PMFSEL REPORT NO. 95-3
July, 1995

ANALYSIS OF A SIX STORY STEEL MOMENT FRAME BUILDING IN SANTA MONICA (SAC BUILDING SITE 7)

Report to the SAC Joint Venture
Task 3.1

FINAL REPORT

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ABSTRACT

This report summarizes the results of an analytical study of a six story steel moment frame building located in Santa Monica, California. This study was conducted as part of the SAC Task 3.1. This particular building is identified under SAC Task 3.1 as Building Site 7.

This building sustained significant damage to its steel moment frame joints in the 1994 Northridge Earthquake. The primary form of damage observed in this building was the fracture of beam flange welds, typically at the beam bottom flange. A large number of the building's 120 moment frame joints experienced some damage.

Analytical studies were conducted to investigate the predicted response of the building in the Northridge Earthquake, and its predicted response under other strong ground motion records. Three types of analyses were conducted in this study: two-dimensional elastic analysis, three-dimensional elastic analysis, and two-dimensional inelastic analysis. A principal objective of the study was to determine if the damage observed in this building was in any way predictable by structural analysis, and if structural analysis could be used as a tool to guide building inspections.

The primary ground motion record used in the analysis of this building was the Santa Monica City Hall record from the 1994 Northridge Earthquake. The record site is very close to the building site, and this record was considered the best estimate available of the actual ground motion experienced by the building during the Northridge Earthquake. In addition to the Santa Monica City Hall record, a variety of other strong motion records were used in the analysis. This included other records from the Northridge Earthquake, as well as records from other strong earthquakes.

The results of both the elastic and inelastic analyses using the Santa Monica City Hall record showed some degree of correlation between the predicted structural response and the damage observed at the joints. There were also numerous exceptions to this trend. Nonetheless, the results of this study suggest that an elastic structural analysis could have been used as a useful guide for inspecting this building. It appears that the chances of locating a damaged joint would be increased by first inspecting joints with the highest predicted beam moment demand capacity ratios (DCRs). For joints with similar levels of beam moment DCR, the chances of locating a damaged joint would be increased by first inspecting the joints for the heaviest beams.

The inelastic analysis indicates that the maximum beam plastic rotations experienced in this building during the Northridge Earthquake were on the order of 0.010 rad. These rather modest inelastic deformation demands suggest that the moment frame joints performed quite poorly.

For the other ground motions considered in this study, the analyses predict significantly greater structural demands on this building (beam moment DCRs, beam plastic rotations, interstory drift ratios, etc.) than were predicted using the Santa Monica City Hall Record. Much higher levels of damage might be expected in this building under these other strong ground motions, than was

experienced in the Northridge Earthquake. The simulated Elysian Park record appeared to be particularly damaging to this building.

Excluding the Elysian Park record, the maximum beam plastic rotations developed in this structure under a variety of very strong ground motions were on the order of 0.02 to 0.03 rad. The Elysian Park record developed a maximum beam plastic rotation of about 0.04 rad.

No attempt was made in this study to model the hysteretic response of a damaged joint. Therefore, no conclusions can be drawn on the consequences of connection damage on the response of this structure in future strong earthquakes.

ACKNOWLEDGMENTS

Funding for this research was provided by the Federal Emergency Management Agency through the SAC Joint Venture. SAC is a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering.

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1.0 INTRODUCTION

This report documents the results of an analytical study of a six story steel moment frame building located in Santa Monica, California. This study was conducted as part of the SAC Task 3.1. This particular building is identified under SAC Task 3.1 as Building Site 7.

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Analytical studies were conducted to investigate the predicted response of the building in the Northridge Earthquake, and its predicted response under other strong ground motion records. Three types of analyses were conducted in this study: two-dimensional elastic analysis, three-dimensional elastic analysis, and two-dimensional inelastic analysis.

The primary objectives of this study are as follows:

- Determine if the damage observed in this building was predictable by structural analysis. That is, can the location of damaged joints be predicted from the results of a structural analysis of the building frames? If so, then it may be possible to use structural analysis as a tool to guide inspections of steel moment frame buildings.

- Investigate the predicted response of the building under other strong ground motion records. Evaluate the expected performance of the building for future earthquakes that may be stronger than Northridge.

- Evaluate the plastic rotation demands at the joints when the frame is subject to strong ground motion records. These predicted demands can be used to guide the design of moment frame joints and can be used to guide experimental programs.

The remainder of this report is organized as follows: Section 2 provides a description of the building; Section 3 describes the damage observed in this building after the 1994 Northridge Earthquake; Section 4 provides an overview of the analyses conducted for the building, and lists the ground motion records used in this study; Section 5 summarizes the results of the elastic analyses; Section 6 summarizes the results of the inelastic analyses. Finally, the conclusions of the study are summarized in Section 7.

2.0 BUILDING DESCRIPTION

The structure under consideration in this study is a six story office building located in Santa Monica, California. The building was constructed in 1988, and was designed to the 1985 Uniform Building Code. The building consists of six stories of steel framing above grade with five subterranean reinforced concrete parking levels. The total gross building area is approximately 91,000 square feet for the above grade levels, and 93,000 square feet for the below grade levels. The approximate plan dimensions for the below grade levels is 130' by 190'. The plan dimensions for the above grade levels vary due to setbacks. The building plan from the second floor through the fifth floor is relatively constant, having an approximate area of 17,000 square feet per floor. The major building setbacks start at the sixth floor and continue onto the roof. The approximate floor area for each of the upper two levels is 9,800 square feet. A mechanical penthouse is located on the roof having an approximate area of 3,600 square feet. The building height measured from the ground floor to the roof is about 81 feet having a typical floor to floor height of 13'-0" for all levels except for that between the ground floor and the second floor. The floor to floor height from the ground floor to the second floor is 15'-6". The floor to floor height for the below grade levels is typically at 9'-0".

Figures 1 and 2 show approximate plan views of the building. As shown in these figures, the building is oriented at approximately 45 degrees from the compass north-south direction. For convenience, a "Reference North" direction is defined to coincide with one of the principal directions of the building.

2.1 BUILDING FOUNDATION

The foundation system is of cast-in-place concrete spread footings with concrete slabs on grade. The basement wall construction is of shotcrete with continuous wall footings. The design soil bearing pressure for the footings is 8,000 pounds per square foot, with a one-third increase permitted for wind and earthquake loading.

2.2 SUBTERRANEAN LEVEL CONSTRUCTION

The below grade structure is of cast-in-place reinforced concrete construction. The floor is constructed using a two way flat plate system with drop panels at columns. The spacings of the concrete columns are from 40'-0" on center to 20'-0" on center. The cast-in-place concrete construction is terminated at the first level below the ground floor, at which point the steel structure begins.

2.3 ABOVE GRADE CONSTRUCTION

The above grade structure including the ground floor is of structural steel construction, with floor construction consisting of concrete on metal deck. The floor deck at the ground floor is of 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 of 3 1/4" light weight concrete on 3" deep metal deck (6 1/4" total thickness). The roof, with the exception of the mechanical penthouse, is of insulating concrete (vermiculite) on 1/2" metal deck. The mechanical penthouse has the same floor deck system as the second floor.

Typically, the floor beams are spaced at 10'-0" on center and are supported by girders. The steel beams and girders consist of welded shear connectors at the top flange and are connected to the concrete deck forming a composite floor system.

The lateral force resisting system of the structure consists of steel moment resisting frames in both the longitudinal (ref. north-south) and transverse (ref. east-west) direction. The longitudinal direction consists of two two-bay exterior frames from the ground to fifth floor. Because of the building setbacks, each two-bay frame transitions into two single bay frames from the fifth floor up. This creates an in-plane offset of the lateral force resisting system in the north-south direction. The column to column spacings are typically 20'-0" on center in this direction. The transverse direction consists of four single bay steel moment resisting frames that run continuously from the ground floor to the roof. All four frames are located in the core of the structure. The column to column spacing is typically 28'-0" on center.

The beams for all moment frames were specified as A36 steel, while all columns (and column doubler plates) are A572 Grade 50 steel. Elevations of the moment frames are shown in Figs. 3 and 4. The elevations show the steel columns pinned at their base, which is one level below the ground level. Thus, 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.

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 many of the joints (see Figs. 3 and 4 for doubler plate thicknesses). Most joints were not provided with stiffeners (continuity plates).

2.4 EXTERIOR WALL CONSTRUCTION

The exterior walls of the building were constructed with 1 1/4" stone panels supported by light gage metal stud framing. Windows occur on all four sides of the building throughout the levels. The inside face of the exterior metal stud framing is finished with dry wall.

3.0 BUILDING DAMAGE DESCRIPTION

Shortly after the January 17 Northridge Earthquake, the building was visually inspected for damage during a brief visit. No significant damage to the non-structural elements of the building was observed. Only cracks at the dry wall taped joints were observed. The building elevator system was in operation which suggested that the building was not significantly out of plumb.

Later based on the increasing evidence that many steel frame buildings had sustained damage at the beam-to-column welded moment connections, an inspection program for this building was conducted. Both visual and ultrasonic examinations were performed at all of the beam-to-column moment frame connections after fireproofing was removed. The findings and observations of the inspection program are summarized below. Damage designations such as "Type W1" , "Type W2", etc. correspond to the damage designations established by Nabih Youssef and Associates for a NIST (National Institute of Standards and Technology) sponsored damage survey and for SAC Task 2.

1. 92 of a total 120 beam-to-column moment connections showed evidence of damage or of welding defects. Of these 92 connections, the damage was as follows:

    a. 30 of the 92 damaged connections had visible fractures passing through the full thickness of the beam flange weld. These fractures occurred within the weld metal itself, or near the weld-column or weld-girder interface. (Damage Type W3 or W4). No significant fractures within the column flange were observed in this building. All of the fractures that extended the full thickness of the beam flange weld occurred at bottom flange welds.

    b. 29 of the 92 damaged connections had weld fractures with depths in excess of 1/3 the beam flange thickness but less than the full flange thickness. (Damage Type W2).

    c. 33 of the 92 damaged connections had weld fractures with a depth less than 1/3 of the beam flange thickness. (Damage Type W1).

2. The majority of fractures were found at the beam bottom flange weld. However, some fractures were also found at the beam top flange weld. Whenever a fracture was found at a top beam flange weld, the bottom flange weld at the same connection was always found to have sustained more severe damage.

3. No bolt or shear tab damage was discovered.

4. The moment frames in the transverse (ref. east-west) direction appear to have sustained more severe damage.

5. The majority of the severely damaged connections were found between the second and the fourth floors in both directions.

Figure 5 provides a graphical summary of the connection damage for this building.

For purposes of later discussion, the types of joint damage that occurred in this building are divided into three broad categories:

1. Severe Damage.
   Joints with fractures that extend the full thickness of the beam flange weld are classified as having sustained severe damage. These joints are shown in Fig. 5 with a solid black circle.

2. Moderate Damage.
   Joints with fracture depths at least one-third the beam flange weld thickness, but less than full thickness, are classified as having sustained moderate damage. These joints are shown in Fig. 5 with a circle that is half black.

3. Minor or No Damage.
   Joints with fracture depths less than one-third the beam flange weld thickness, or joints with no fractures, are classified as having sustained minor or no damage. These joints are shown in Fig. 5 with open circles (minor damage) or with no circle (no damage).

Of the 120 moment frame joints in this building, 30 sustained "severe damage", 30 sustained "moderate damage", and 60 sustained "minor or no damage", according to the above categories.

4.0 OVERVIEW OF BUILDING ANALYSES

This chapter provides an overview of the types of analyses conducted as part of this study, the types of analytical models developed for the buildings, and the ground motion records used in this study. Details of the analytical models and the results of the analyses are covered in the following chapters.

4.1 TYPES OF ANALYSES AND MODELS

Three different types of analyses were conducted for this building:

- Two-dimensional elastic analysis.
  The moment frames on Lines F and G of the building were analyzed using 2-D elastic analysis. The two frames were coupled at the floor levels to enforce common horizontal displacements at the floor levels. The analyses were conducted using the SAP90 computer program.

- Three-dimensional elastic analysis.
  A 3-D elastic model was constructed using the SAP90 computer program. This model included all moment frames in both directions of the building, and assumed rigid floor diaphragms.

- Two-dimensional inelastic analysis.
  The four moment frames in the building's transverse direction were analyzed using 2-D inelastic analysis. The four frames (along Lines D, E, F, and G) were coupled at the floor levels to enforce common horizontal displacements at the floor levels. The analyses were conducted using the ANSR-1 computer program (Mondkar and Powell 1975) employing a variety of element subroutines for modeling inelastic response of frame members.

For each type of analysis listed above, several analytical models of the building frames were developed. These generally included a "benchmark" model, as well as more refined models. The benchmark modeling assumptions were specified by SAC, and generally represented a fairly simple structural model. Various refinements were then made to the benchmark models in an attempt to develop more realistic models. The models are described in greater detail in the following chapters.

4.2 GROUND MOTION RECORDS

For both the elastic and inelastic analyses, a variety of ground motion records were used in this study. Table 1 provides a listing of the records. These include records from the 1994 Northridge Earthquake, as well as a variety of other strong motion records. With the exception of the 1985 Mexico City record, all other records listed in Table 1 were supplied through SAC Task 4. Note that only selected components of each record were used in the analyses of the Site 7 building, as indicated in Table 1.

The Santa Monica City Hall record was used in this study as the best estimate of the ground motion experienced by the Site 7 building in the 1994 Northridge Earthquake. The draft report for SAC Task 4 states: "Building site #7 is very close to the CDMG Santa Monica record. ...we consider that the CDMG Santa Monica record is a reasonable representation of the motion at building site #7, and is the only appropriate record for that site." (Somerville 1995). Since the Santa Monica City Hall (smch) record provided the best estimate of the actual ground motion at Site 7, none of the simulated Northridge Earthquake records for Site 7 were used in this study. For the 3-D elastic analysis, both the north-south (smch.000) and east-west (smch.090) components of this record were used. The 2-D elastic and inelastic analyses were conducted for the moment frames in the transverse direction (in the reference east-west direction) of the building. This direction is oriented at 45 degrees from compass north. Thus, for the 2-D analyses, a component of the Santa Monica City Hall record was developed for motion in this direction (smch.045).

Ground motion time histories, response spectra, and other data for the records listed in Table 1 (except for the 1985 Mexico City record) are provided by Somerville (1995). The 1985 Mexico City SCT-1 record (N90E component) was also used in this study for inelastic analysis of the Site 7 building frames. This record was chosen for its long duration, and for its long period content. It should be noted that the SCT-1 record was taken at a site with very deep soft soils, and therefore is not characteristic of the actual soil conditions at the Site 7 building.

In addition to conducting time history analysis using the ground motion records listed in Table 1, a response spectrum analysis was conducted on the 2-D elastic model as one of the benchmark cases. For this analysis, the probabilistic spectrum provided by SAC Task 4 was used. This spectrum is identified as an equal hazard spectrum representing a 10% exceedance probability in 50 years for each period on the spectrum. The spectrum was developed for a site in the northern San Fernando Valley. In this report, this spectrum is referred to as the "Equal Hazard" spectrum. Details of this spectrum are described in the Task 4 report (Somerville 1995).

5.0 ELASTIC ANALYSES

5.1 OVERVIEW

This chapter summarizes the results of the 2-D and 3-D elastic analyses conducted for the Site 7 building. For both the 2-D and 3-D analyses, the results of the benchmark analyses are presented first, followed by the results based on various refinements to the models.

For the large number of benchmark analysis cases, a simple two dimensional elastic model was constructed for the building's transverse (ref. east-west) direction. The 2-D model was used for the various benchmark ground motions, as well as for the Santa Monica City Hall Record. The 2-D model does not represent the most accurate elastic model for this building, but provided a reasonable basis for comparative assessments of frame response under various ground motion records.

The 3-D model was constructed to provide a more accurate estimate of the building's elastic response. This model was only used with the Santa Monica City Hall (smch) record, in order to evaluate possible correlations of predicted elastic response with actual observed damage.

For all analyses conducted on this building, the seismic mass was based on the dead load of the structure, including the facade, and a 10 psf allowance for partitions. The total weight of the building was computed as 7727 kips. The distribution of weight between the floors is listed in Table 2.

