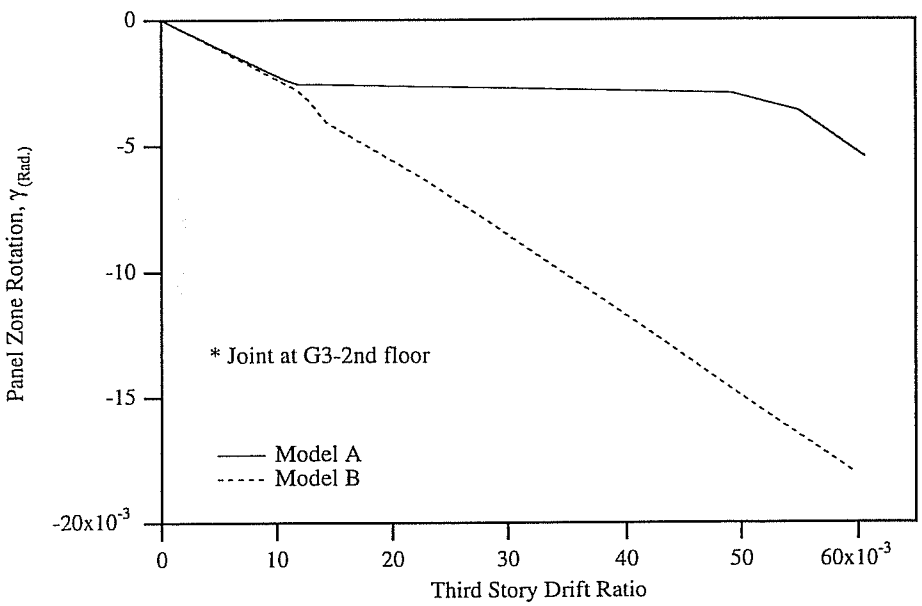


(d) Panel Zone Response at Selected Joints

FIGURE 12 - MODEL A VERSUS MODEL B: STATIC PUSHOVER ANALYSIS (CONT.)

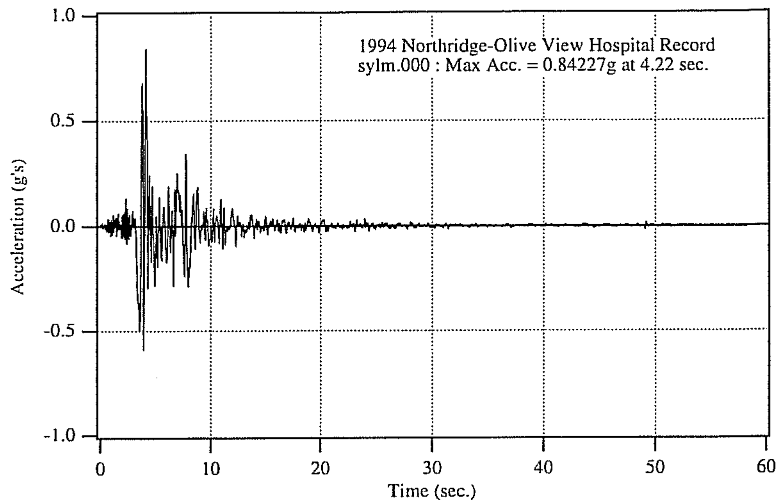


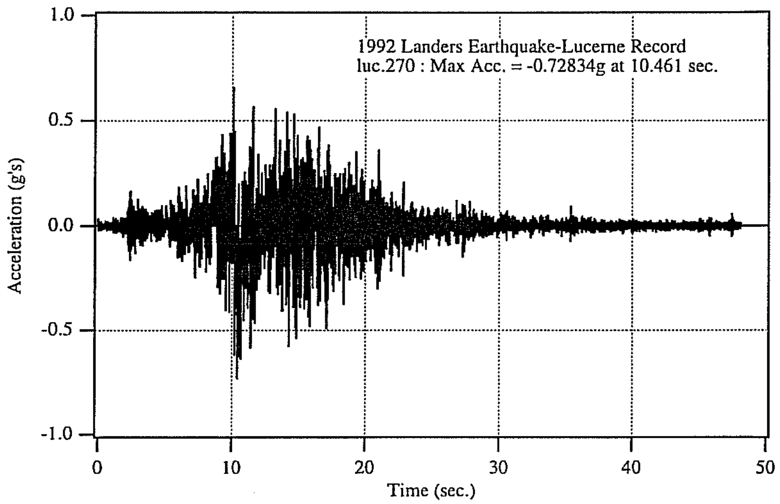
(e) Beam Plastic Rotation versus Story Drift Ratio



(f) Panel Zone Rotation versus Story Drift Ratio

FIGURE 12 - MODEL A VERSUS MODEL B: STATIC PUSHOVER ANALYSIS (CONT.)





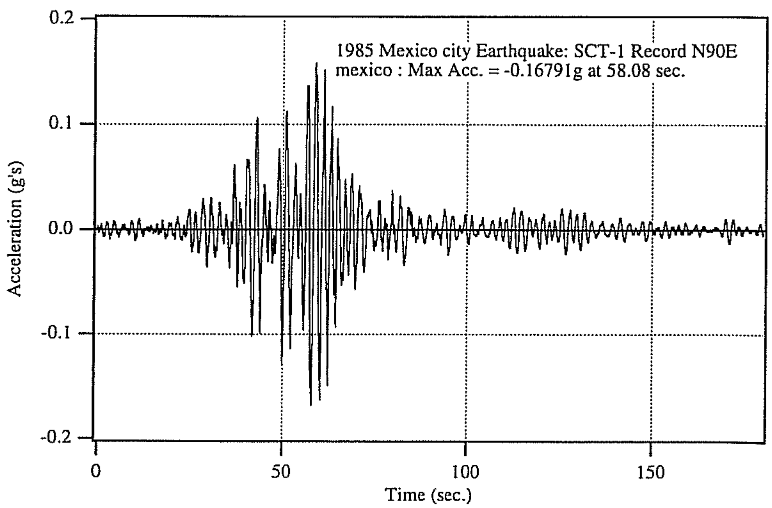
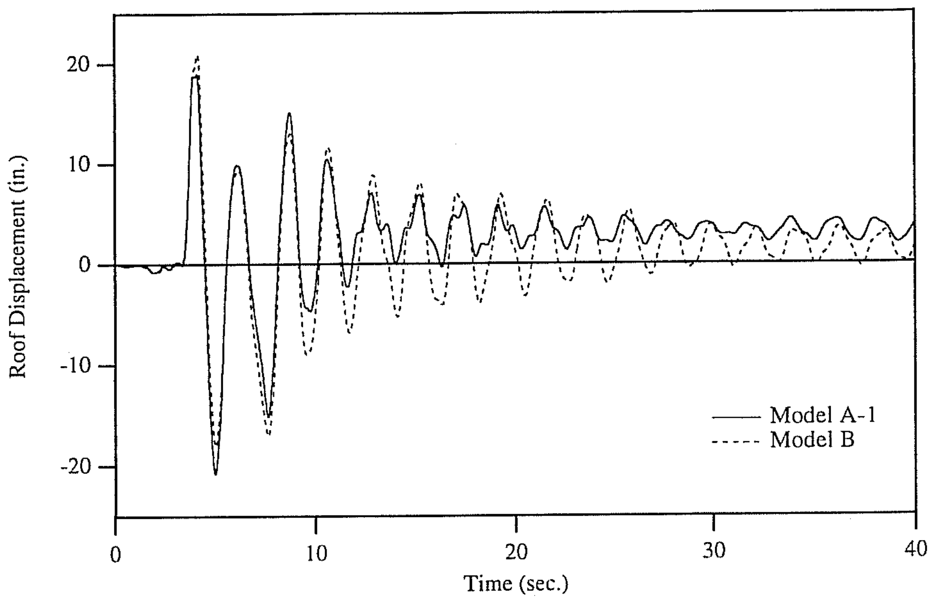
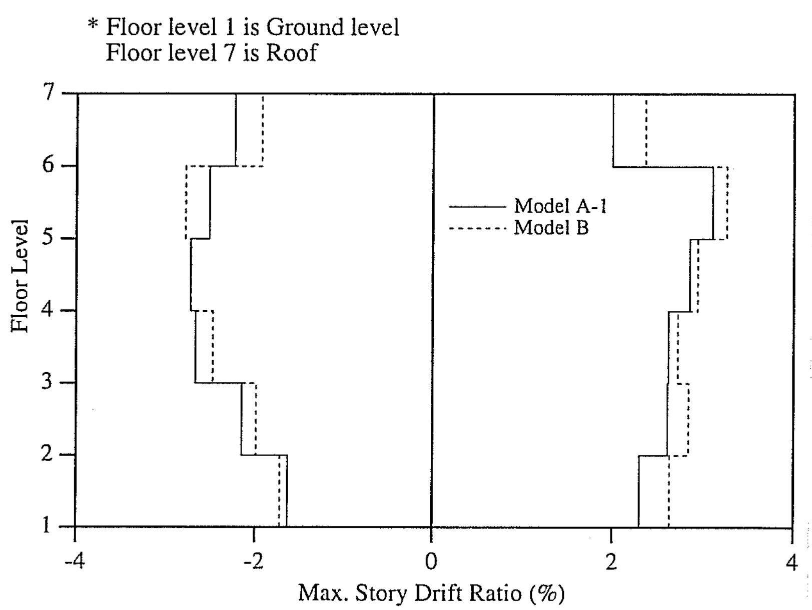