The frame response parameters reported from the elastic analyses include frame displacements, story drift ratios, and story shears. The primary response parameter considered in the analyses, however, is the elastic demand to capacity ratio (DCR) for beam end moments. The DCR is computed at the maximum moment generated at the end of the beam, divided by the plastic moment (M_p) of the beam. For this purpose, M_p of the beam was computed using a steel yield stress based on the mean values reported in the recent AISI statistical study. These values vary by ASTM shape group. The values of F_y used to compute M_p of the beams for the beam DCRs are summarized in Table 3.

All beams in the moment frames of this building were of A36 steel. Most of the beam sizes fall within ASTM Shape Group 2, with some of the lightest sections falling in Group 1. The columns of the moment frames were A572 Gr. 50 steel, and fell within either Shape Group 2 or 3. Note that DCRs were not computed for the columns in the elastic analyses. The information on A572 Gr. 50 steel is included in Table 3, as this will be used later for the inelastic analysis.

In the remainder of this chapter, the results of the 2-D analyses are presented first, followed by the results of the 3-D analyses. Discussion of the results is provided at the end of the chapter. This chapter only provides a summary of the analyses. More details are provided in Appendix A for the 2-D analyses, and in Appendix B for the 3-D analyses.

5.2 TWO DIMENSIONAL ELASTIC ANALYSES

Two dimensional elastic analyses were conducted for moment frames in the building's transverse direction. Two moment frames were selected for these analyses: the frames along grid lines F and G (see Fig. 1). Only two of the four frames were selected because all of the frames had similar lateral stiffness, and in order to keep the model as simple as possible. The primary purpose of the 2-D model was to provide comparisons of response for a number of ground motion inputs.

The 2-D model was constructed using the SAP90 computer program. The two frames were connected together at each floor with rigid links. The ground floor was considered the seismic base of the model, and was provided with a lateral restraint. The columns were extended one level below the ground floor and then pinned, in order to simulate the as-built condition of the structure. The boundary conditions at the base of the frame are as indicated in Fig. 3. Since the 2-D model included only two of the four transverse frames, the seismic mass was based on one-half of the weights listed in Table 2.

The benchmark analyses were based on the following assumptions:

- Bare steel frame without slab participation is modeled.
- Centerline dimensions between members are used (no panel zones or rigid offsets).
- Gravity loads are based on (1.0 X dead load) + (0.5 X live load). The design live load is taken as 50 psf. Thus, 25 psf live load is included as gravity load on the model. The gravity dead load included 10 psf for partitions.
- No patterned live loads is considered.
- No accidental torsion is considered.
- 5% damping is used.
- P-delta effects are included.

In addition to analysis based on the above benchmark assumptions, further analyses were conducted by varying some of the modeling assumptions listed above. A total of five computer models were constructed. The key parameters of the models are listed in Table 4.

The following ground motions and response spectrum were used for the 2-D elastic analyses:

- 10/50 Equal Hazard Spectrum (EH Spectrum)

- 1994 Northridge - Canoga Park Record (cnpk.106)
- 1994 Northridge - Sylmar Olive View Hospital Record (sylm.090)
- 1940 Imperial Valley - El Centro Record (ivir.270)
- 1978 Iran - Tabas Record (taba.344)
- 1994 Northridge - Santa Monica City Hall Record (smch.045)

5.2.1 RESULTS OF 2-D ELASTIC ANALYSES - BENCHMARK CASES

Key results for the benchmark analyses are summarized in Table 5, and in Figs. 6 and 7. These figures show maximum frame displacements, maximum story drift ratios, and maximum story shears. Table 5 lists DCRs for the beam end moments. The beam locations are indicated in the first column of this table. For example, "F.5.3" denotes the frame along line F, 5th floor beam, beam end at grid line 3. Table 5 also includes DCRs for the case of gravity load only. For most beams, the gravity load moments appear to be insignificant, with typical DCRs on the order of .01 to .03.

5.2.2 RESULTS OF 2-D ELASTIC ANALYSES - ADDITIONAL CASES

Using the Santa Monica City Hall record (smch.045), 2-D elastic analyses were conducted using the five different computer models listed in Table 4. The key results for these analyses are summarized in Table 6, and in Figs. 8 and 9. The same structural response parameters are provided as for the benchmark analyses. The results of the analyses are discussed in Section 5.4.

5.3 THREE DIMENSIONAL ELASTIC ANALYSES

A three dimensional elastic model was constructed of the building using the SAP90 computer program. Several different models were constructed using a variety of modeling assumptions. All of the 3-D models were run only with the Santa Monica City Hall (smch) record. The purpose of this phase of the analysis was to develop a more accurate elastic model than possible with 2-D analysis, and to evaluate the model with the best estimate of the actual ground motion experienced by this building during the 1994 Northridge Earthquake.

For the 3-D analysis, only the moment frames in the building were modeled. Rigid diaphragms were assumed at each story level. The boundary conditions at the column bases were the same as those assumed for the 2-D analysis (see Figs. 3 and 4).

Using the smch record, five different trial analyses were conducted. The basic model started with the benchmark modeling assumptions listed in Section 5.2 above, and then introduced various model changes for the different trial analyses, as explained below and as summarized in Table 7. The primary structural response parameter examined for the trial analyses is the demand to capacity ratio (DCR) for beam end moments. For most analyses, DCRs are reported only for the frames on lines F and G. However, for one of the analyses, DCRs were computed for all moment frame beams in the building.

- Trial 1: The first trial used approximate mass moments of inertia based on a rectangular plan for each story. The gravity loads used were the expected loads, taken as 1.0D + 0.08L. Gravity loads for beams were applied only to moment frames on lines F and G. Damping was taken as 5% and the building was modeled using centerline dimensions without rigid end offsets. Vertical accelerations were not included.

- Trial 2: More precisely calculated mass moments of inertia were used for each story. The gravity loads were modified to the benchmark combination of 1.0D + 0.5L. Vertical accelerations were included.

- Trial 3: This trial is the same as Trial 2, except that vertical accelerations were not included.

- Trial 4: In this trial, an attempt was made to use more realistic modeling assumptions than the benchmark model. Rigid offsets were used at beam-column joints with 50% rigidity of the joints. The damping was taken as 2%. Gravity loads remained at the benchmark value of 1.0D + 0.5L, as the effect of gravity load on beam DCRs was found to be negligible for most beams.

- Trial 4a: This was the same as Trial 4, except gravity loads were not included. DCRs were computed for all moment frame beams in the building for this trial.

- Trial 5: This was the same as Trial 4, except that torsion was locked in the analysis. This was done to provide a comparison with the 2-D analysis.

5.3.1 RESULTS OF 3-D ELASTIC ANALYSIS

Results of the various trial analyses are documented in Appendix B of this report. In this section, only a few key results are summarized. DCRs for beam end moments for the Trial 4 analysis are summarized in Table 8, and are graphically displayed in Figs. 10 and 11. The Trial 4 model, in the opinion of the writers, provided the most realistic elastic model of the five different trial analyses considered. In Table 8, in addition to reporting the beam DCR, the type of damage observed at each joint is also listed. The damage types in Table 8 correspond to those listed in Section 3 of this report. Further results of the 3-D elastic analysis will be considered in the following section.

5.4 DISCUSSION OF ELASTIC ANALYSES

In this section, the results of both the 2-D and 3-D elastic analyses of the Site 7 building are discussed. The discussion is organized to consider various issues of interest.

2-D versus 3-D Elastic Analysis.
Comparison of the 2-D and 3-D model results, for similar modeling parameters (same damping, same joint modeling, etc.), show some significant differences, particularly for beam moment DCRs. For example, Case 4 for the 2-D analysis and Trial 4 for the 3-D analysis used essentially identical modeling assumptions. Comparing the DCRs for these two cases (Tables 6 and 8) indicates that the DCRs from the 3-D analysis are typically 20 to 40 percent higher than from the 2-D analysis. This suggests that three dimensional effects, particularly torsion, may be important for this structure. In order to examine the importance of torsional response, the 3-D model was run with torsion prevented in the analysis (Trial 5). The DCRs for the 3-D model with torsion locked agreed closely with the DCRs from the 2-D analysis. This comparison tends to confirm the importance of torsion in the response of this building. Thus, the results of the 3-D analysis are considered the most accurate representation of the building's elastic response. The results of the 2-D analysis are useful for examining comparative response of the moment frames for varying modeling assumptions or for varying ground motion inputs.

Influence of Modeling Assumptions.
The effect of varying the modeling assumptions was examined for both the 2-D and 3-D analyses. A number of modeling assumptions were considered, including the benchmark model assumptions as well as several variants of the benchmark model. In general, these comparisons showed that the predicted building response can be quite sensitive to the model parameters. For example, Figs. 8 and 9 show that for the same ground motion record, the floor displacements, story drift ratios, and story shears can vary by up to a factor of two by varying the model parameters. A similar observation can be made for beam moment DCRs by examining Table 6. The comparisons suggest that the predicted response is particularly sensitive to the assumed damping ratio. The elastic response increased significantly in going from 5% damping down to 2% damping.

Influence of Gravity Loads.
Both the 2-D and the 3-D analyses suggest that gravity loads have a largely insignificant effect on beam moments. This is illustrated by Tables 5 and 6. For most beams, gravity loads generated moments less than about 5% of M_p. Gravity load moments were somewhat higher for roof beams.

Correlation of Beam DCRs with Observed Damage.
One of the objectives of this study was to determine if the damage observed in the building correlates in some way with the results of a structural analysis. That is, can the location of damaged joints be predicted from a structural analysis of the building frames? If this is possible, then a structural analysis could be used to guide inspection of a steel moment frame building after an earthquake.

The primary structural response parameter that will be compared with the actual observed damage is the beam moment demand-capacity ratio (DCR). Using beam moment DCR as an indicator of damage is based on the rationale that the beam flange weld fractures observed in this building are related to the flexural demands at the beam end. A DCR value greater than 1.0 suggests the beam moments reached M_p, and flexural yielding occurred at the beam end. This observation may not be valid, however, if the column or panel zone yields prior to the beam at a joint. For joints with weak panel zones (permitted since the 1988 UBC), panel zone yielding may occur at beam moments substantially less than M_p. For such cases, the beam end may never achieve M_p, even though the beam moment DCR exceeds 1.0 in an elastic analysis. For such cases, the beam moment DCR may be quite misleading. The Site 7 building was designed prior to the 1988 UBC, and thus is provided with relatively strong panel zones. Thus, the use of beam moment DCRs as a possible indicator of damage may be reasonable for this building.

Observed joint damage will be compared with beam moment DCRs based on analysis using the Santa Monica City Hall ground motion record. As noted in Section 4, this record, which is very close to Site 7, represents the best estimate available of the actual ground motion that occurred at this site during the 1994 Northridge Earthquake.

As a first attempt to correlate damage with DCRs, the results of the 3-D analysis using the Trial 4 model parameters will be used. Of all the elastic analyses conducted for this study, the Trial 4 3-D analysis is considered by the writers to be the most realistic. For this analysis, DCRs were computed only for the frames on lines F and G, in the transverse direction of the building. These DCRs are plotted in Figs. 10 and 11, and are listed in Table 8. By examining Table 8, it appears that there are some correlations between beam moment DCR and damage. A number of joints with the very highest DCRs in this table experienced severe damage. Likewise, a number of joints with the very lowest DCRs experienced minor or no damage. However, there also appear to be a number of contradictions to this trend. For example, some joints with very high DCRs experienced no damage, while some joints with very low DCRs experienced moderate damage. Thus, while there may be some general trends between beam moment DCR and damage, there appear to be many exceptions to this trend.

To further examine damage correlations, beam moment DCRs were averaged over a number of joints, and compared with damage observations. This was done for both the 2-D and the 3-D elastic analyses. Table 9 shows the average beam moment DCR for the three classifications of joint damage (severe, moderate, minor or none), based on the 2-D analysis with the Case 4 model parameters. This table shows some general correlations. The average DCR for joints sustaining severe damage was higher than for the other damage classifications. However, the average DCR for joints sustaining moderate damage is nearly the same as for the joints sustaining minor or no damage. Table 10 shows similar results using the DCRs computed in the 3-D analysis. Note that the average DCRs in Table 10 (3-D analysis) are considerably higher than in Table 9 (2-D analysis), indicating the differences between the 2-D and 3-D models. The DCRs in Table 10 show a similar trend to the 2-D results. The average DCR for joints sustaining severe damage is noticeably higher than for the other damage classifications.

Table 11 lists average beam moment DCRs for all moment frame joints in the building. These were determined from the 3-D elastic analysis, Trial 4a. This table again shows a reasonable correlation between average DCRs and the type of damage observed at the joints. Finally, Table 12 shows beam moment DCRs averaged by floor level. These averages again include all moment frame joints in the building. The correlations in Table 12 are not as clear as in Table 11. In Table 12, the 2nd and 3rd floors show high average DCRs, and also show a large number of severely damaged joints. However, the 6th floor shows equally high DCRs, but has very little joint damage. Thus, it appears that there is a stronger correlation between beam moment DCR and damage when the DCRs are averaged over the entire building, rather than being averaged on a floor by floor basis.

Based on the above observations, it appears that for this building, beam moment DCRs determined from an elastic analysis have some value. There is a correlation, in an average sense, between beam moment DCR and the level of joint damage. The DCRs cannot necessarily be used to pick out specific joints that are damaged with great certainty. However, it appears that inspecting joint locations with the highest predicted beam moment DCRs would increase the probability of locating damaged joints for this building. It must be emphasized, however, that there are many exceptions to the trend. It is also observed that the DCR value at a particular joint varies considerably based on the modeling assumptions made by the analyst. Thus it may be difficult to associate significance with a particular absolute value of DCR. For example, it is not possible to say that average DCRs above the value of "X" correlate with the most severely damaged joints, since the value of "X" will depend on the modeling assumptions. What is of greater significance is the comparative value of DCRs among different joints for a particular model. While the absolute value of beam DCRs will vary as the model parameters vary, it appears that for a given model, the highest average DCRs can still be expected to correlate with the most severely damaged joints for this particular building.

It is emphasized again that there are many exceptions to the correlation between beam moment DCR and damage, and the correlations appear most significant on an average basis. The numerous exceptions to the correlation suggest that factors not considered in the analysis also influence the likelihood of damage. Factors such as variations in weld quality, variations in steel yield strength, etc. may have as strong or stronger an influence on the likelihood of damage at a joint as does the level of beam bending moment.

Predicted Response for Other Ground Motions

As part of the benchmark 2-D elastic analysis, the frames along lines F and G were analyzed for a variety of ground motion inputs. The results of these analyses are summarized in Table 5 and in Figs. 6 and 7. These can be compared with the response predictions for the Santa Monica City Hall record for the benchmark 2-D model (Model Case 1) in Table 6 and in Figs. 8 and 9. These analyses permit a comparative assessment of the structural demands on these frames for the various records.

Comparing the frame response for the various records suggests that the Santa Monica City Hall record demanded less of this building than any of the other ground motion inputs considered. Of the Northridge Earthquake records considered in these analyses, both the Canoga Park (cnpk.106) and Sylmar (sylm.090) records showed higher beam moment DCRs and story drift ratios. The Sylmar record caused particularly high demands. Likewise, the 1978 Tabas record as well as the Equal Hazard Spectrum caused beam moment DCRs and story drift ratios much higher than predicted for the Santa Monica City Hall record. These comparisons suggest that other strong motion records may cause significantly more damage to this building than was experienced at this site in the 1994 Northridge earthquake.

Uncertainties in Elastic Analysis

There are always considerable uncertainties in modeling structural response under strong earthquake ground motions. These numerous uncertainties indicate that there is considerable judgment required in the interpretation of the analysis. This section briefly considers some of the sources of uncertainty and possible error in the model.

First and foremost, it must be recognized that inelastic frame response is expected under strong ground motion inputs, and so the results of an elastic analysis are questionable at best. However, even if the response was expected to be elastic, there are still numerous sources of uncertainty and possible error in the elastic models used in this study:

- Composite slab effects are not modeled.
- The panel zone is not accurately modeled.
- Non-structural elements are not modeled.
- The gravity load framing is not modeled.
- Modeling of viscous damping is uncertain.
- The actual yield strength of the structural members is uncertain (will affect predicted DCRs).
- Soil-structure interaction effects are not modeled.
- Etc.