FIGURE 13 - GROUND MOTION RECORDS

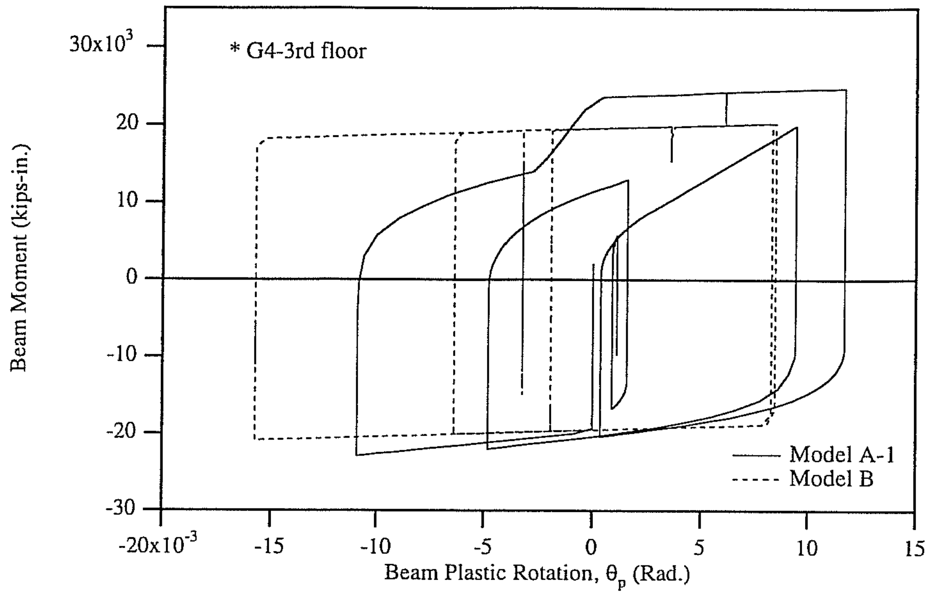


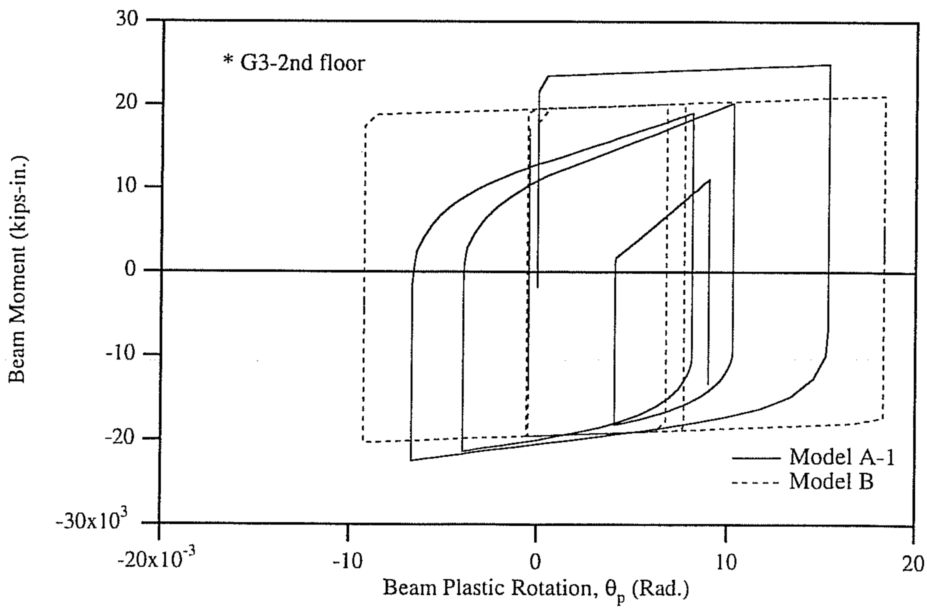
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

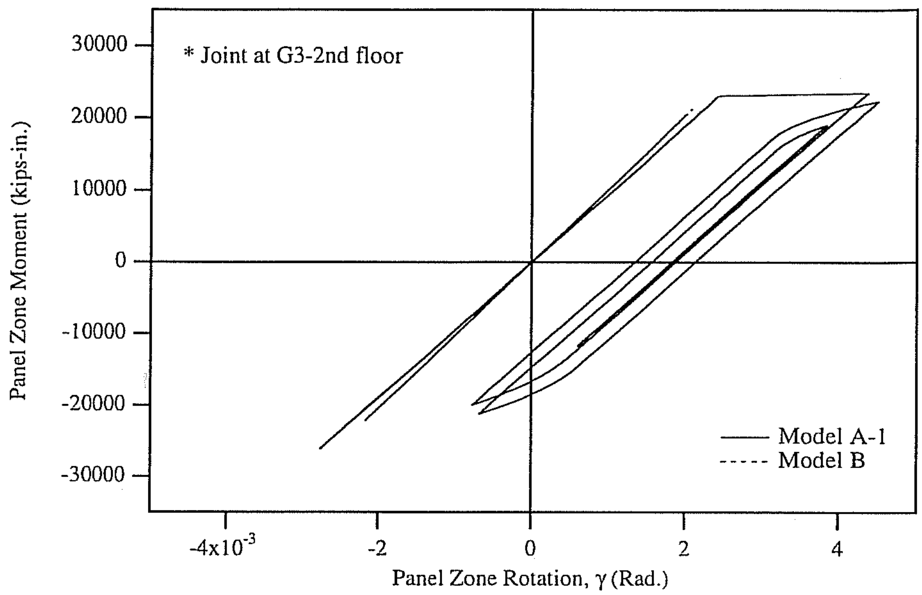
FIGURE 14 - MODEL A VERSUS MODEL B: SYLM.000

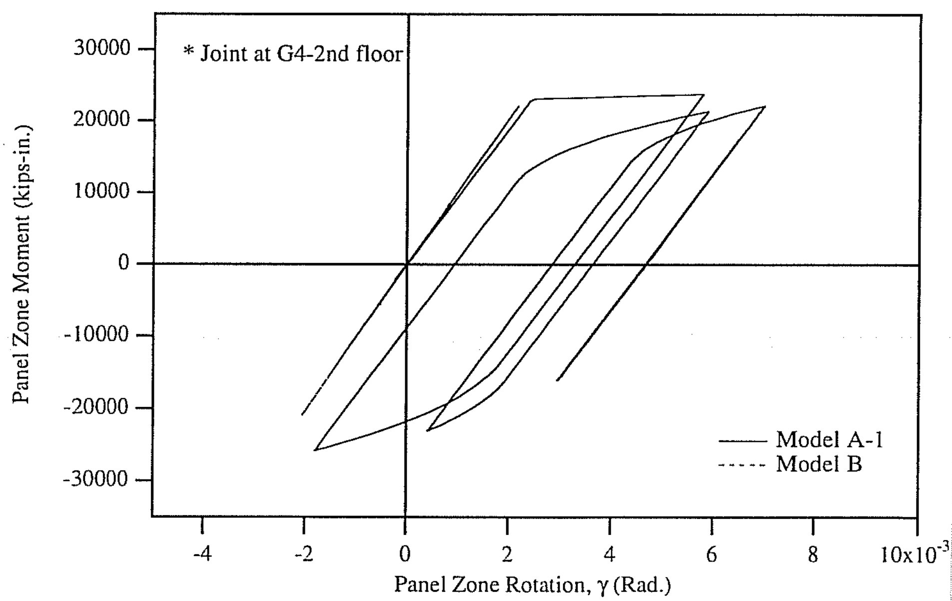




(c) Beam Moment versus Plastic Rotation at Selected Joints

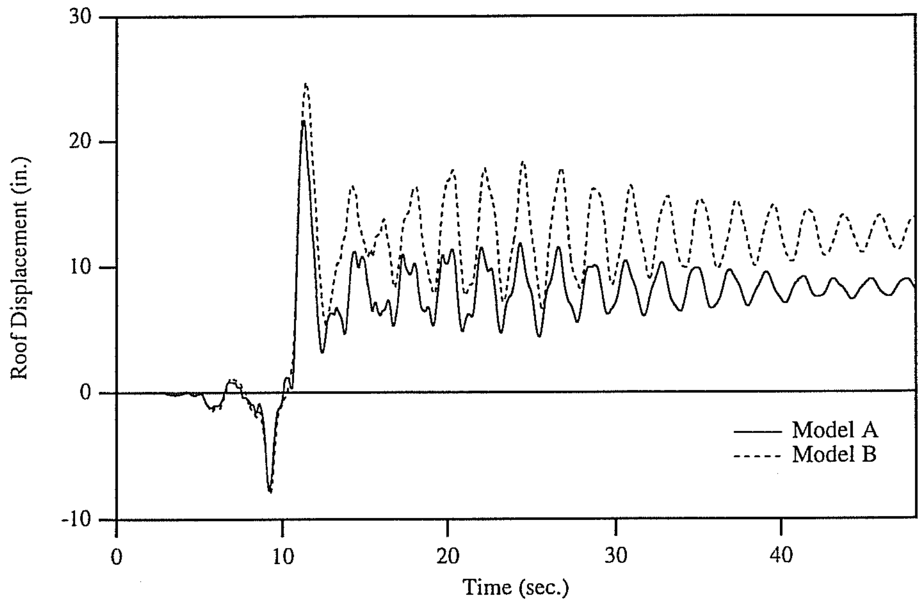
FIGURE 14 - MODEL A VERSUS MODEL B: SYLM.000 (CONT.)



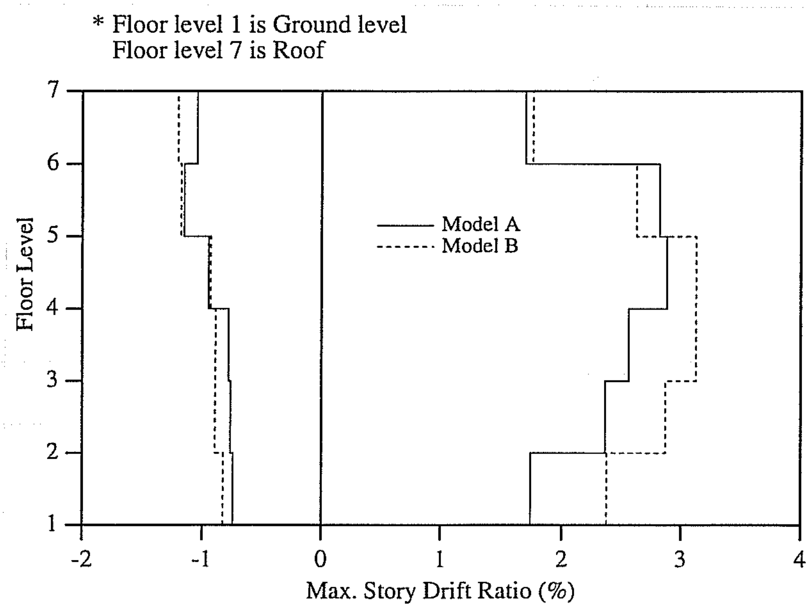


(d) Panel Zone Response at Selected Joints

FIGURE 14 - MODEL A VERSUS MODEL B: SYLM.000 (CONT.)