The uncertainties in the structural model, combined with uncertainties in ground motion suggest the need for significant caution and engineering judgment when drawing conclusions from the earthquake analysis of building frames.

6.0 INELASTIC ANALYSES

6.1 OVERVIEW

This chapter summarizes the results of the 2-D inelastic analysis conducted for the Site 7 building. For this analysis, the four moment frames in the building's transverse direction were modeled. These are the frames on lines D, E, F, and G (see Fig. 1). The transverse direction of the building was chosen because the framing is somewhat simpler and more regular than in the longitudinal direction. The four frames were connected at the floor levels to impose common horizontal displacements at the floor levels. The boundary conditions at the base of the frames were modeled in the same way as for the elastic analyses (see Fig. 3).

Two different inelastic models were constructed. The first was a "baseline" model, which represented relatively simple inelastic models for the frame members. The second was a refined model that used more accurate inelastic models. The models are described below. Both models were constructed and run on the ANSR-1 computer program. The refined model used a number of element subroutines developed by one of the writers.

The following loading cases were considered for inelastic analysis:

- Static pushover analysis, using a UBC specified distribution of lateral force over the height of the frame, including the "F_t" term at the roof level;
- 1994 Northridge - Santa Monica City Hall record (smch.045);
- 1994 Northridge - Sylmar Olive View Hospital record (sylm.000);
- 1994 Northridge - Newhall record (newh.000);
- 1978 Iran - Tabas record (taba.344);
- 1992 Landers - Lucerne record (luc.270);
- Simulated Elysian Park record - Station 7, North component (elpark7.n);
- 1985 Mexico City - SCT-1 record, N90E component (mexico)

All of the above loading cases were run using the refined inelastic model. Only the static pushover and the sylm.000 record were run for the baseline model.

6.2 DESCRIPTION OF MODELS

This section summarizes key aspects of the baseline and refined inelastic models.

Baseline Model
Following is a list of key modeling assumptions used for the baseline model:

- Bare steel frames were modeled, without participation of the composite slabs.

- Beams and columns were modeled using a bilinear representation of beam hinge moment-rotation characteristics. A 2% strain hardening ratio was used.

- Panel zones were explicitly modeled, using a bilinear representation of panel zone moment (unbalanced beam moment at joint) versus panel zone rotation. Shear stiffness of the panel zone was computed as $G t_{cw} d_c$. The panel zone was assumed to yield when the panel zone shear reached the shear strength value specified by Eq. 2710-1 of the 1991 UBC. No strain hardening was included.

- The building weights shown in Table 2 were used to compute the seismic mass for the model.

- Yield values for the elements were based on the average statistical yield stress values listed in Table 3.

- No gravity loads were included on the moment frames. This was based on the observation from elastic analysis that gravity load effects were minimal for these frames.

- Frame P-Delta effects were included. The entire gravity load on each floor was placed on a fictitious column linked to the moment frames.

- Mass and stiffness proportional damping factors were chosen to provide 2% damping in the first and fourth modes.

- No participation of the "gravity" columns was considered in the analysis. Thus, it was assumed that all lateral stiffness and strength for the structure is provided by the moment frames.

Refined Model
The refined inelastic model incorporates some element subroutines recently developed to model steel moment frame components. The models are described in detail by Kim (1995). A brief description of the elements is provided below:

- Composite Beam Element.
  The composite beam element used in the refined model includes different stiffnesses and strengths for positive and negative moments. Pinching effects due to closing of cracks in the slab under positive moment is modeled. Hysteretic response under negative moment is curvilinear based on bounding surface modeling approaches. The model accounts for variations in the location of the inflection point in the composite beam. Figure 12

illustrates the basic outline of the hysteretic model for this element. The various model parameters were chosen based on calibration of the model to experimental data (Kim 1995). Note that this model does not simulate connection failure.

- Panel Zone Element.
  The basic outline of the panel zone hysteretic model is illustrated in Fig. 13. For the first half-cycle of loading, the response follows a trilinear monotonic type model. After the first half cycle of inelastic response, the model follows a curvilinear response based on a bounding surface modeling approach. Model parameters are chosen based on calibration to experimental data. The parameters are specifically chosen to model panel zones in frames with composite slabs. One of the significant effects of the slab is to increase the moment arm between the tension and compression force resultants at the face of the column. This effectively reduces the amount of panel zone shear force developed for a given value of unbalanced moment.

- Column Element.
  The column element is based on a trilinear hysteretic model. The model includes the interaction of flexure and axial force on the hinge properties, and includes both plastic flexural hinge rotations and plastic axial deformations.

The elements described above have undergone a rather extensive calibration and verification process by comparison with experimental data and by comparison with more sophisticated inelastic models (Kim 1995).

Besides the incorporation of the element subroutines described above, the refined model was the same as the baseline model. In the discussion of analysis results in the following section, the models are referred to as follows:

Refined model: "Model I"

Baseline model: "Model II"

6.3 RESULTS AND DISCUSSION OF INELASTIC ANALYSIS

The results of the inelastic analysis are provided in detail in Appendix C. This section summarizes some of the highlights of the analytical results.

The results of the static pushover analysis are shown in Figure 14 and in Table 13. Figure 14 plots the lateral force on the frame versus roof displacement for the two inelastic models. Note that the refined model (Model I) shows somewhat higher strength and stiffness than the baseline model. This can be attributed largely to the inclusion of composite beam effects in Model I. The higher stiffness of Model I also resulted in somewhat lower natural periods. The first natural period of Model I was 1.97 seconds, versus 2.25 seconds for Model II. The maximum lateral strength of the frame is predicted to be approximately 0.13 W for Model II, and 0.16 W for Model

I. P-Delta effects are also clearly evident in Fig. 14.

As an example of the inelastic response of the two models to the static pushover loading, Table 13 lists the maximum plastic rotations developed in each model for the 2nd floor joints. The joint location designation in the table indicates the frame and grid line location of the joint. For example, "G3" denotes the joint located at grid line 3, for the beam in the frame on line G. Table 13 shows the location and degree of yielding at the joints. Note that Model II (baseline model) predicts that all yielding will occur in the beams, with no inelastic action in the panel zones. Model II, one the other hand, indicates significant sharing of inelastic action between the beam and panel zone at many of the joints. Table 13 suggests that the predicted location of yielding within a joint is quite sensitive to the inelastic model parameters. Thus, accuracy in modeling the inelastic behavior of the beams, panel zones, and columns is quite important for developing realistic estimates of plastic rotation demands.

The remainder of this section discusses the results of the inelastic time history analyses for the ground motion records listed in Section 6.1. Unless otherwise noted, the results are based on the refined model (Model I).

For each of the ground motion records, Fig. 15 shows the roof displacement versus time, and the envelope of maximum story drift ratios. The maximum story drift ratios are also listed in Table 15. The maximum story drift ratios predicted under the smch.045 record (best estimate of actual ground motion) is 1.7%. For all other records considered in this analysis, including the two other Northridge records (Sylmar and Newhall), the building would have experienced substantially higher drifts, on the order of 3 to 3.5%. The response to the simulated Elysian Park record is particularly large. Drift ratios up to nearly 6% occur at the building's upper level for this record. The structure also experiences large drifts under the Mexico City SCT-1 record. The peak in the acceleration response spectrum for this record occurs at a period of 2 seconds, very nearly equal to the first natural period of the structure. For most of the records, other than the smch.045 record, the analysis predicts the building will develop a permanent offset at the roof level of 10 to 15 inches. The results shown in Fig. 15 suggest that the Site 7 building would be expected to sustain significantly more damage for the strong motion records considered herein, compared to the damage it sustained in the Northridge Earthquake.

Plastic rotation demands at the joints are presented in a variety of formats. For each ground motion record, Table 14 lists the maximum plastic rotations for joints on the 2nd floor. The table reports maximum plastic rotations in the beams, panel zones, and columns. This table shows several trends. First, it appears that most of the inelastic deformation occurs in the beams, rather than in the panel zones. This is particularly true for the frames on lines D, E, and G. For the frame on line F, however, substantial panel zone yielding occurs. Note that shallower beams are used in this frame, as compared to the other three frames. The shallower beams will generate higher panel zone shear forces, and therefore more inelastic action in the panel zones. Table 14 also indicates that the plastic rotation demands are rather low under the smch.045 record, compared to the other records.

Table 16 lists the maximum plastic rotations developed in the beams, panel zones, and columns under the various records. In developing these maximum values, all joints in all four of the frames were considered. Under the smch.045 record, a maximum beam plastic rotation of 0.010 rad. is predicted, with essentially no inelastic action in the panel zones or columns. This suggests that this building experienced only minor inelastic demands in the Northridge Earthquake. For the remainder of the records, substantially higher plastic rotation demands are predicted. For many of the records, maximum beam plastic rotations are on the order of 0.02 to 0.03 rad. As before, the Elysian Park and Mexico City records show extraordinarily high plastic rotation demands. The maximum beam plastic rotations listed in Table 16 were not isolated values that occurred at only a small number of joints. Rather, for most of the records, beam plastic rotations at or near the maximum values listed in Table 16 occurred at a large number of joints within the structure. Thus, any connection problems associated with the development of inelastic action in the beams might be expected to be rather widespread throughout the frames.

Table 16 also lists the maximum plastic rotation developed in the panel zones and columns. The maximum panel zone plastic rotation always occurred at a joint in the frame on line F. Table 16 also indicates the formation of column hinges under most of the records. With the exception of the Elysian Park record, column hinges formed only at the ground level, as would be anticipated. Under the Elysian Park record, however, particularly severe column yielding occurred at the upper story.

Figure 16 shows plastic rotations for the joints in the frame on line G. The number plotted over the beam-column intersection is the maximum panel zone plastic rotation, the number over the beam end is the maximum beam plastic rotation, etc. Results are plotted for three records: smch.045, sylm.000, and elpark7.n. These plots show that beam plastic rotations are fairly uniformly distributed over the height of the frame, between the 2nd floor and the 6th floor. No beam yielding is predicted at the ground or roof levels. Figure 16(c) confirms the very high inelastic demands on this frame under the Elysian Park record.

Table 17 lists the maximum beam plastic rotation developed at each joint of the four frames under the smch.045 record. Recall that this record represents the best estimate of the actual ground motion experienced at this site during the Northridge Earthquake. Also listed in this table is the type of damage observed at each joint. The damage classifications are defined in Section 3.

As with the elastic DCRs, there appears to be some correlation between beam plastic rotation demand and the observed damage. Many of the joints with the highest plastic rotations sustained severe damage. Further, many of the joints with the lowest plastic rotation demands sustained minor or no damage. Unfortunately, however, there are again numerous exceptions to this trend. There are several joints with very low or zero plastic rotations that sustained severe damage. Likewise, there are many joints with large plastic rotations that experienced no damage.

To further examine damage correlations, beam plastic rotations were averaged over all joints for each of the three damage classifications. The results are shown in Table 18. This table suggests a correlation, on an averaged basis, between beam plastic rotation level and damage. The correlation, however, does not appear to be particularly strong.

In examining the data in Table 17, there appears to be an anomaly at the 6th floor joints. The analysis predicts high plastic rotation demands at all of the 6th floor joints. Yet, these joints sustained virtually no damage. (A similar anomaly exists for the beam moment DCRs in Table 8). The frame elevations shown in Fig. 3 indicate that the beam on the 6th floor is substantially lighter than the beams at the lower levels. For all four frames, the 6th floor beam is a W24x55, which is much lighter than the W24x117 or W30x99 beams used for the 4th and 5th floors. This observation suggests that for the same level of beam plastic rotation demand, damage was more likely to occur in the heavier beams. Table 19 shows average beam plastic rotations, with the 6th floor beams excluded. The correlation between beam plastic rotation and damage is now much stronger.

Finally, as with the elastic analysis, it should be noted that there are numerous uncertainties in the inelastic analysis. Many of the uncertainties listed for the elastic analyses in Section 5.4 also apply to the inelastic analysis, i.e., gravity load framing is not modeled, steel yield strengths are highly uncertain, non-structural elements are not modeled, etc. In addition, there are large uncertainties in modeling the cyclic inelastic response of steel moment frame members and joints. Further, for this building, torsional response appears to be significant, but could not be modeled with a two-dimensional inelastic analysis. These many sources of uncertainty indicate that the analytical results must be interpreted with caution.

7.0 CONCLUSIONS

This report has summarized the results of an analytical study of a six story steel moment frame building located in Santa Monica, California (SAC Building Site 7). This building sustained significant damage to its steel moment frame joints in the 1994 Northridge Earthquake. The primary form of damage observed in this building was the fracture of beam flange welds, typically at the beam bottom flange. A large number of the building's 120 moment frame joints experienced some damage.

Analytical studies were conducted to investigate the predicted response of the building in the Northridge Earthquake, and its predicted response under other strong ground motion records. Three types of analyses were conducted in this study: two-dimensional elastic analysis, three-dimensional elastic analysis, and two-dimensional inelastic analysis. A principal objective of the study was to determine if the damage observed in this building was in any way predictable by structural analysis, and if structural analysis could be used as a tool to guide building inspections.

The primary ground motion record used in the analysis of this building was the Santa Monica City Hall record from the 1994 Northridge Earthquake. The record site is very close to the building site, and this record was considered the best estimate available of the actual ground motion experienced by the building during the Northridge Earthquake. In addition to the Santa Monica City Hall record, a variety of other strong motion records were used in the analysis. This included other records from the Northridge Earthquake, as well as records from other strong earthquakes.

The key findings of this study are summarized below:

- The results of both the elastic and inelastic analyses using the Santa Monica City Hall record showed some degree of correlation between the predicted structural response and the damage observed at the joints.

- For the elastic analysis, beam moment demand-capacity ratio (DCR) was the primary structural response parameter used as a possible indicator of joint damage. Both the 2-D and the 3-D analysis showed some correlation between beam moment DCR and damage. Many of the joints with high predicted DCRs experienced severe damage. Likewise, many of joints with very low DCRs experienced only minor or no damage. There were, however, many exceptions to the trend. That is, there were a number of joints with low DCRs that experienced severe damage, as well as joints with high DCRs that experienced no damage. Nonetheless, when averaged over a large number of joints, there appeared to be a distinct correlation between the beam moment DCR and the level of damage at a joint.

- The values of beam moment DCR predicted by elastic analysis were quite sensitive to modeling assumptions, particularly to the assumed level of damping. Even though the absolute value of DCRs varied with the elastic modeling assumptions, the relative value of DCRs among different joints in the building remained rather constant. That is, for a variety of modeling assumptions, the highest beam moment DCRs among all the joints of the building correlated with the most severely damaged joints.

- Beam plastic rotations predicted by inelastic analysis are quite sensitive to the model assumptions used for the beam, panel zone and column. Changes in the model assumptions can shift the location of yielding at a joint between the beam, panel zone and column.

- For the inelastic analysis, beam plastic rotation was the primary structural response parameter used as a possible indicator of joint damage. Two-dimensional inelastic time history analysis using the Santa Monica City Hall record showed some correlation between beam plastic rotation and damage. Many of the joints with high beam plastic rotations experienced severe damage. Likewise, many of joints with very low beam plastic rotations experienced only minor or no damage. There were, however, many exceptions to the trend. That is, there were a number of joints with low beam plastic rotations that experienced severe damage, as well as joints with high beam plastic rotations that experienced no damage. Nonetheless, when averaged over a large number of joints, there appeared to be a reasonable correlation between the beam plastic rotation and the level of damage at a joint. Correlations with damage did not appear to be significantly better for the inelastic analysis than for the elastic analysis of this building.

- For joints with approximately the same level of predicted beam plastic rotation demand, it appeared that damage was more likely to occur in the heavier beams. For the same level of plastic rotation, the lighter weight beams experienced significantly less damage. A similar observation holds for the elastic analysis. For joints with approximately the same level of predicted beam moment DCR, it appeared that damage was more likely to occur in the heavier beams.