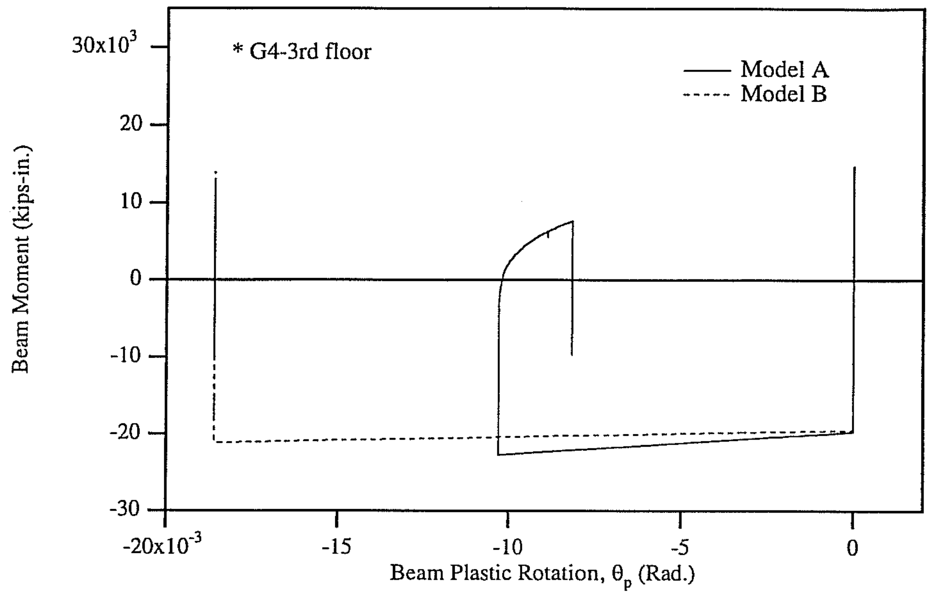


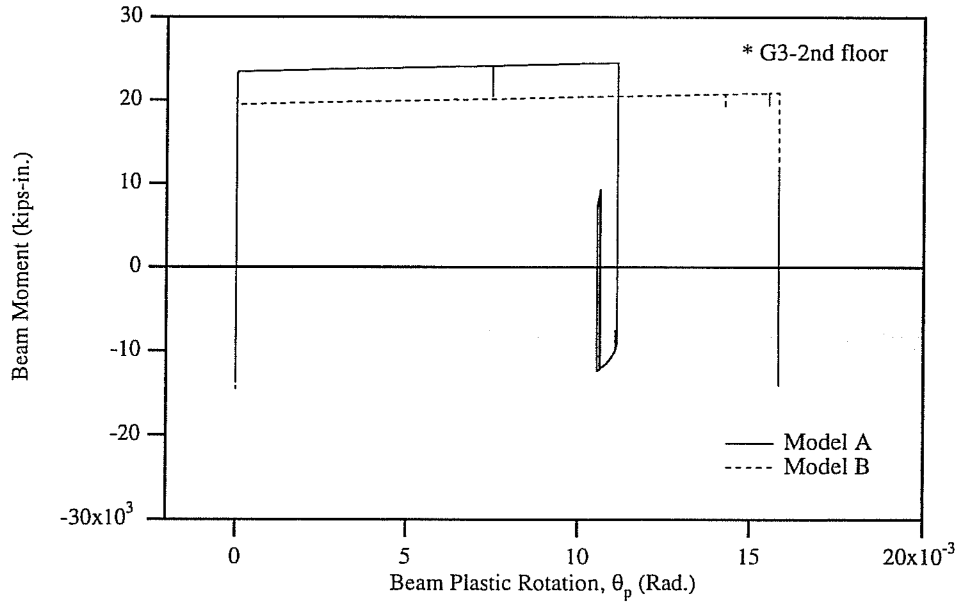
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

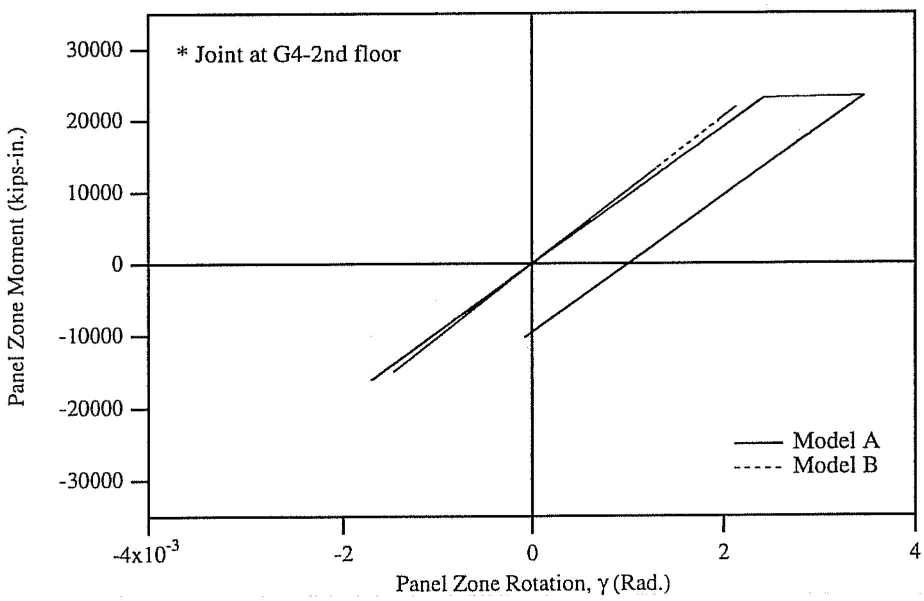
FIGURE 15 - MODEL A VERSUS MODEL B: LUC.270





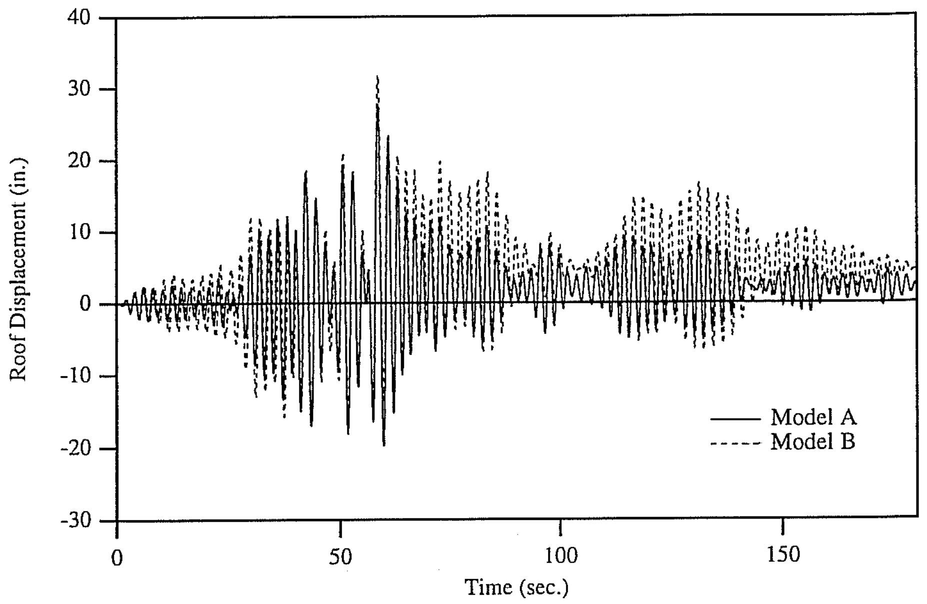
(c) Beam Moment versus Plastic Rotation at Selected Joints

FIGURE 15 - MODEL A VERSUS MODEL B: LUC.270 (CONT.)

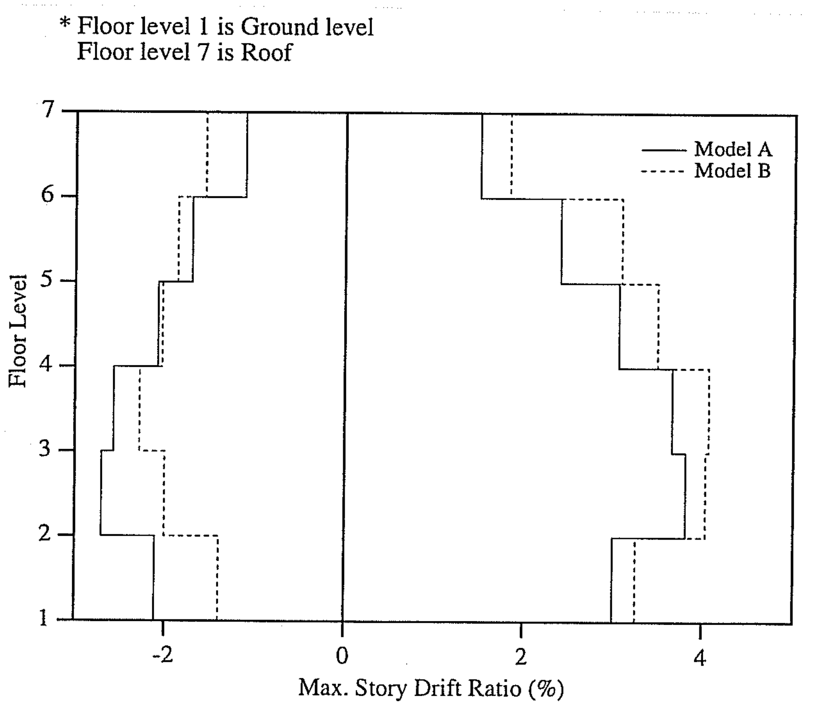


(d) Panel Zone Response at Selected Joints

FIGURE 15 - MODEL A VERSUS MODEL B: LUC.270 (CONT.)

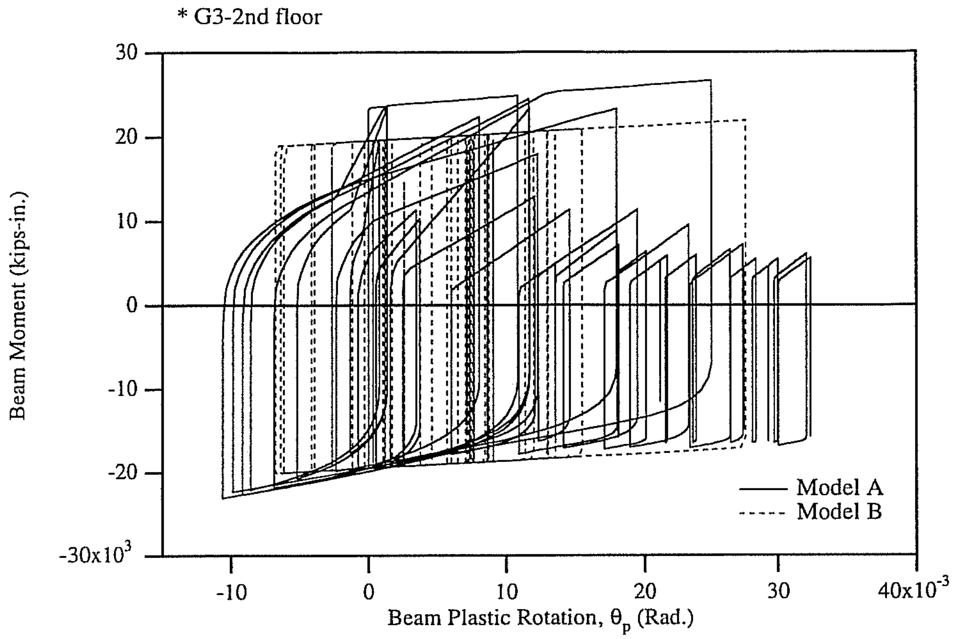


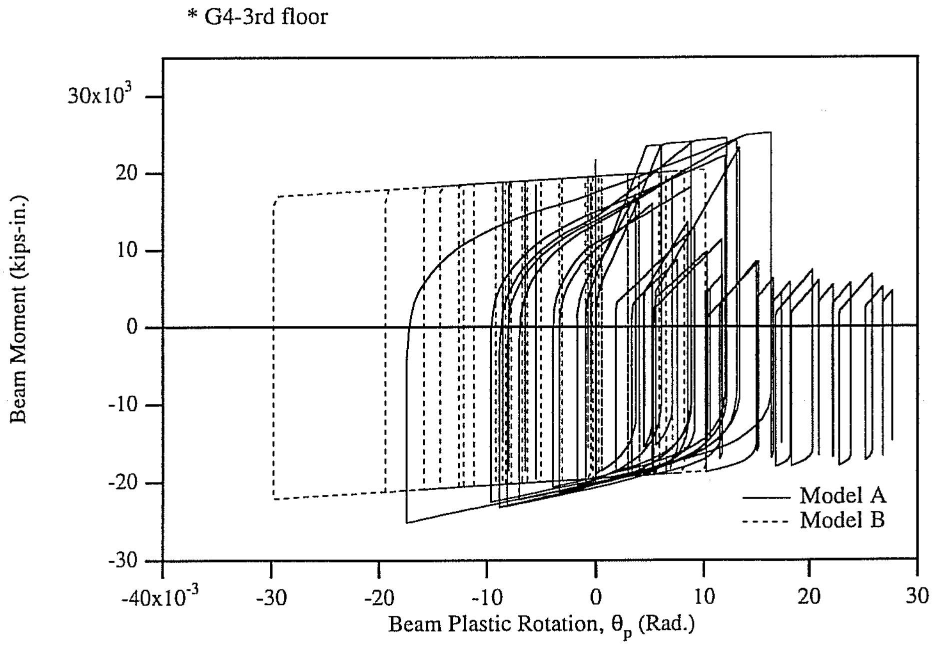
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

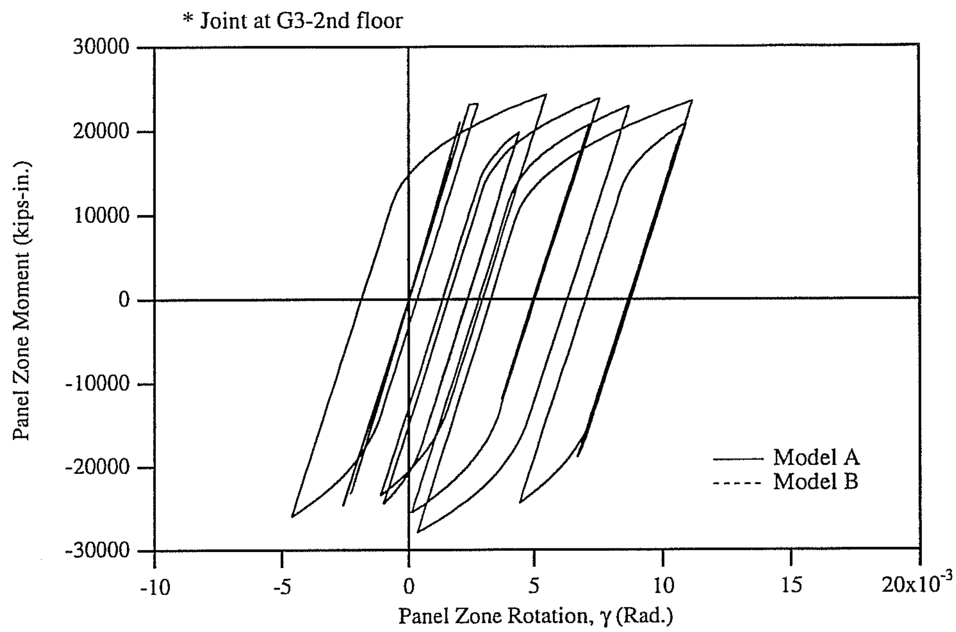
FIGURE 16 - MODEL A VERSUS MODEL B: SCT-1 N90E

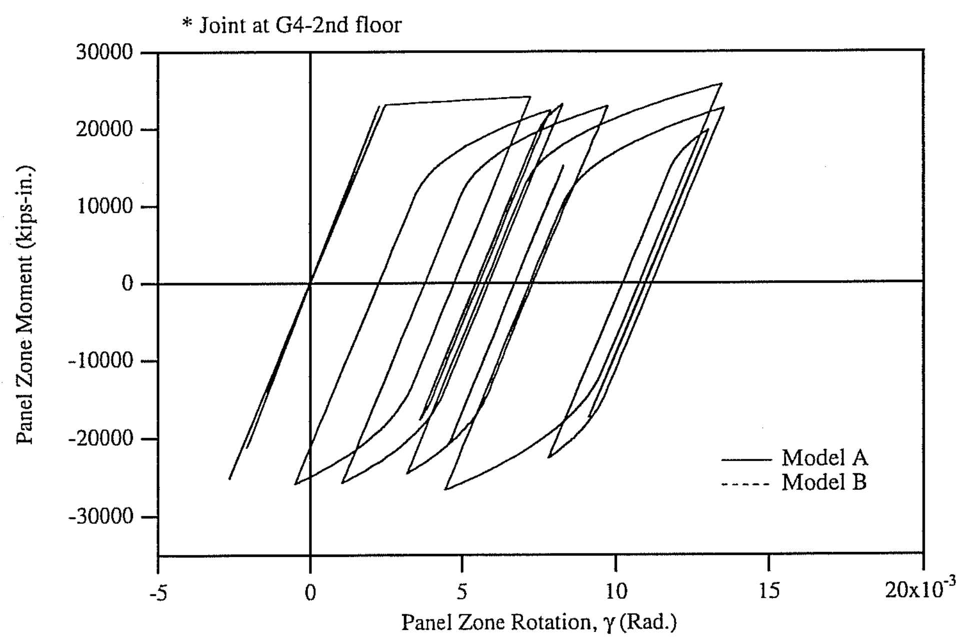




(c) Beam Moment versus Plastic Rotation at Selected Joints

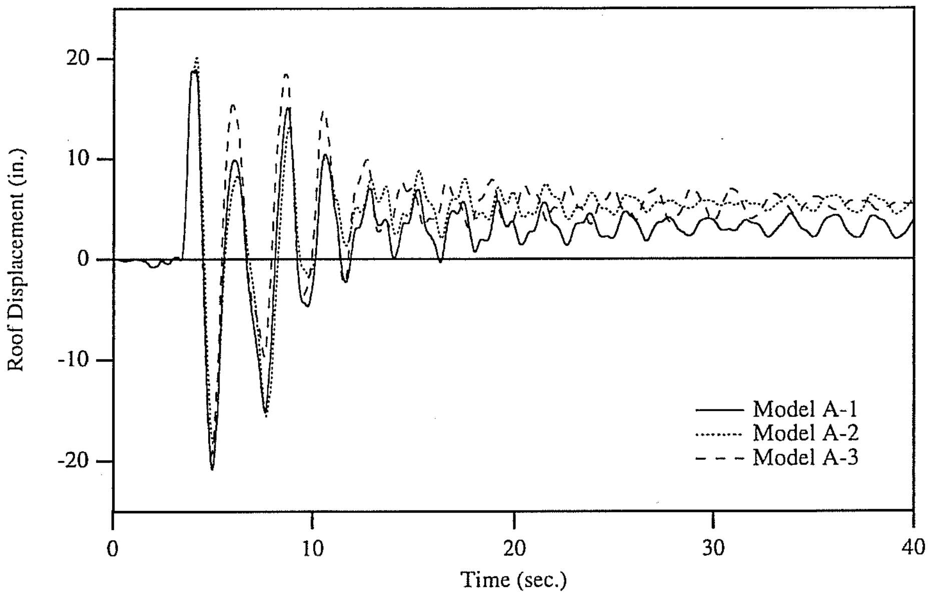
FIGURE 16 - MODEL A VERSUS MODEL B: SCT-1 N90E (CONT.)



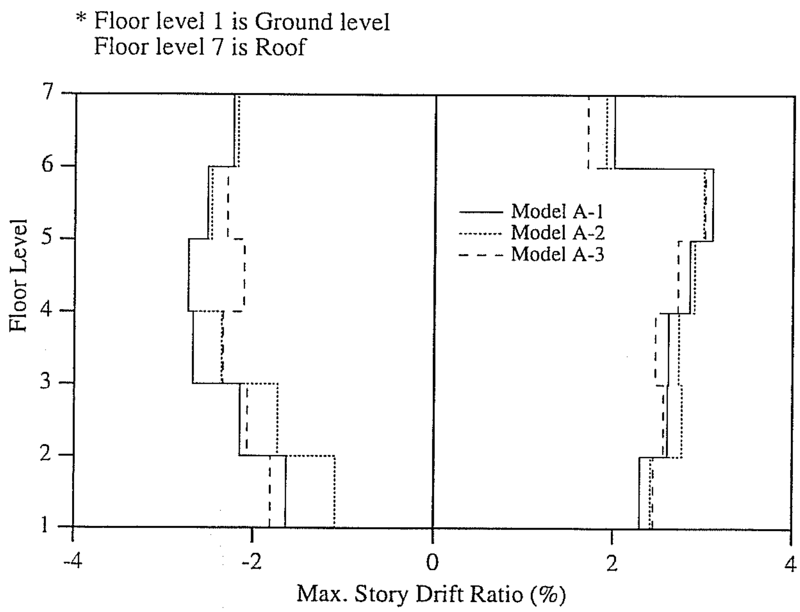


(d) Panel Zone Response at Selected Joints

FIGURE 16 - MODEL A VERSUS MODEL B: SCT-1 N90E (CONT.)