- The results of this study suggest that an elastic structural analysis could have been used as a useful guide for inspecting this building. It appears that the chances of locating a damaged joint would be increased by first inspecting joints with the highest predicted beam moment DCRs. For joints with similar levels of beam moment DCR, the chances of locating a damaged joint would be increased by first inspecting the joints for the heaviest beams.

- The inelastic analysis indicates that this building experienced a maximum interstory drift ratio on the order of 1.7% in its transverse direction during the Northridge Earthquake. No significant nonstructural damage was observed in the building.

- The inelastic analysis indicates that the maximum beam plastic rotations experienced in this building during the Northridge Earthquake were on the order of 0.010 rad. These rather modest inelastic deformation demands suggest that the moment frame joints performed quite poorly.

- For the other ground motions considered in this study, the analyses predict significantly greater structural demands on this building (beam moment DCRs, beam plastic rotations, interstory drift ratios, etc.) than were predicted using the Santa Monica City Hall Record. Much higher levels of damage might be expected in this building under these other strong ground motions, than was experienced in the Northridge Earthquake. The simulated Elysian Park record appeared to be particularly damaging to this building.

- Inelastic analysis was also conducted using the 1985 Mexico City SCT-1 record. This soft soil record was very damaging to this building, generating very high interstory drift ratios and very high beam plastic rotations. The predominant long period response of this record was very close to the first natural period of this very flexible structure. The very high predicted values of structural response are not viewed as completely realistic, since this building was not designed for a soft soil site (which would have required a higher design base shear coefficient). Nonetheless, the analysis suggests that steel moment frames, due to their inherent flexibility, may be more vulnerable on soft soil sites.

- Excluding the Elysian Park record, the maximum beam plastic rotations developed in this structure under a variety of strong ground motions were on the order of 0.02 to 0.03 rad. The Elysian Park record developed a maximum beam plastic rotation of about 0.04 rad.

- Excluding the Elysian Park record, the maximum interstory drift ratios developed in this structure under a variety of strong ground motions were on the order of 3 to 3.5%. The Elysian Park record produced a maximum interstory drift ratio of nearly 6%.

- No attempt was made in this study to model the hysteretic response of a damaged joint. Therefore, no conclusions can be drawn on the consequences of connection damage on the response of this structure in future strong earthquakes.

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TABLE 1 - GROUND MOTION RECORDS

EARTHQUAKE	RECORD	RECORD DESIGNATION
1994 Northridge	Santa Monica City Hall	smch.000
		smch.090
		smch.045
1994 Northridge	Sylmar Olive View Hospital	sylm.000
		sylm.090
1994 Northridge	Canoga Park	cnpk.106
1994 Northridge	Newhall	newh.000
1940 Imperial Valley	El Centro	ivir.270
1978 Iran	Tabas	taba.344
1992 Landers	Lucerne	luc.270
1985 Mexico City	SCT-1	Mexico SCT-1 N90E
Simulated Elysian Park Earthquake	simulated ground motion at Station 7, N-S component	elpark7.n

TABLE 2 - BUILDING WEIGHTS USED TO COMPUTE SEISMIC MASS

Floor Level	Weight (kips)
Roof	1090
6th	1184
5th	1270
4th	1401
3rd	1420
2nd	1362
TOTAL	7727

TABLE 3 - VALUES OF YIELD STRENGTH USED TO COMPUTE M_p

Steel Grade	Yield Strength Used to Compute M_p (ksi)		
	Shape Group 1	Shape Group 2	Shape Group 3
A36	50.5	47.3	45.6
A572 Gr. 50	57.9	57.5	57.3

TABLE 4 - 2-D ELASTIC MODELS

Computer Model	Model Assumptions	First Mode Natural Period
Case 1	Benchmark	2.41 sec.
Case 2	Same as benchmark, except use 50% rigid offset at column.	2.27 sec.
Case 3	Same as benchmark, except use 100% rigid offset at column.	2.13 sec.
Case 4	Same as benchmark, except use 50% rigid offset at column, and 2% damping.	2.27 sec.
Case 5	Same as benchmark, except use 50% rigid offset at column, and neglect p-Delta effects.	2.22 sec.

TABLE 5
DEMAND-CAPACITY RATIOS FOR BEAM END MOMENTS
2-D ELASTIC ANALYSIS - BENCHMARK MODEL AND GROUND MOTIONS

MEMBER #	CAP('K)	GRAV	E.H.S	CNPK	SYLM	IVIR	TABAS
F.G.3	1636	0.02	1.25	0.54	1.37	0.49	1.11
F.G.4	1636	0.02	1.25	0.54	1.37	0.49	1.11
F.2.3	1648	0.02	2.38	1.03	2.64	0.94	2.01
F.2.4	1648	0.02	2.38	1.03	2.64	0.94	2.01
F.3.3	1648	0.02	2.09	0.87	2.34	0.83	1.80
F.3.4	1648	0.02	2.09	0.87	2.34	0.83	1.80
F.4.3	1289	0.03	2.20	0.88	2.68	1.01	2.03
F.4.4	1289	0.02	2.20	0.88	2.68	1.01	2.03
F.5.3	1289	0.03	2.01	0.95	2.65	0.94	1.92
F.5.4	1289	0.02	2.01	0.95	2.65	0.94	1.92
F.6.3	564	0.08	2.75	1.30	3.40	1.19	2.73
F.6.4	564	0.07	2.75	1.30	3.40	1.19	2.73
F.R.3	564	0.13	1.51	0.80	1.73	0.64	1.51
F.R.4	564	0.13	1.51	0.80	1.73	0.64	1.51
G.G.3	1636	0.01	1.28	0.55	1.40	0.50	1.13
G.G.4	1636	0.02	1.28	0.55	1.40	0.50	1.13
G.2.3	1636	0.01	2.73	1.18	3.03	1.08	2.30
G.2.4	1636	0.02	2.73	1.18	3.03	1.08	2.30
G.3.3	1636	0.01	2.40	1.00	2.70	0.95	2.08
G.3.4	1636	0.02	2.40	1.00	2.70	0.95	2.08
G.4.3	1230	0.02	2.48	1.02	3.03	1.14	2.30
G.4.4	1230	0.03	2.48	1.02	3.03	1.14	2.30
G.5.3	1230	0.02	2.22	1.05	2.92	1.04	2.12
G.5.4	1230	0.02	2.22	1.05	2.92	1.04	2.12
G.6.3	564	0.05	2.82	1.34	3.48	1.23	2.81
G.6.4	564	0.06	2.82	1.34	3.48	1.23	2.81
G.R.3	962	0.08	1.12	0.60	1.28	0.47	1.12
G.R.4	962	0.09	1.12	0.60	1.28	0.47	1.12

TABLE 6
DEMAND-CAPACITY RATIOS FOR BEAM END MOMENTS
2-D ELASTIC ANALYSIS
SANTA MONICA CITY HALL RECORD (SMCH.045)
VARIOUS STRUCTURAL MODELS

MEMBER #	CAP('K)	GRAV	CASE1	CASE2	CASE3	CASE4	CASE5
F.G.3	1636	0.02	0.35	0.38	0.37	0.58	0.36
F.G.4	1636	0.02	0.35	0.38	0.37	0.58	0.36
F.2.3	1648	0.02	0.70	0.87	1.05	1.30	0.81
F.2.4	1648	0.02	0.70	0.87	1.05	1.30	0.81
F.3.3	1648	0.02	0.72	0.82	1.01	1.18	0.82
F.3.4	1648	0.02	0.72	0.82	1.01	1.18	0.82
F.4.3	1289	0.03	0.79	0.98	1.12	1.37	0.97
F.4.4	1289	0.02	0.79	0.98	1.12	1.37	0.97
F.5.3	1289	0.03	0.64	0.89	1.04	1.29	0.90
F.5.4	1289	0.02	0.64	0.89	1.04	1.29	0.90
F.6.3	564	0.08	0.90	1.25	1.45	1.64	1.29
F.6.4	564	0.07	0.90	1.25	1.45	1.64	1.29
F.R.3	564	0.13	0.50	0.78	0.87	1.05	0.81
F.R.4	564	0.13	0.50	0.78	0.87	1.05	0.81
G.G.3	1636	0.01	0.36	0.40	0.40	0.60	0.38
G.G.4	1636	0.02	0.36	0.40	0.40	0.60	0.38
G.2.3	1636	0.01	0.80	1.03	1.28	1.54	0.96
G.2.4	1636	0.02	0.80	1.03	1.28	1.54	0.96
G.3.3	1636	0.01	0.83	0.97	1.24	1.40	0.97
G.3.4	1636	0.02	0.83	0.97	1.24	1.40	0.97
G.4.3	1230	0.02	0.89	1.13	1.31	1.58	1.12
G.4.4	1230	0.03	0.89	1.13	1.31	1.58	1.12
G.5.3	1230	0.02	0.70	1.00	1.19	1.45	1.01
G.5.4	1230	0.02	0.70	1.00	1.19	1.45	1.01
G.6.3	564	0.05	0.92	1.29	1.50	1.69	1.34
G.6.4	564	0.06	0.92	1.29	1.50	1.69	1.34
G.R.3	962	0.08	0.37	0.62	0.74	0.83	0.64
G.R.4	962	0.09	0.37	0.62	0.74	0.83	0.64

TABLE 7 - SUMMARY OF 3-D ELASTIC MODELS

Model Assumption	Trial 1	Trial 2	Trial 3	Trial 4	Trial 4a	Trial 5
Mass Moment of Inertia	approx.	calculated	calculated	calculated	calculated	calculated
Gravity Loads	1.0D+.08L	1.0D+.5L	1.0D+.5L	1.0D+.5L	none	1.0D+.5L
Joint Model	centerline	centerline	centerline	50% rigid offset	50% rigid offset	50% rigid offset
Vertical Acceleration	no	yes	no	no	no	no
Torsion Lock	no	no	no	no	no	yes
Damping	5%	5%	5%	2%	2%	2%

TABLE 8
DEMAND-CAPACITY RATIOS FOR BEAM END MOMENTS
3-D ELASTIC ANALYSIS - TRIAL 4 MODEL
SANTA MONICA CITY HALL RECORD

Beam End Location			Beam End Moment DCR	Joint Damage
Frame Line	Floor Level	Grid Line at Beam End		
F	Ground	3	.77	N
		4	.77	N
	2nd	3	1.66	N
		4	1.65	M
	3rd	3	1.49	S
		4	1.49	S
	4th	3	1.66	N
		4	1.65	S
	5th	3	1.46	M
		4	1.46	N
	6th	3	1.90	N
		4	1.89	N
	Roof	3	1.00	M
		4	.99	N
G	Ground	3	.99	N
		4	.99	N
	2nd	3	2.17	S
		4	2.18	S
	3rd	3	1.93	S
		4	1.94	S
	4th	3	2.08	S
		4	2.09	S
	5th	3	1.60	M
		4	1.60	S
	6th	3	1.92	N
		4	1.91	N
	Roof	3	.80	N
		4	.80	M

Joint Damage: S=Severe M=Moderate N=Minor or None

TABLE 9
COMPARISON OF BEAM DCRs WITH JOINT DAMAGE
2-D ELASTIC ANALYSIS - CASE 4 MODEL
FRAMES ON LINES F AND G

Joint Damage Type	Number of Joints	Average DCR
Severe	10	1.42
Moderate	5	1.18
Minor or None	13	1.14

TABLE 10
COMPARISON OF BEAM DCRs WITH JOINT DAMAGE
3-D ELASTIC ANALYSIS - TRIAL 4 MODEL
FRAMES ON LINES F AND G

Joint Damage Type	Number of Joints	Average DCR
Severe	10	1.86
Moderate	5	1.30
Minor or None	13	1.36

TABLE 11
COMPARISON OF BEAM DCRs WITH JOINT DAMAGE
3-D ELASTIC ANALYSIS - TRIAL 4a MODEL
ALL MOMENT FRAMES IN BUILDING

Joint Damage Type	Number of Joints	Average DCR
Severe	30	2.03
Moderate	30	1.80
Minor or None	60	1.62

TABLE 12
COMPARISON OF BEAM DCRs WITH JOINT DAMAGE FOR EACH FLOOR LEVEL
3-D ELASTIC ANALYSIS - TRIAL 4a MODEL
ALL MOMENT FRAMES IN BUILDING

Floor Level	Number of Joints	Average DCR	Number of Joints with Damage Type		
			Severe	Moderate	Minor or None
Ground	16	1.43	0 (0%)	0 (0%)	16 (100%)
2nd	16	2.26	9 (56%)	5 (31%)	2 (13%)
3rd	16	2.03	12 (75%)	4 (25%)	0 (0%)
4th	16	1.92	8 (50%)	2 (12%)	6 (38%)
5th	24	1.43	1 (4%)	12 (50%)	7 (46%)
6th	16	2.21	0 (0%)	2 (12%)	12 (75%)
Roof	16	1.23	0 (0%)	4 (25%)	12 (75%)

TABLE 13
STATIC PUSHOVER ANALYSIS
PLASTIC ROTATIONS AT 2ND FLOOR JOINTS
MODEL I VERSUS MODEL II

Static Pushover Analysis : Model I

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.05215	0.00000	0.00000	-0.00799	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.02917	0.02756	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.05217	0.00000	0.00000	-0.00799	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.02917	0.02756	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.02656	0.00000	0.00000	-0.03065	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.01179	0.04396	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.05220	0.00000	0.00000	-0.00800	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.02916	0.02755	0.00000	0.00000	0.00000	0.00000	0.00000

Static Pushover Analysis : Model II

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.06050	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.06052	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.05754	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.05754	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

TABLE 14
INELASTIC DYNAMIC ANALYSIS
PLASTIC ROTATIONS AT 2ND FLOOR JOINTS

Analysis of Model I : SMCH.O45

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00824	-0.00051	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00097	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00821	-0.00054	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00096	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00332	-0.00066	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00202	-0.00106	0.00000	0.00000	0.00000	0.00000
G3	0.00811	-0.00063	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00095	-0.00226	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 2nd

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01789	-0.00505	0.00051	-0.00038	0.00000	0.00000	0.00000	0.00000
D4	0.00768	-0.01172	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01791	-0.00507	0.00054	-0.00037	0.00000	0.00000	0.00000	0.00000
E4	0.00767	-0.01172	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00045	0.00000	0.00474	-0.01498	0.00000	0.00000	0.00000	0.00000
F4	0.00736	0.00000	0.01677	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01796	-0.00515	0.00062	-0.00030	0.00000	0.00000	0.00000	0.00000
G4	0.00766	-0.01173	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00932	-0.02041	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00939	-0.02046	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00934	-0.02042	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00939	-0.02046	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00627	-0.01731	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00628	-0.01732	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00940	-0.02047	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00940	-0.02047	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

TABLE 14
INELASTIC DYNAMIC ANALYSIS
PLASTIC ROTATIONS AT 2ND FLOOR JOINTS
(CONTINUED)

Analysis of Model I : NEWH.000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01320	-0.00384	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00703	-0.00903	0.00148	-0.00053	0.00000	0.00000	0.00000	0.00000
E3	0.01173	-0.00386	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00702	-0.00903	0.00148	-0.00053	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00368	-0.00970	0.00000	0.00000	0.00000	0.00000
F4	0.00254	0.00000	0.01100	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01179	-0.00399	0.00002	-0.00001	0.00000	0.00000	0.00000	0.00000
G4	0.00697	-0.00903	0.00149	-0.00052	0.00000	0.00000	0.00000	0.00000

Analysis of Model I : 1978 Iran Earthquake: Taba.344

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02799	-0.00058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00614	-0.01323	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.02772	-0.00061	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00615	-0.01323	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01814	0.00000	0.00869	-0.00615	0.00000	0.00000	0.00000	0.00000
F4	0.01261	-0.00383	0.02371	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.02904	-0.00071	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00614	-0.01324	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000

Analysis of Model I : LUC.270

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01862	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01864	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	-0.01583	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.01723	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01870	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000

TABLE 14
INELASTIC DYNAMIC ANALYSIS
PLASTIC ROTATIONS AT 2ND FLOOR JOINTS
(CONTINUED)