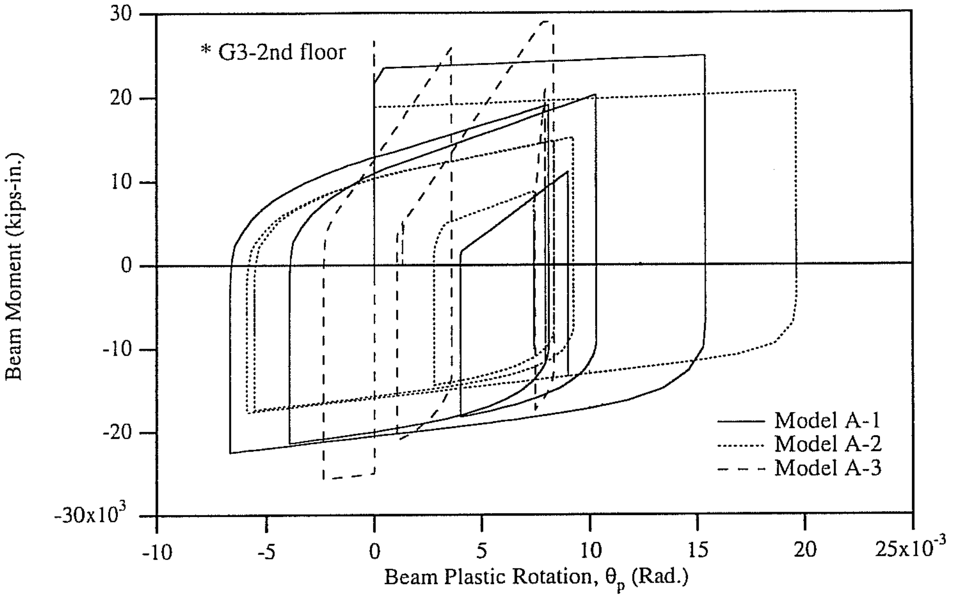


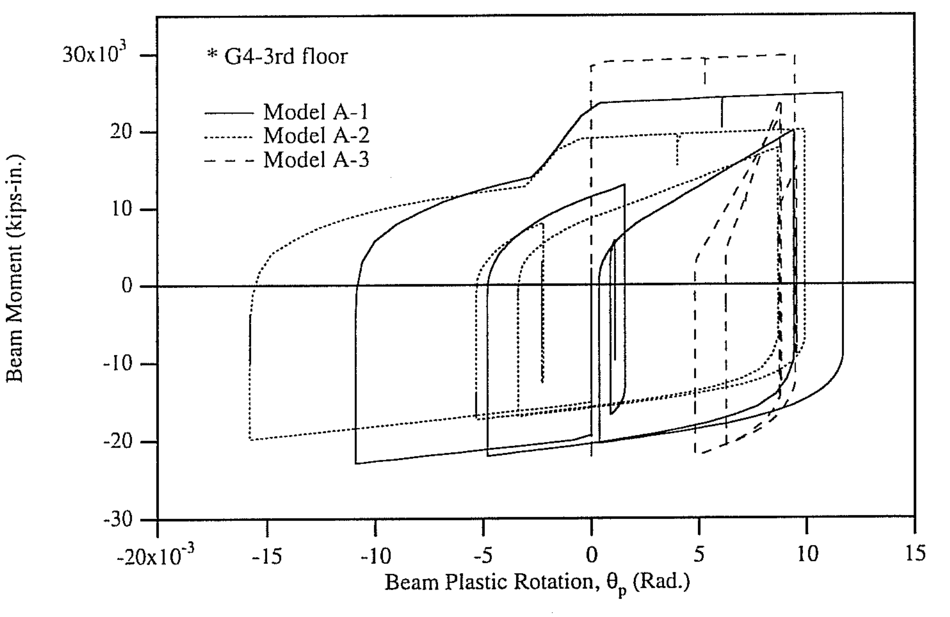
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

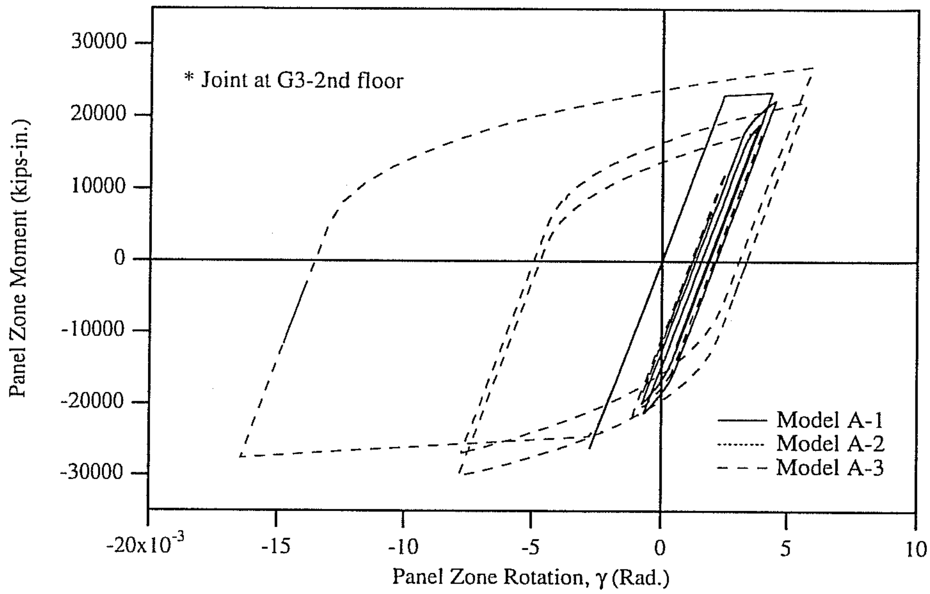
FIGURE 17 - EFFECT OF VARYING YIELD STRESS

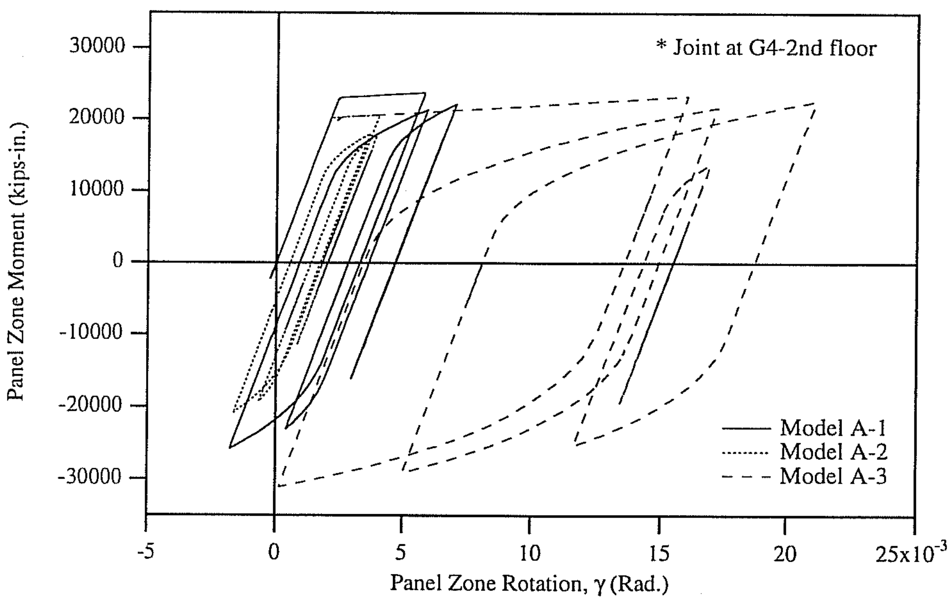




(c) Beam Moment versus Plastic Rotation at Selected Joints

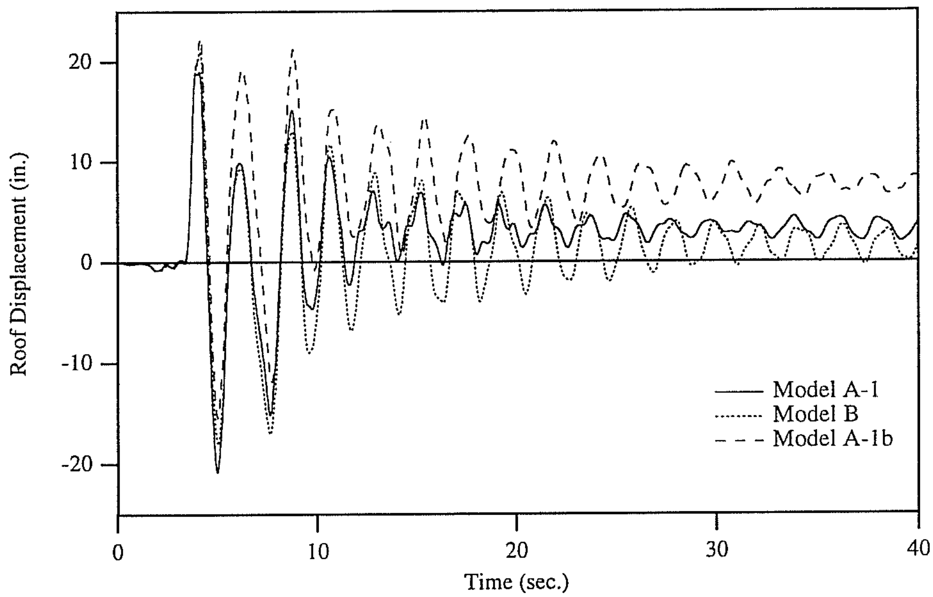
FIGURE 17 - EFFECT OF VARYING YIELD STRESS (CONT.)



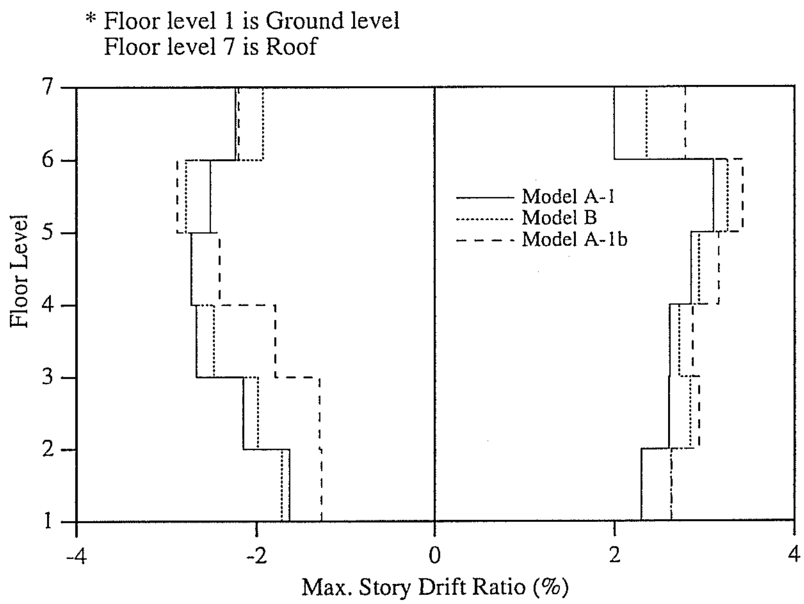


(d) Panel Zone Response at Selected Joints

FIGURE 17 - EFFECT OF VARYING YIELD STRESS (CONT.)

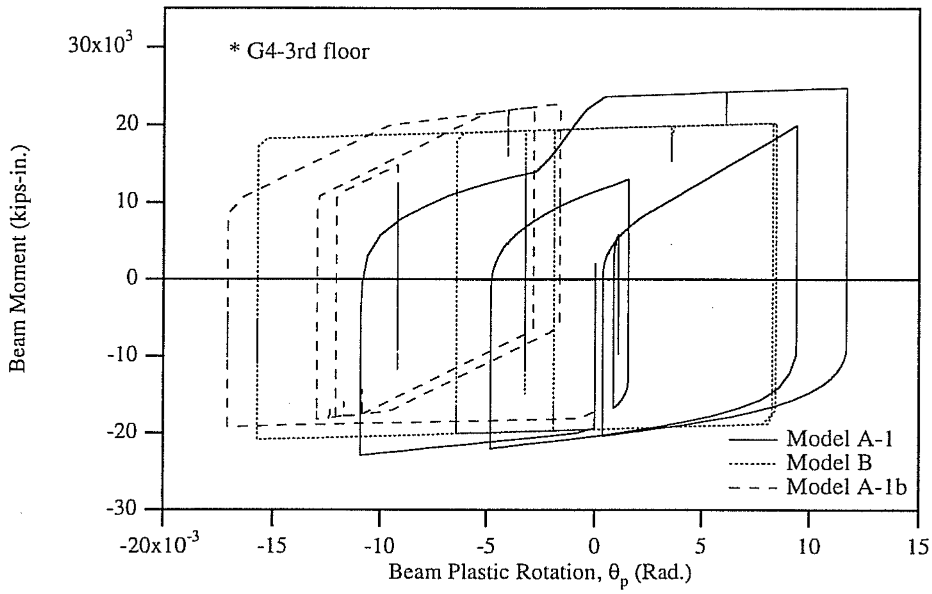


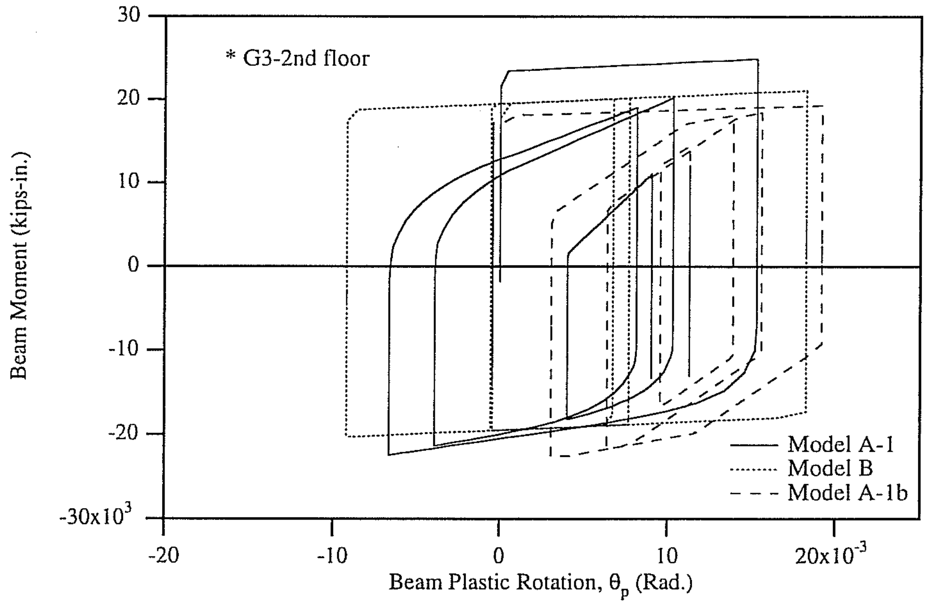
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

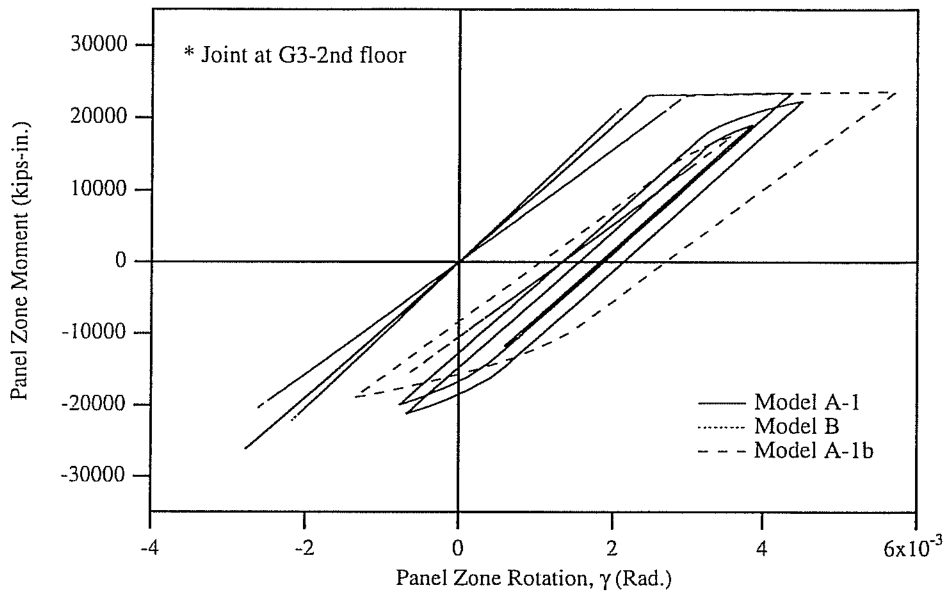
FIGURE 18 - COMPARISON OF BARE STEEL MODELS: REFINED VS. BASELINE

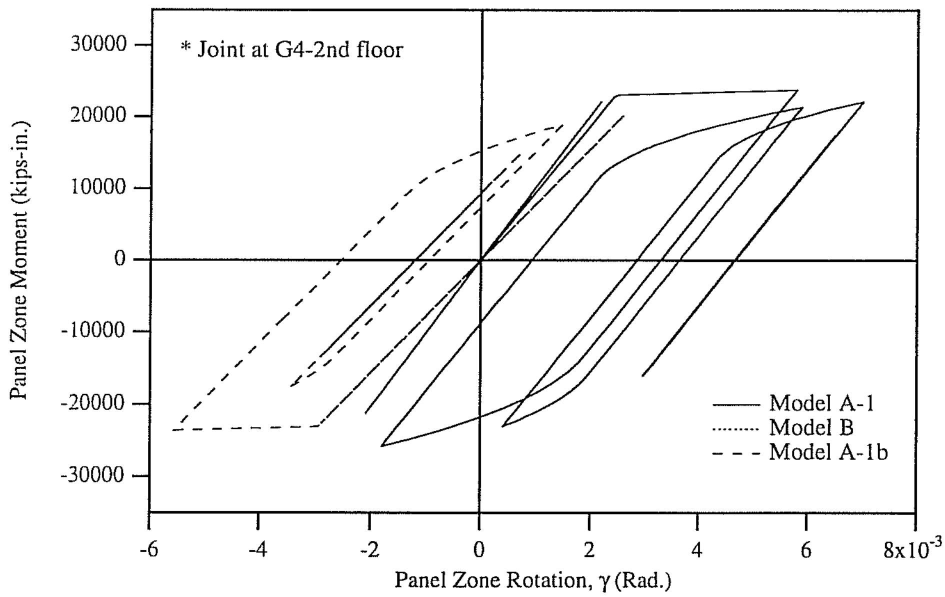




(c) Beam Moment versus Plastic Rotation at Selected Joints

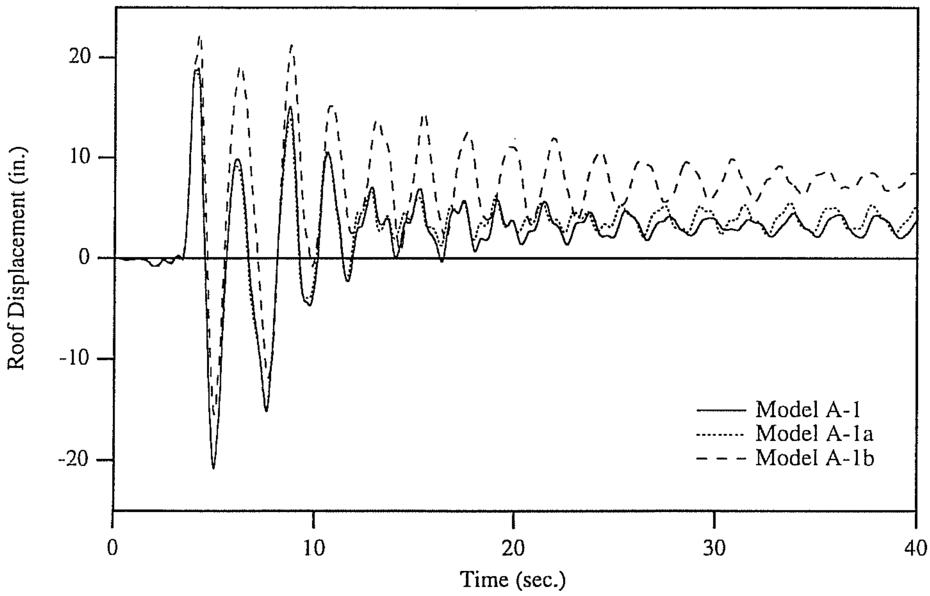
FIGURE 18 - COMPARISON OF BARE STEEL MODELS: REFINED VS. BASELINE (CONT.)