Analysis of Model I : ELPARK7.N

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02278	-0.01189	0.00855	0.00000	0.00231	0.00000	0.00000	0.00000
D4	0.01943	-0.01376	0.00445	-0.00239	0.00189	0.00000	0.00000	0.00000
E3	0.02280	-0.01191	0.00857	0.00000	0.00234	0.00000	0.00000	0.00000
E4	0.01942	-0.01376	0.00445	-0.00239	0.00186	0.00000	0.00000	0.00000
F3	0.01343	-0.00468	0.01341	-0.00696	0.00248	0.00000	0.00000	0.00000
F4	0.01530	0.00000	0.01983	-0.00313	0.00067	0.00000	0.00000	0.00000
G3	0.02295	-0.01194	0.00863	0.00000	0.00246	0.00000	0.00000	0.00000
G4	0.01943	-0.01377	0.00446	-0.00238	0.00179	0.00000	0.00000	0.00000

Analysis of Model I : 1985 Mexico SCT-1 N90E

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04559	-0.00523	0.00549	-0.00007	0.00000	0.00000	0.00000	0.00000
D4	0.01683	-0.01966	0.01342	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.04701	-0.00524	0.00553	-0.00004	0.00000	0.00000	0.00000	0.00000
E4	0.01679	-0.01968	0.01341	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.03665	0.00000	0.01961	-0.00175	0.00000	0.00000	0.00000	0.00000
F4	0.02507	0.00000	0.03498	-0.00271	0.00000	0.00000	0.00000	0.00000
G3	0.04695	-0.00529	0.00567	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01601	-0.01969	0.01340	0.00000	0.00000	0.00000	0.00000	0.00000

TABLE 15
INELASTIC DYNAMIC ANALYSIS
MAXIMUM STORY DRIFT RATIOS

Earthquake Record	Maximum Story Drift Ratio
smch.045	1.7 % (5th - 6th floor)
sylm.000	2.8 % (5th - 6th floor)
newh.000	3.0 % (4th - 5th floor)
taba.344	3.5 % (2nd - 3rd floor)
luc.270	3.1 % (2nd - 3rd floor)
elpark7.n	5.8 % (6th - roof)
mexico	4.4 % (2nd- - 3rd floor)

TABLE 16
INELASTIC DYNAMIC ANALYSIS
MAXIMUM PLASTIC ROTATIONS

Earthquake Record	Maximum Plastic Rotation (Rad.)		
	Beam	Panel Zone	Column
smch.045	.010	.003	0
sylm.000	.018	.017	.013
newh.000	.019	.011	.005
taba.344	.029	.024	.017
luc.270	.020	.017	.010
elpark7.n	.042	.020	.045
mexico	.047	.035	.018

TABLE 17
INELASTIC DYNAMIC ANALYSIS
SANTA MONICA CITY HALL RECORD (SMCH.045)
MAXIMUM BEAM PLASTIC ROTATIONS

Beam End Location			Maximum Beam Plastic Rotation (rad.)	Joint Damage
Frame Line	Floor Level	Grid Line at Beam End		
D	Ground	3	0	N
		4	0	N
	2nd	3	.008	S
		4	.002	S
	3rd	3	.002	S
		4	.008	S
	4th	3	.003	S
		4	.008	S
	5th	3	.003	N
		4	.009	M
	6th	3	.007	N
		4	.010	M
	Roof	3	0	N
		4	0	M
E	Ground	3	0	N
		4	0	N
	2nd	3	.008	N
		4	.002	S
	3rd	3	.002	S
		4	.008	S
	4th	3	.003	S
		4	.008	S
	5th	3	.003	N
		4	.009	M
	6th	3	.007	M
		4	.010	N
	Roof	3	0	M
		4	0	N

Joint Damage: S=Severe M=Moderate N=Minor or None

TABLE 17 (CONT.)
INELASTIC DYNAMIC ANALYSIS
SANTA MONICA CITY HALL RECORD (SMCH.045)
MAXIMUM BEAM PLASTIC ROTATIONS

Beam End Location			Maximum Beam Plastic Rotation (rad.)	Joint Damage
Frame Line	Floor Level	Grid Line at Beam End		
F	Ground	3	0	N
		4	0	N
	2nd	3	0	N
		4	0	M
	3rd	3	0	S
		4	0	S
	4th	3	.006	N
		4	.001	S
	5th	3	.002	M
		4	0	N
	6th	3	.007	N
		4	.010	N
	Roof	3	0	M
		4	0	N
G	Ground	3	0	N
		4	0	N
	2nd	3	.008	S
		4	.002	S
	3rd	3	.002	S
		4	.008	S
	4th	3	.002	S
		4	.008	S
	5th	3	.003	M
		4	.009	S
	6th	3	.007	N
		4	.010	N
	Roof	3	0	N
		4	0	N

Joint Damage: S=Severe M=Moderate N=Minor or None

TABLE 18
INELASTIC DYNAMIC ANALYSIS
SANTA MONICA CITY HALL RECORD (SMCH.045)
COMPARISON OF BEAM PLASTIC ROTATIONS WITH DAMAGE

Joint Damage Type	Number of Joints	Average Maximum Beam Plastic Rotation
Severe	21	.0045 rad.
Moderate	10	.0040 rad.
Minor or None	25	.0028 rad.

TABLE 19
INELASTIC DYNAMIC ANALYSIS
SANTA MONICA CITY HALL RECORD (SMCH.045)
COMPARISON OF BEAM PLASTIC ROTATIONS WITH DAMAGE

BEAMS ON 6TH FLOOR EXCLUDED

Joint Damage Type	Number of Joints	Average Maximum Beam Plastic Rotation
Severe	21	.0045 rad.
Moderate	8	.0028 rad.
Minor or None	19	.0010 rad.