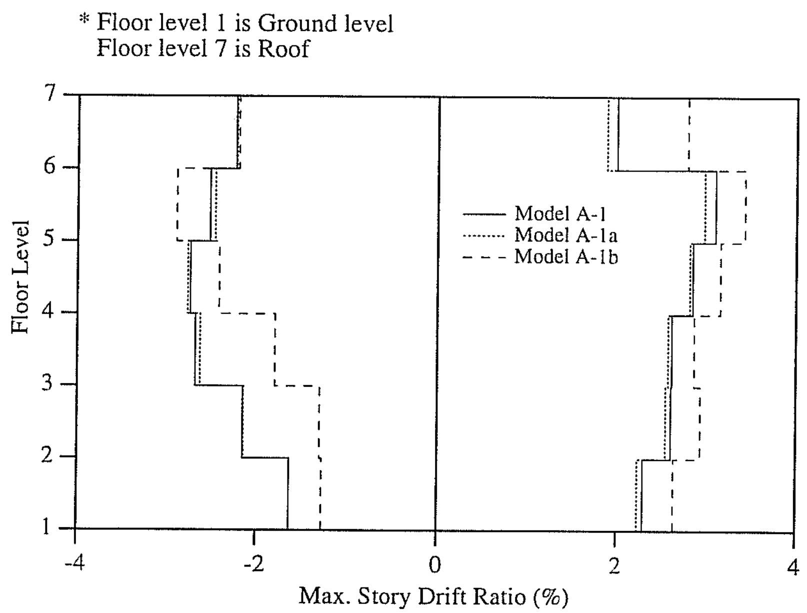


(d) Panel Zone Response at Selected Joints

FIGURE 18 - COMPARISON OF BARE STEEL MODELS: REFINED VS. BASELINE (CONT.)

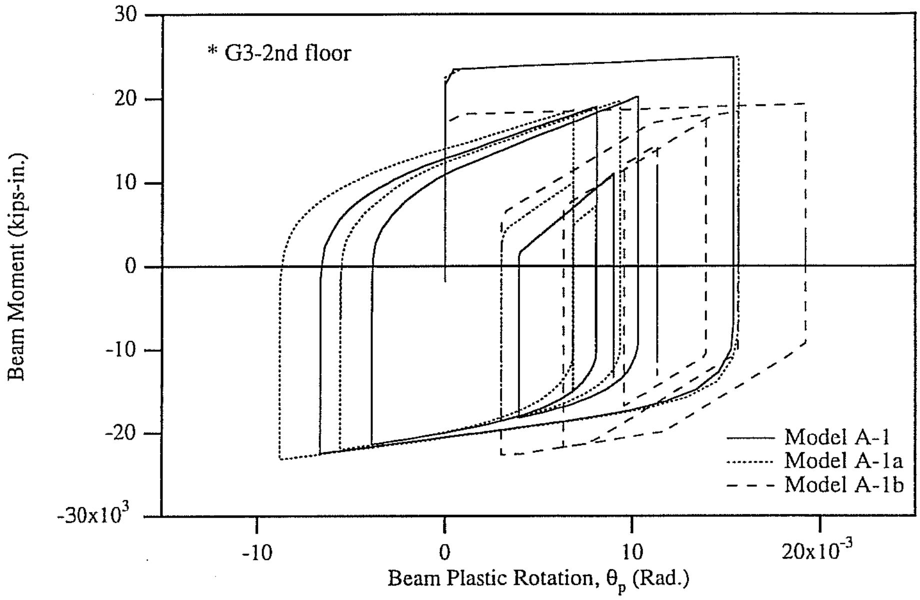


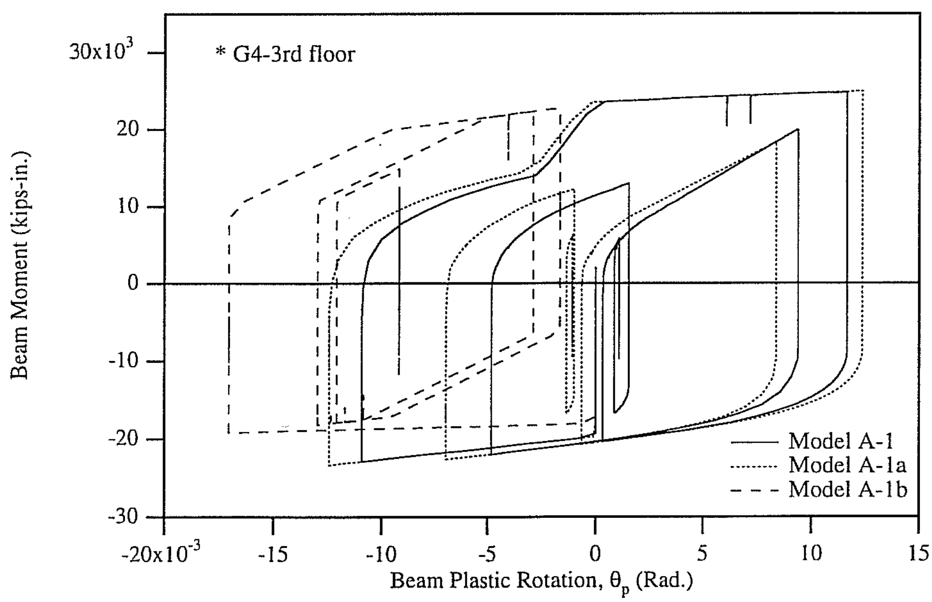
(a) Roof Displacement



(b) Envelope of Maximum Story Drift Ratios

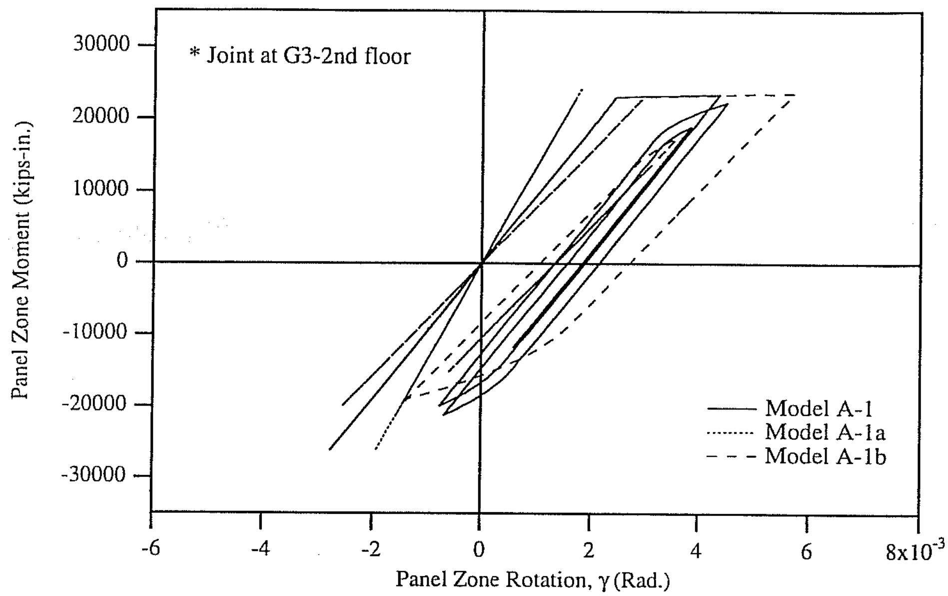
FIGURE 19 - EFFECT OF DOUBLER PLATE PARTICIPATION

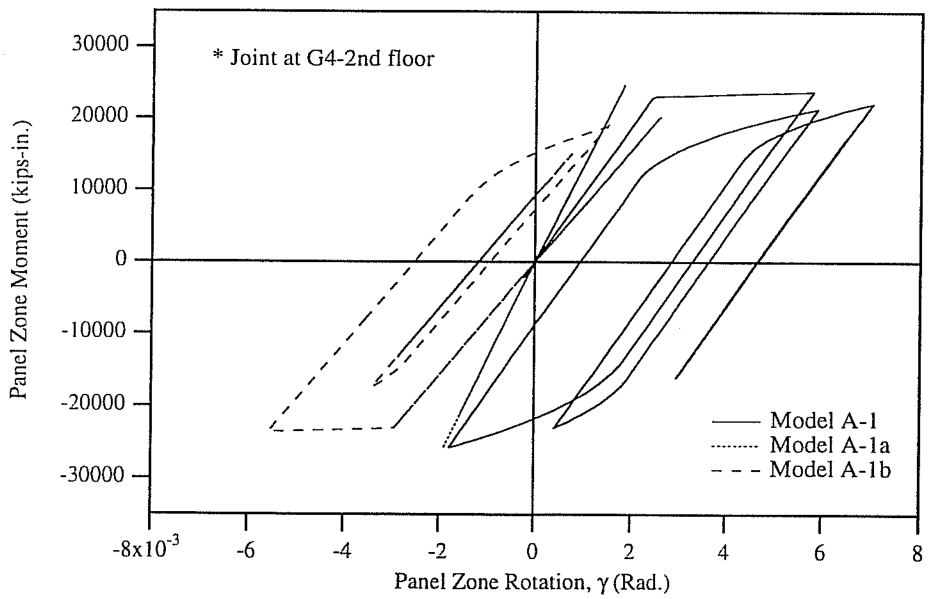




(c) Beam Moment versus Plastic Rotation at Selected Joints

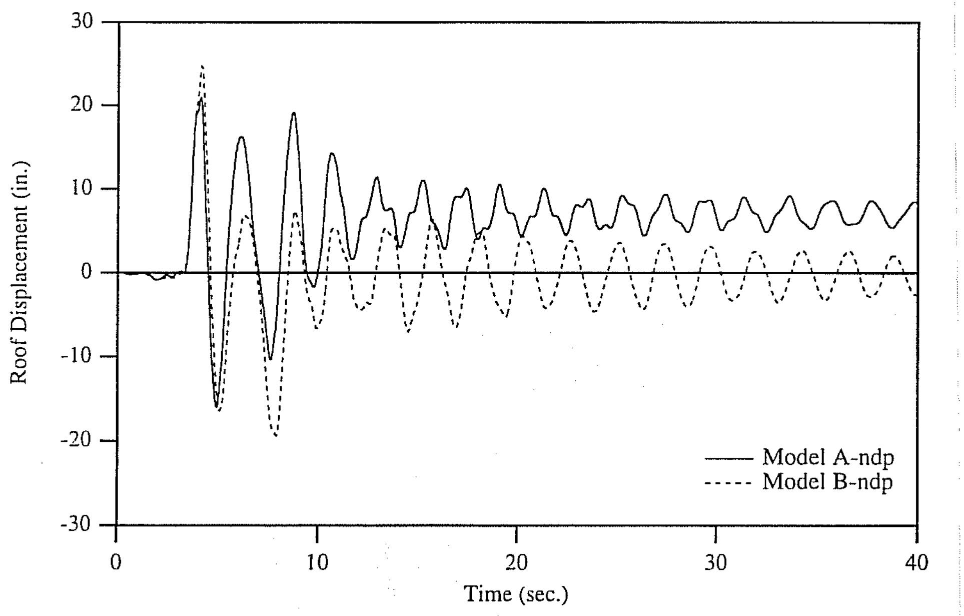
FIGURE 19 - EFFECT OF DOUBLER PLATE PARTICIPATION (CONT.)