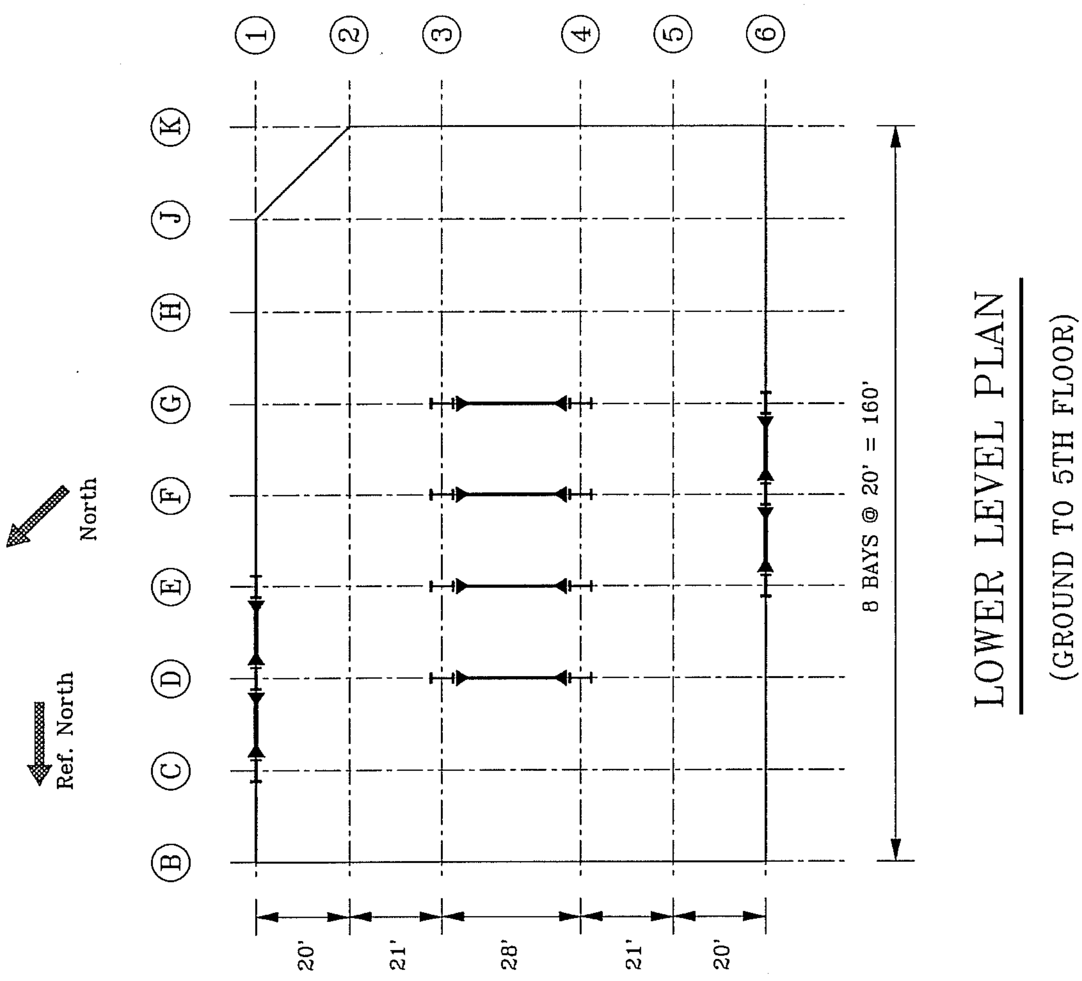


FIGURE 1 - BUILDING PLAN AT LOWER LEVELS

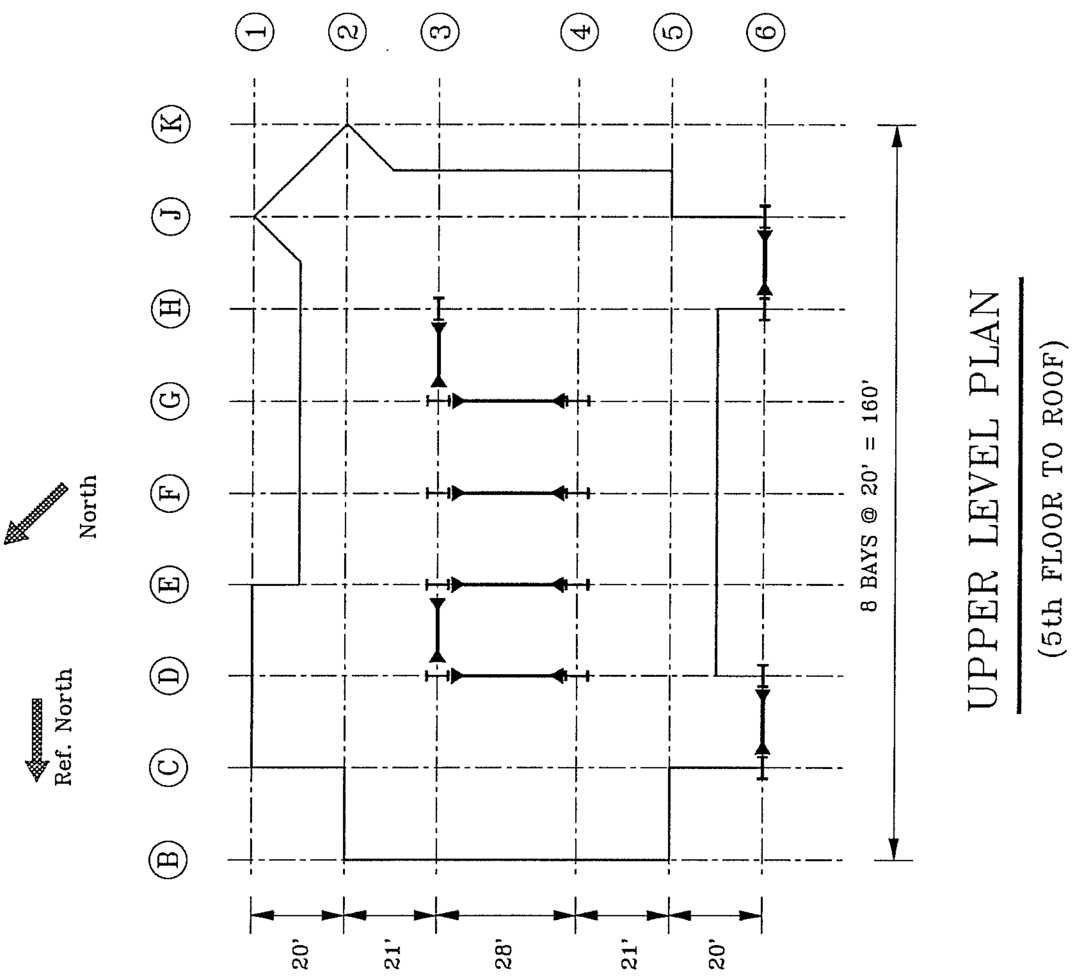


FIGURE 2 - BUILDING PLAN AT UPPER LEVELS

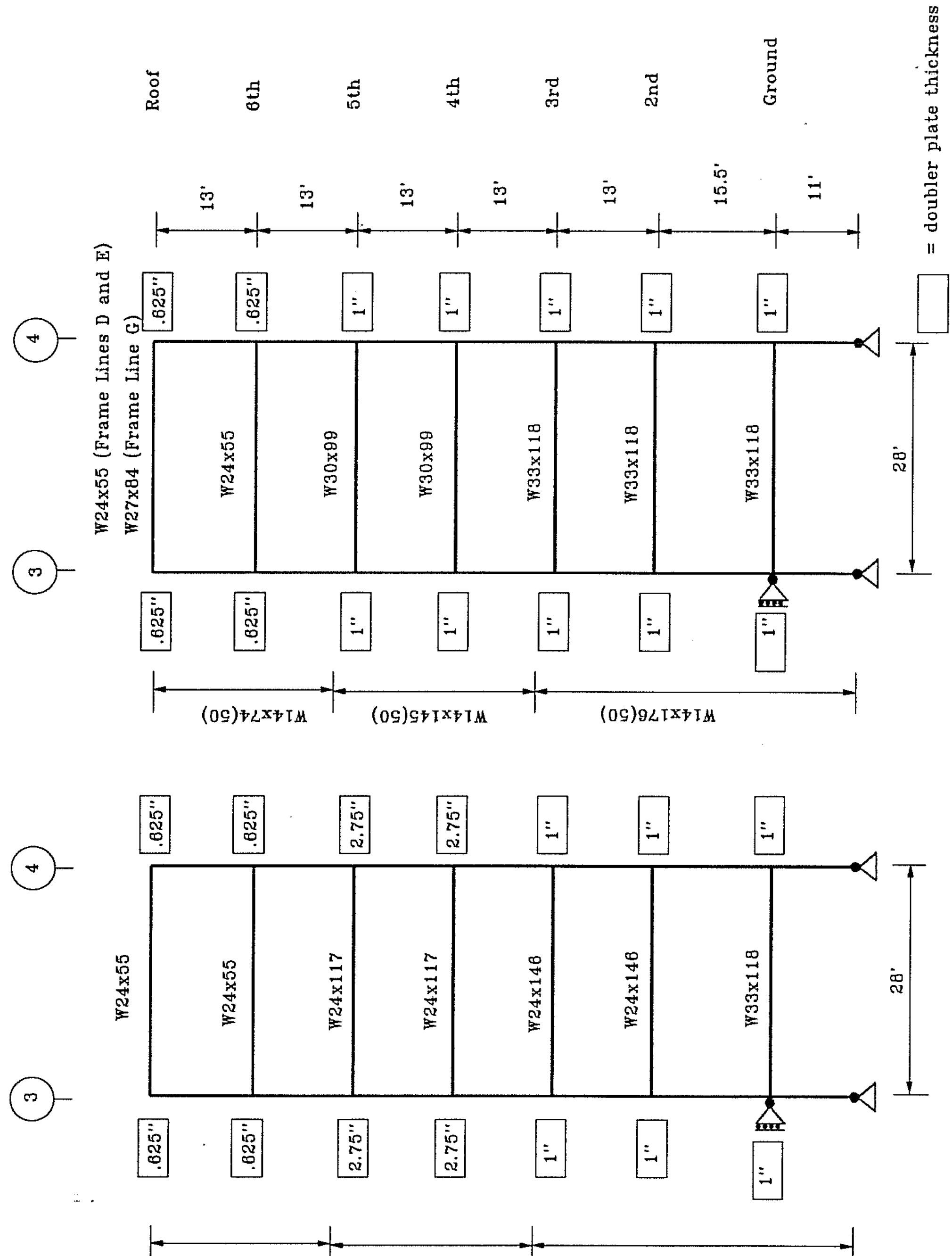


FIGURE 3 - FRAME ELEVATIONS FOR TRANSVERSE DIRECTION OF BUILDING

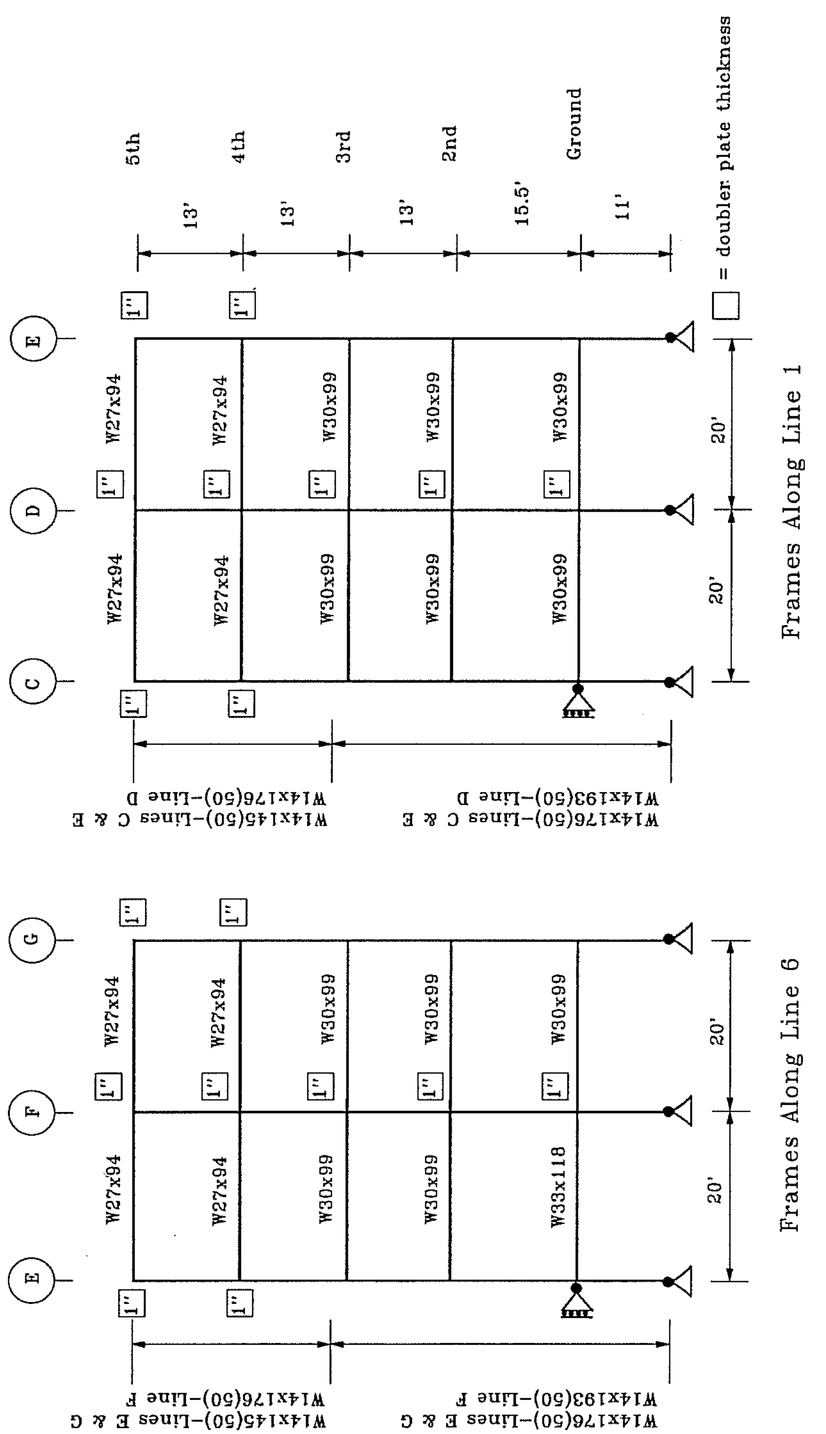


FIGURE 4 - FRAME ELEVATIONS FOR LONGITUDINAL DIRECTION OF BUILDING

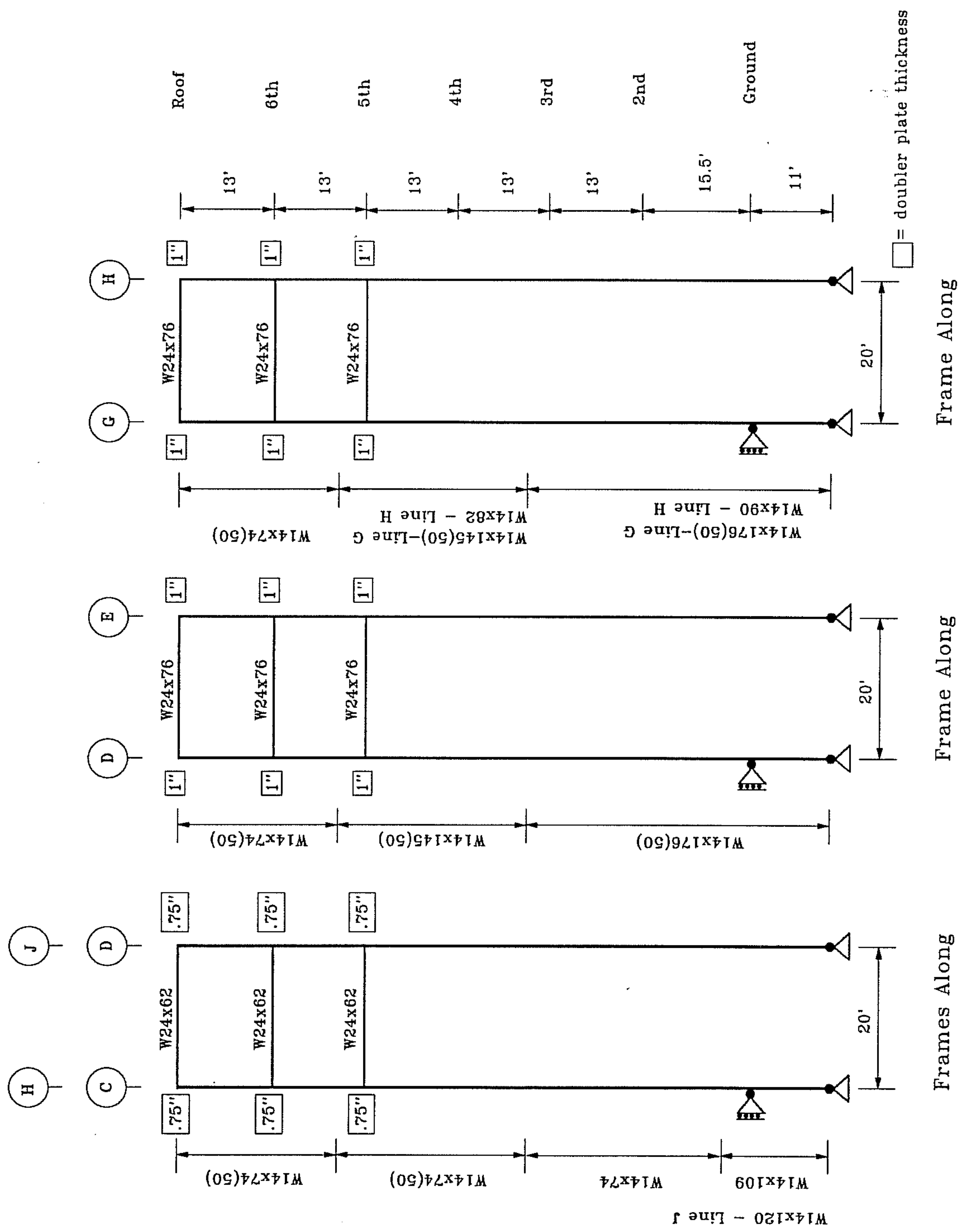


FIGURE 4 - FRAME ELEVATIONS FOR LONGITUDINAL DIRECTION OF BUILDING (CONT.)

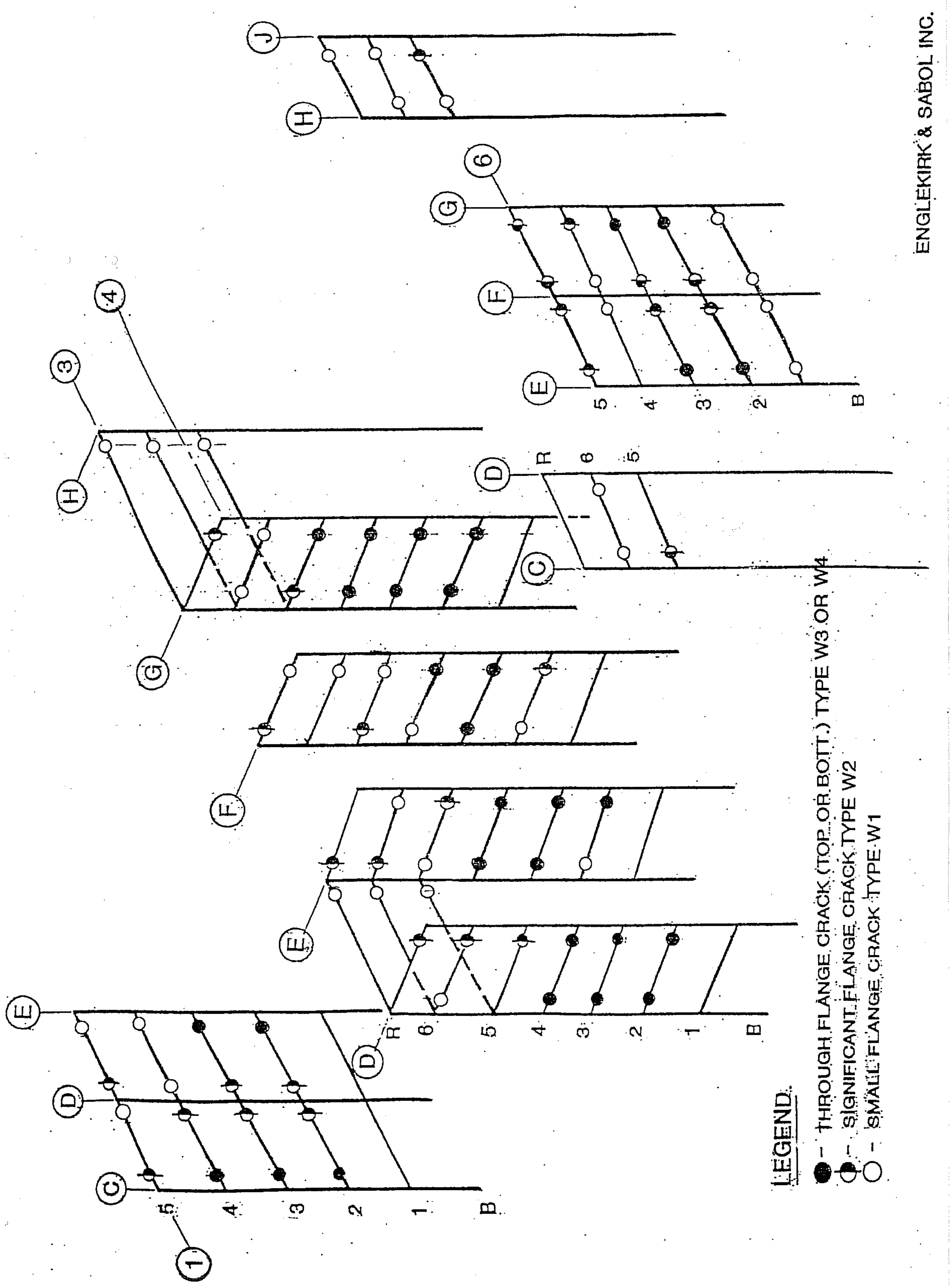


FIGURE 5 - SUMMARY OF JOINT DAMAGE

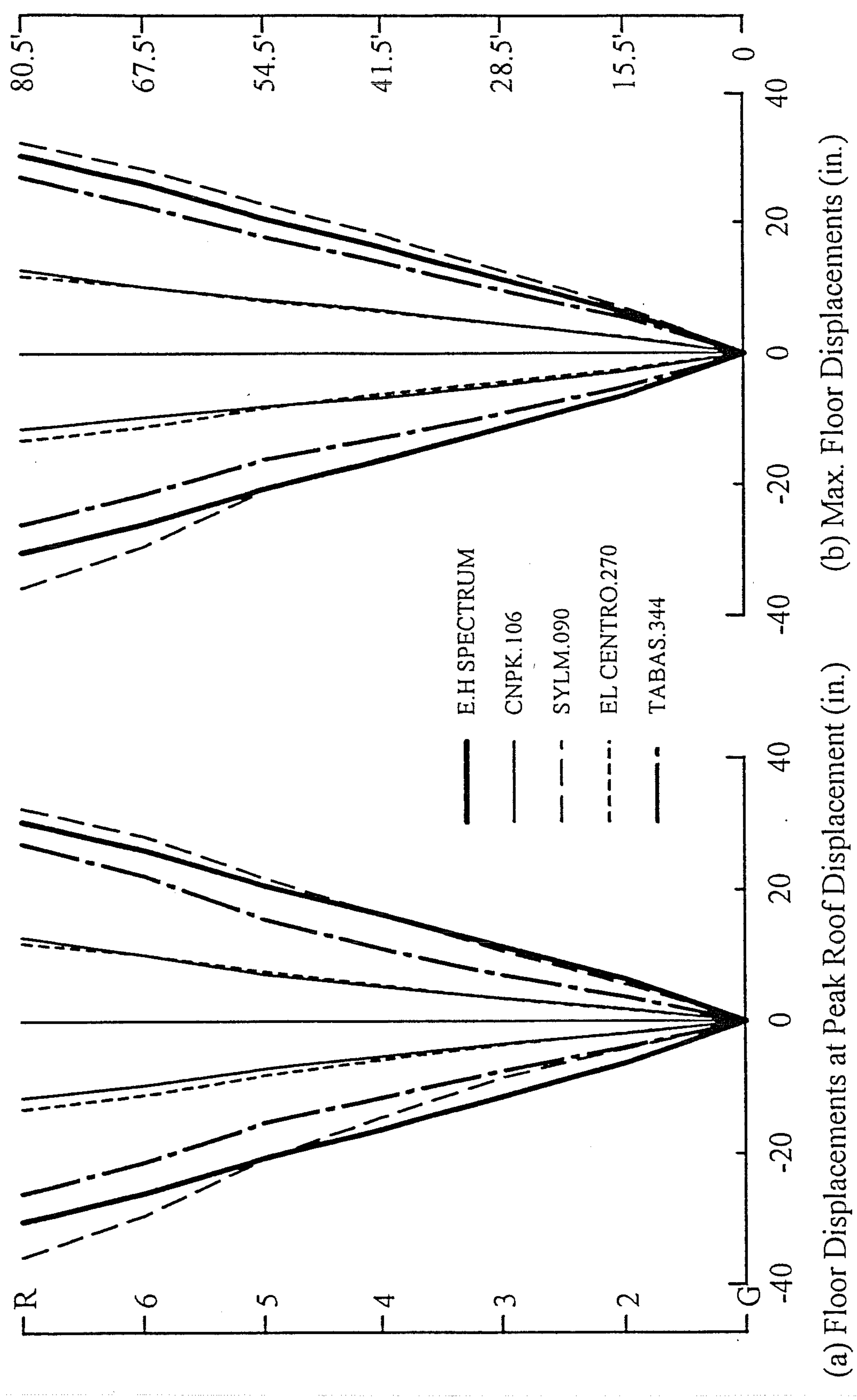


FIGURE 6 - 2-D ELASTIC ANALYSIS, BENCHMARK MODEL
MAXIMUM FLOOR DISPLACEMENTS

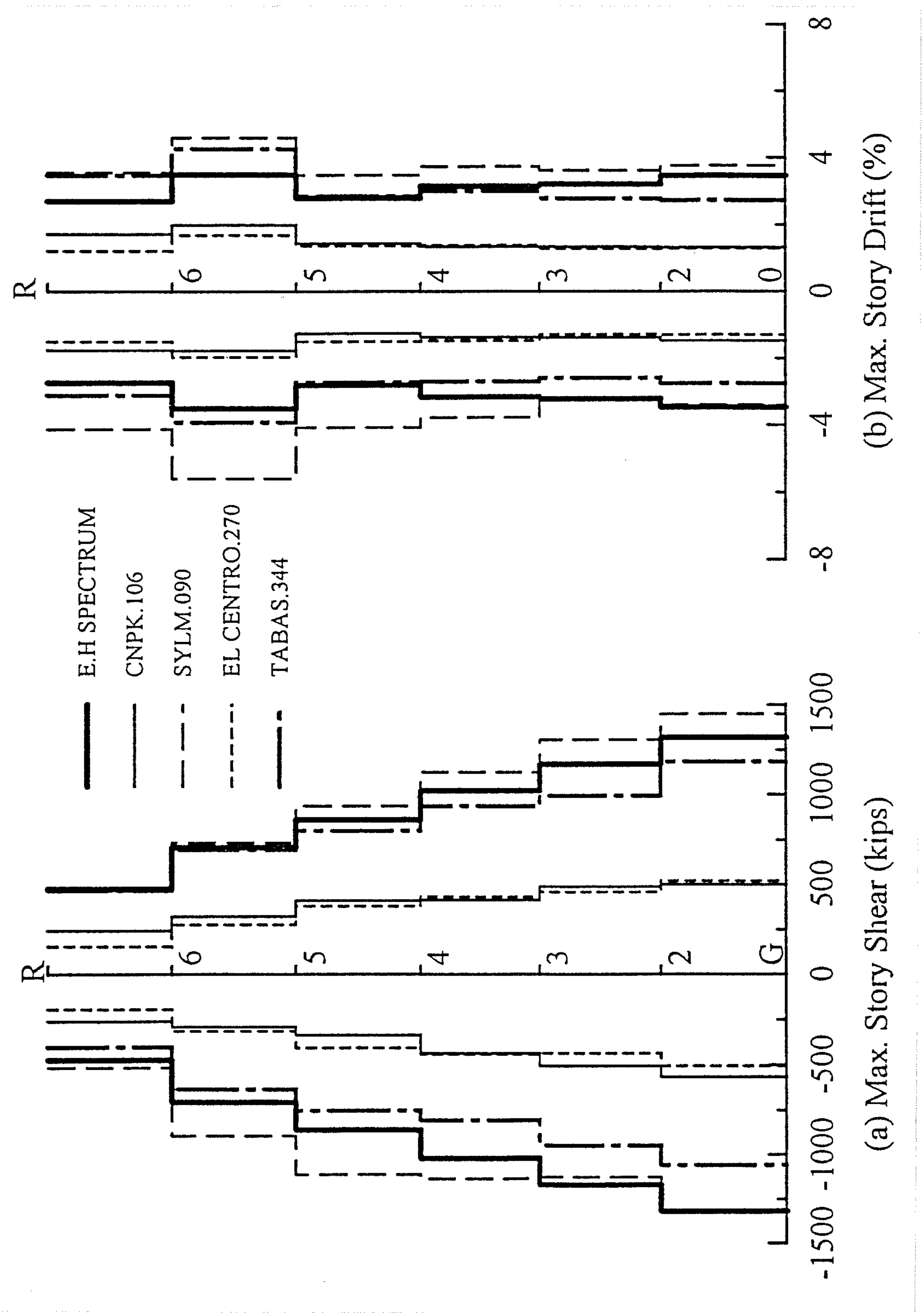


FIGURE 7 - 2-D ELASTIC ANALYSIS, BENCHMARK MODEL
MAXIMUM STORY SHEARS AND STORY DRIFTS

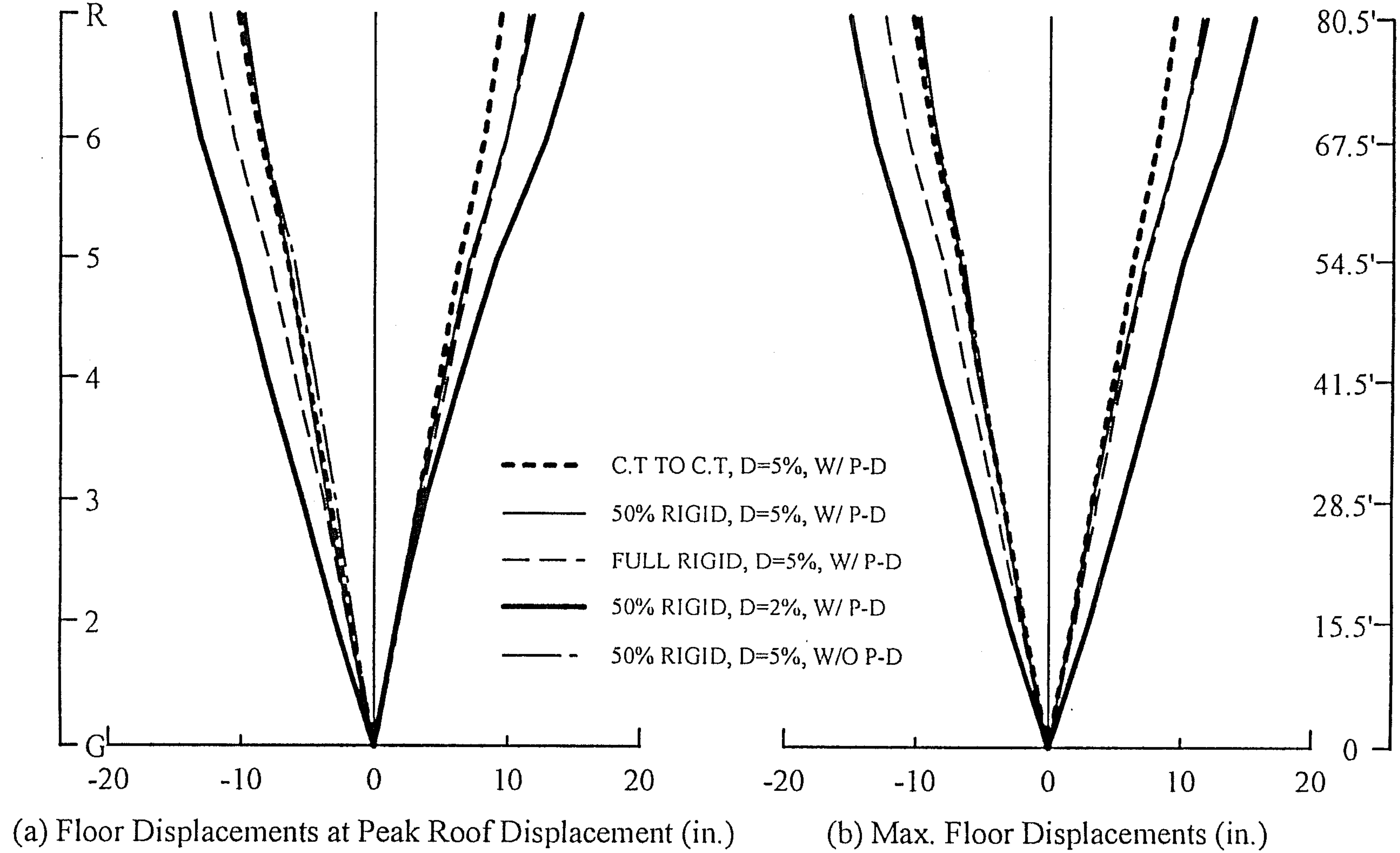


FIGURE 8 - 2-D ELASTIC ANALYSIS, SANTA MONICA CITY HALL RECORD (SMCH.045) MAXIMUM FLOOR DISPLACEMENTS

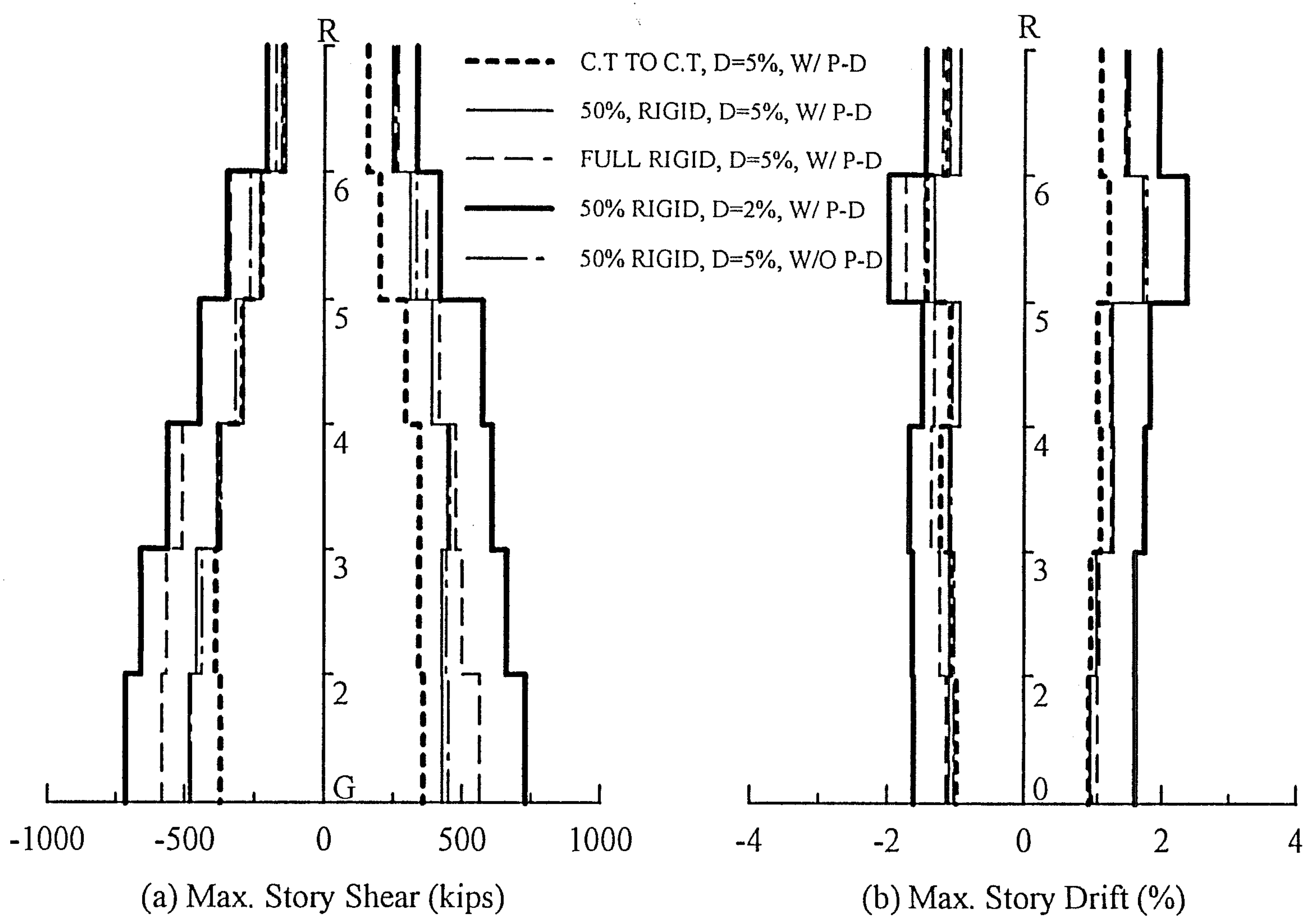


FIGURE 9 - 2-D ELASTIC ANALYSIS, SANTA MONICA CITY HALL RECORD (SMCH.045) MAXIMUM STORY SHEARS AND STORY DRIFTS

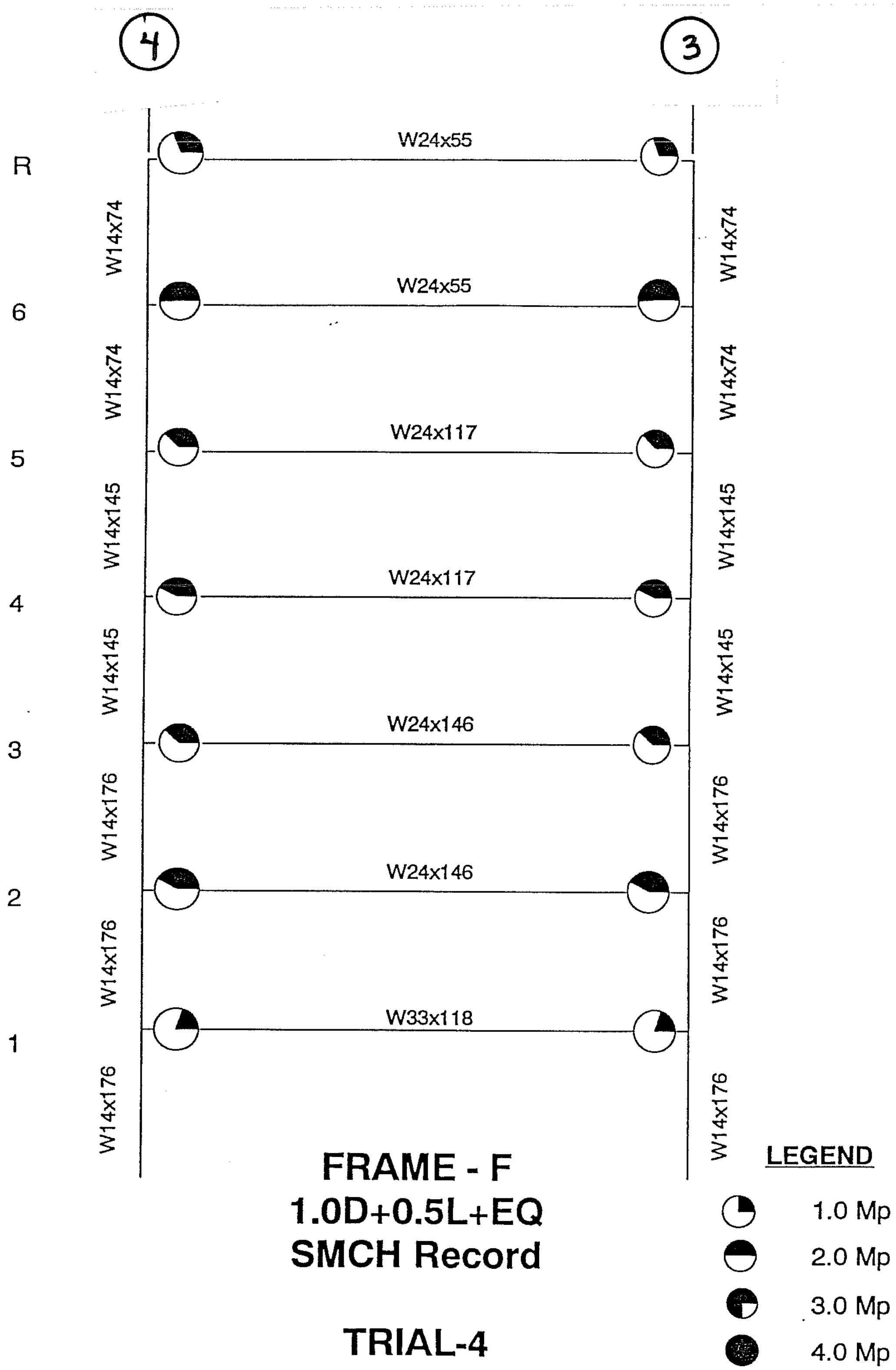


FIGURE 10 -DCRs FOR BEAM MOMENTS - FRAME ON LINE F