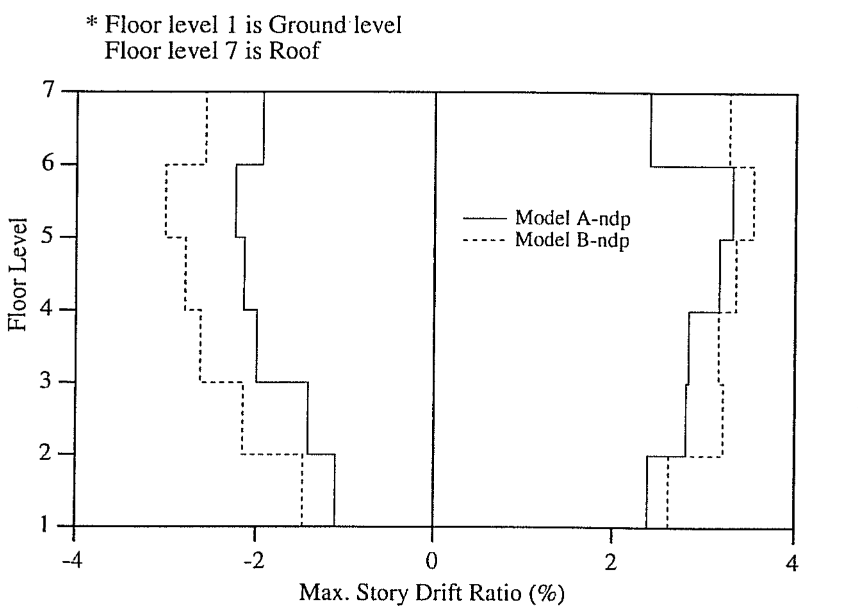


(d) Panel Zone Response at Selected Joints

FIGURE 19 - EFFECT OF DOUBLER PLATE PARTICIPATION (CONT.)

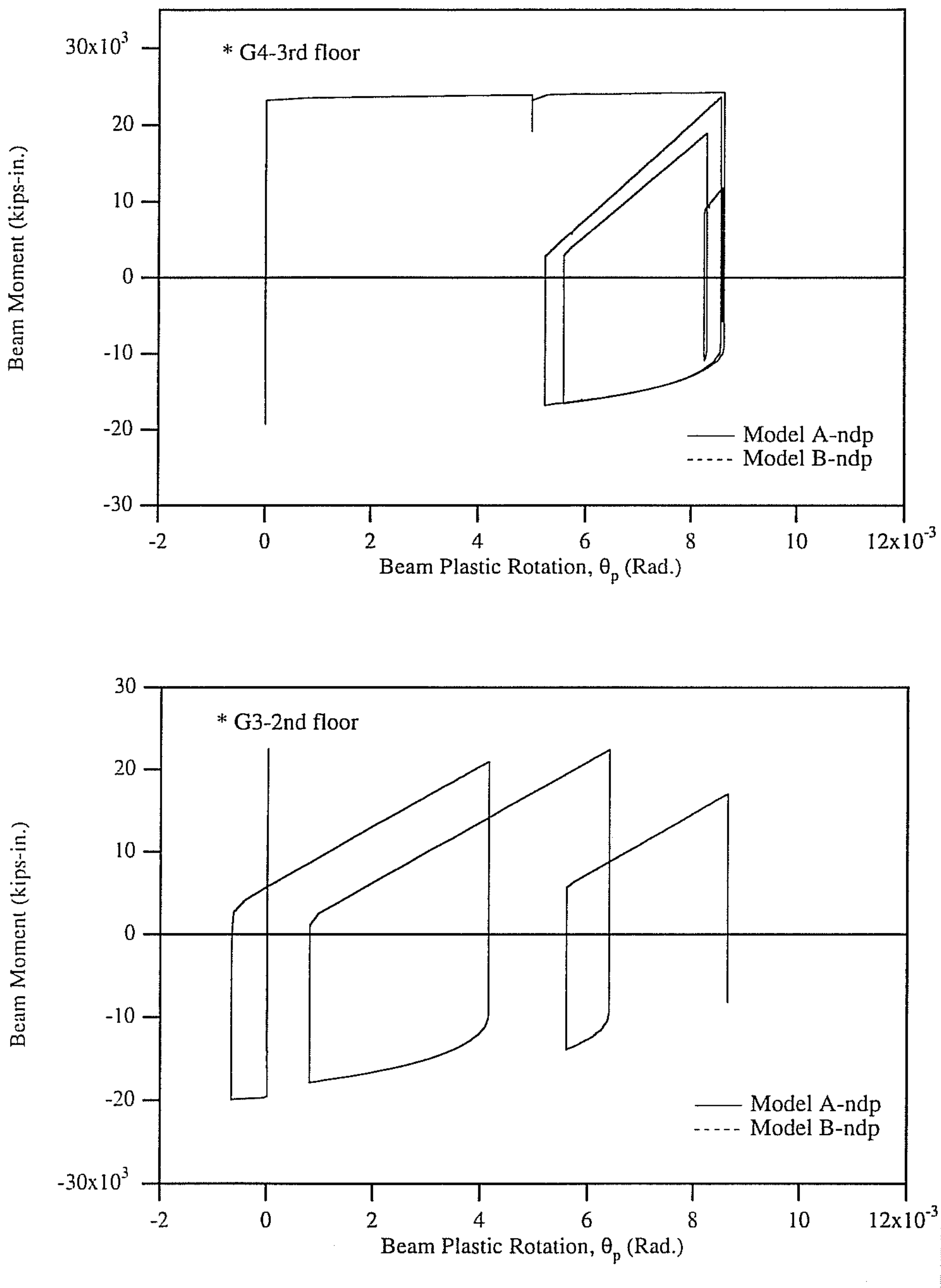


(a) Roof Displacement



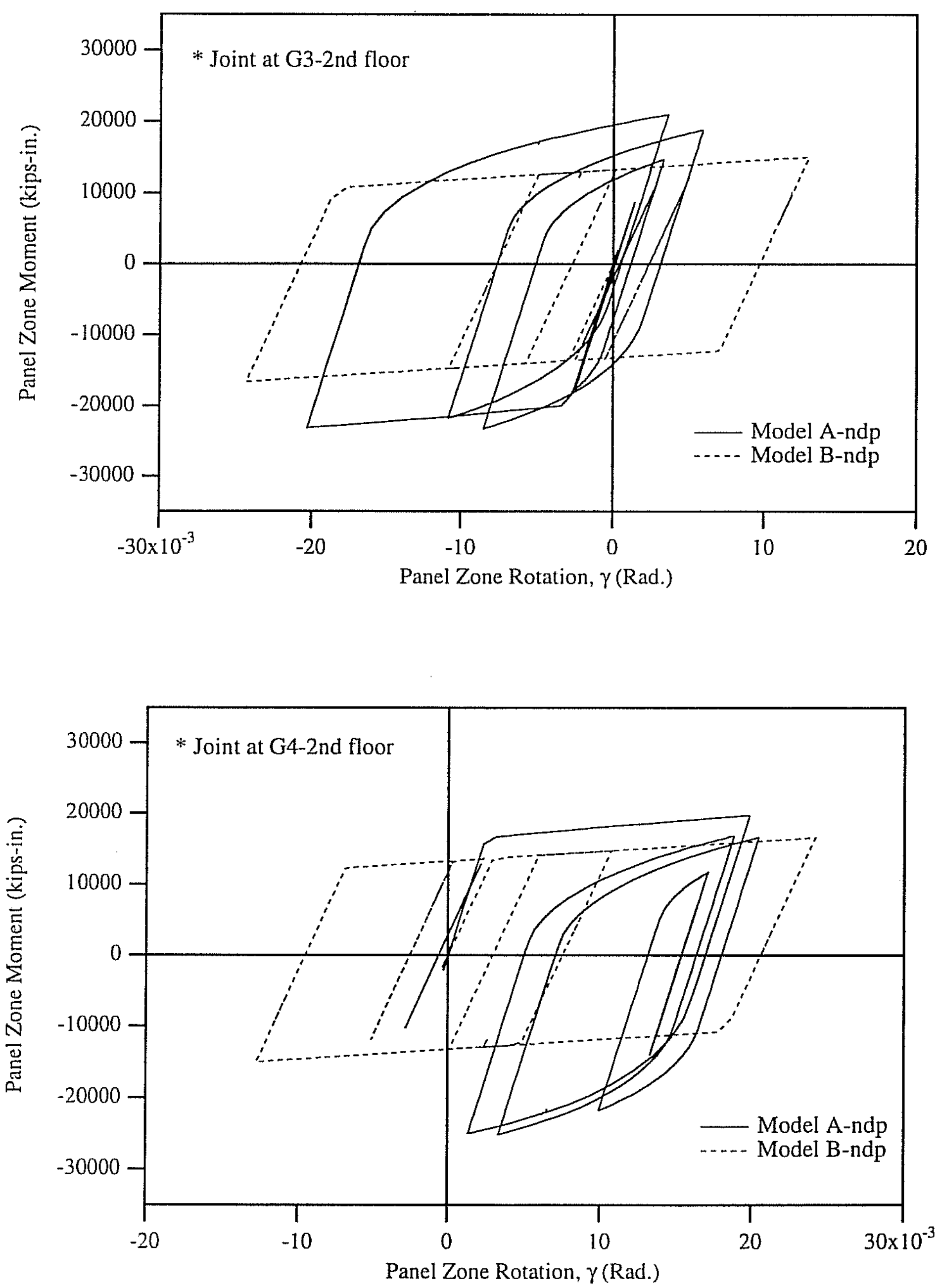
(b) Envelope of Maximum Story Drift Ratios

FIGURE 20 - RESPONSE OF FRAME WITHOUT DOUBLER PLATES



(c) Beam Moment versus Plastic Rotation at Selected Joints

FIGURE 20 - RESPONSE OF FRAME WITHOUT DOUBLER PLATES (CONT.)



(d) Panel Zone Response at Selected Joints

FIGURE 20 - RESPONSE OF FRAME WITHOUT DOUBLER PLATES (CONT.)