3-D ELASTIC ANALYSIS - TRIAL 4 MODEL
SANTA MONICA CITY HALL RECORD

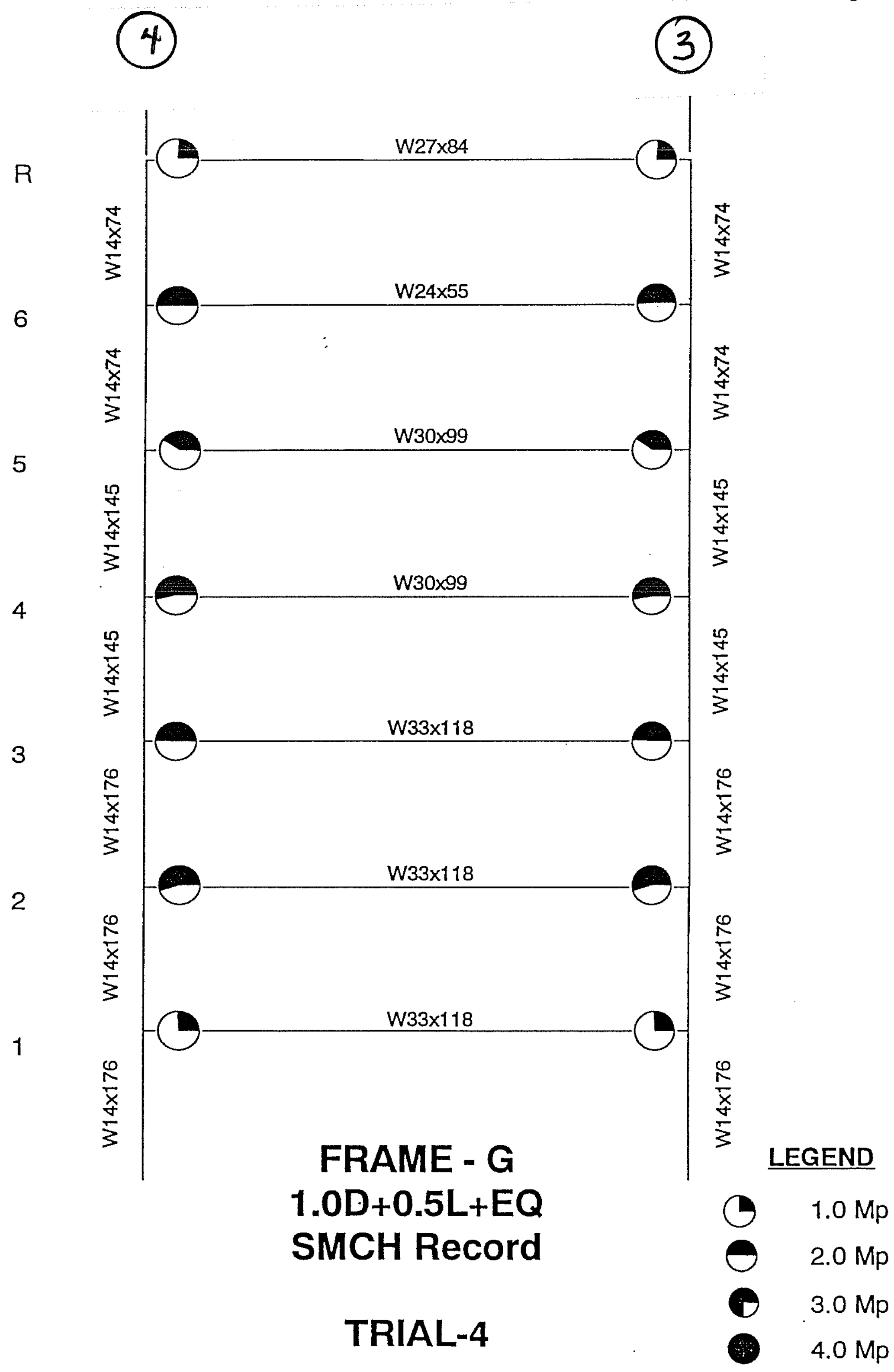
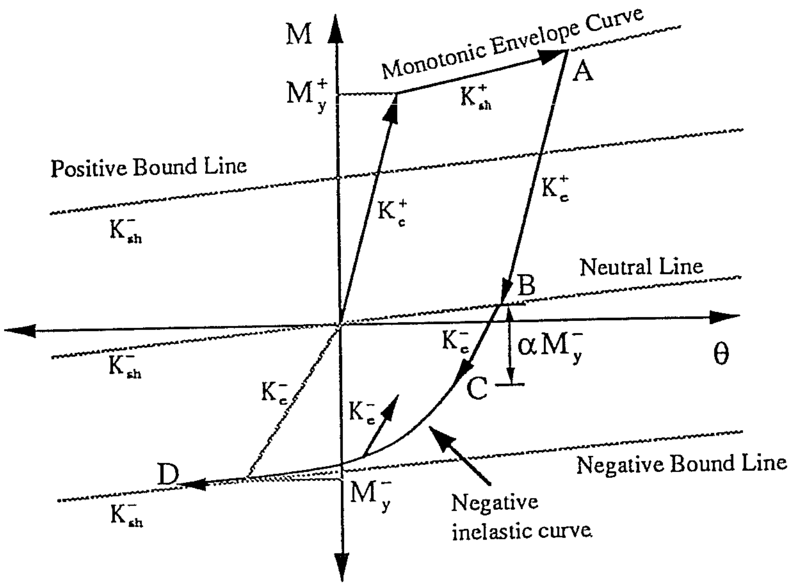
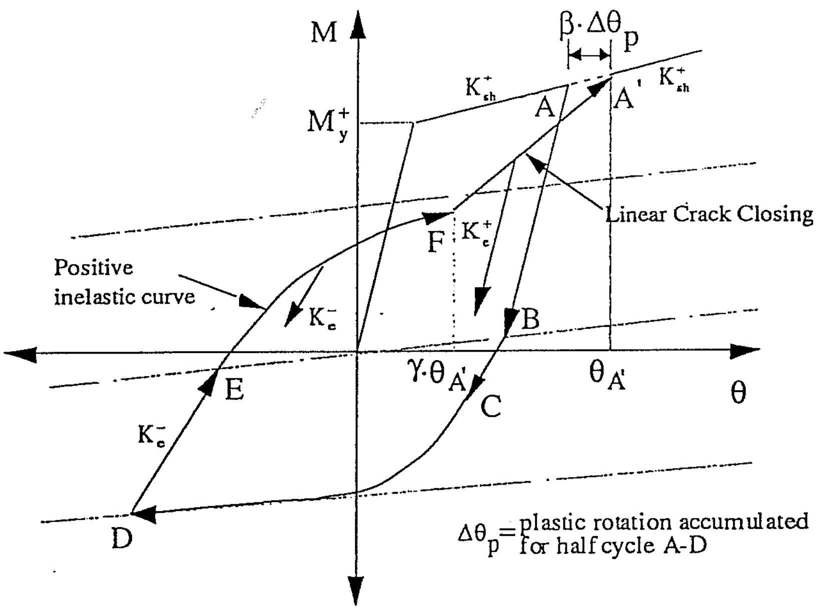


FIGURE 11 -DCRs FOR BEAM MOMENTS - FRAME ON LINE G
3-D ELASTIC ANALYSIS - TRIAL 4 MODEL
SANTA MONICA CITY HALL RECORD



a) Stiffness Degradation for Negative Moment



b) Stiffness Degradation and Pinching for Positive Moment

FIGURE 12 - OUTLINE OF HYSTERETIC MODEL FOR COMPOSITE BEAM ELEMENT

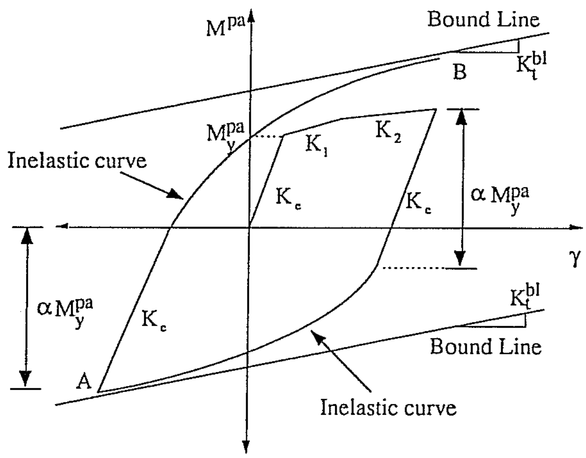


FIGURE 13 - OUTLINE OF HYSTERETIC MODEL FOR PANEL ZONE ELEMENT

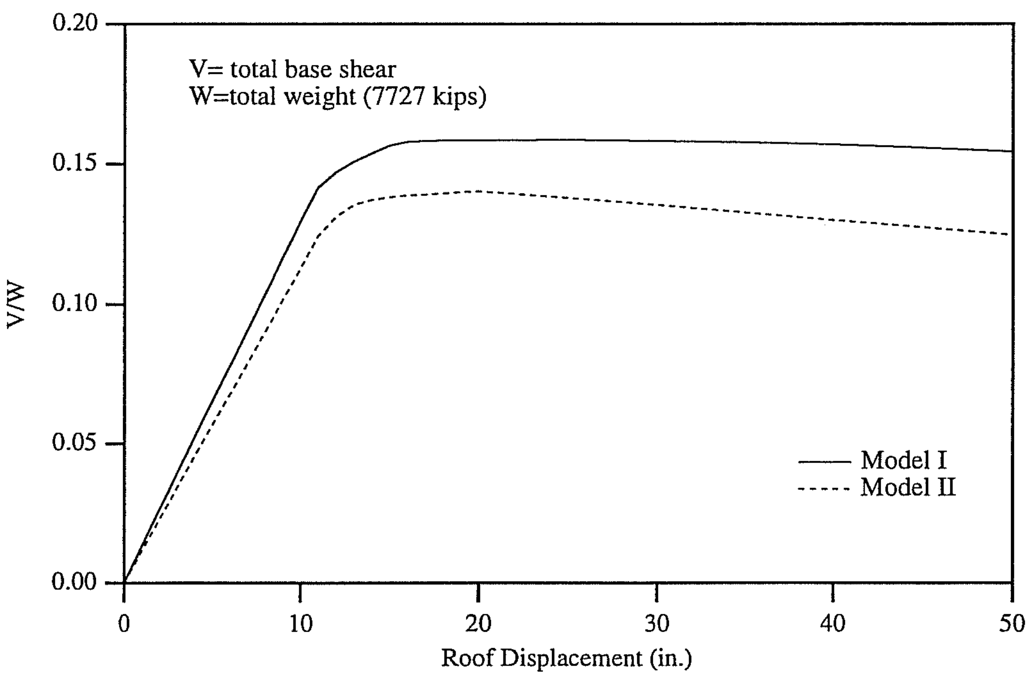
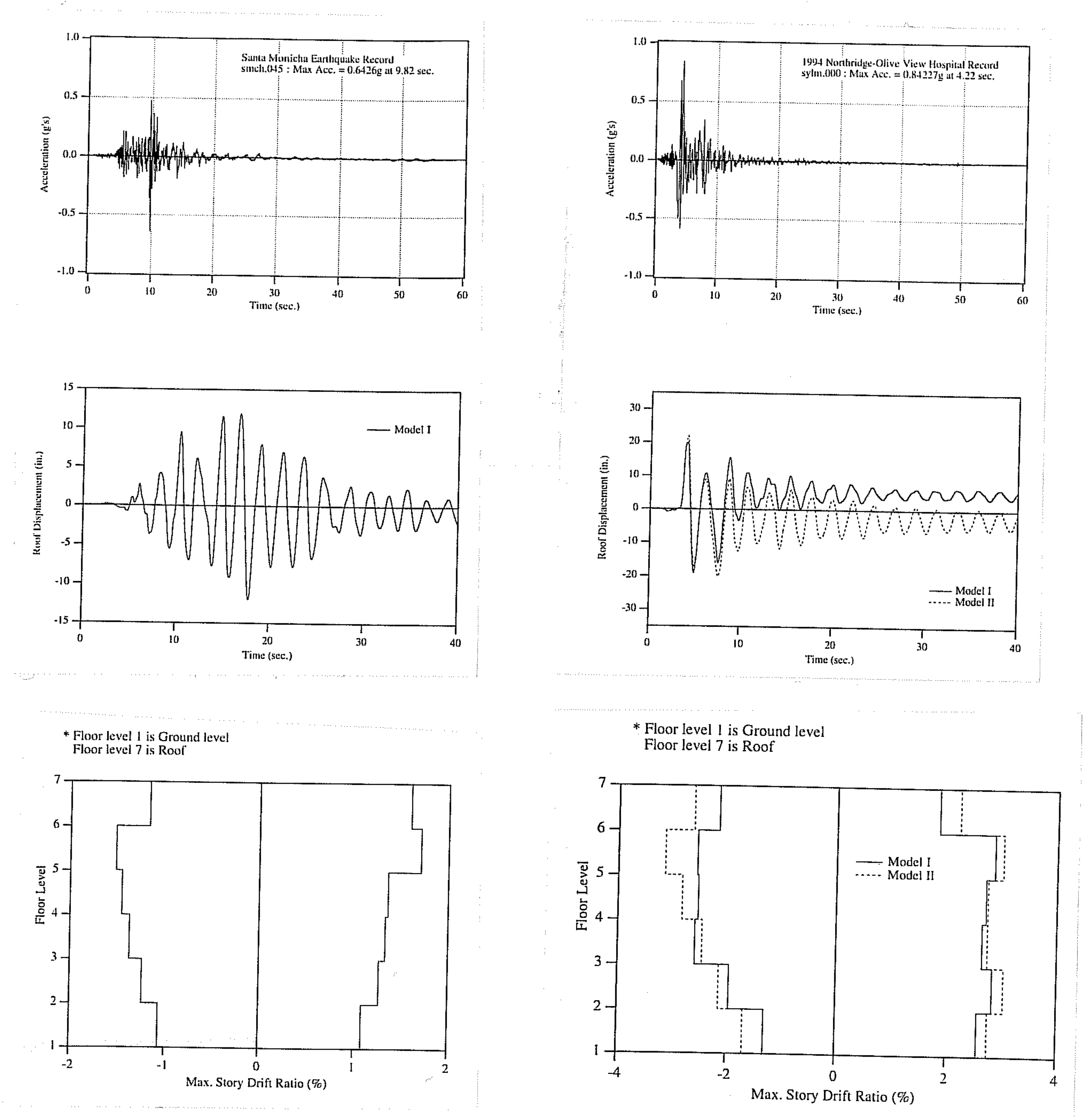


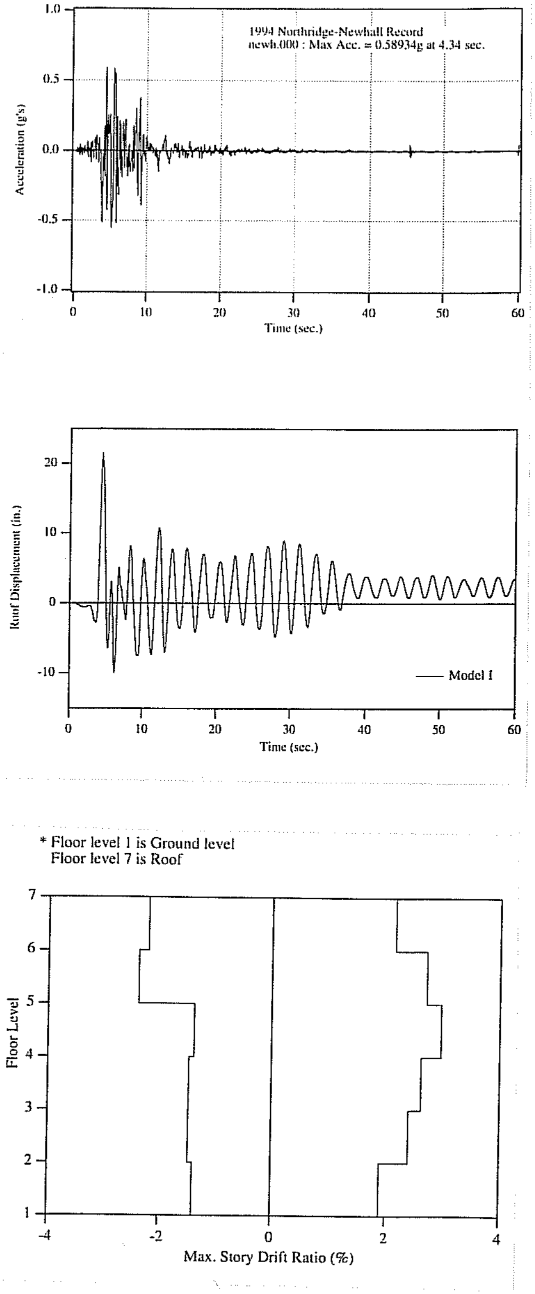
FIGURE 14 - RESULTS OF STATIC PUSHOVER



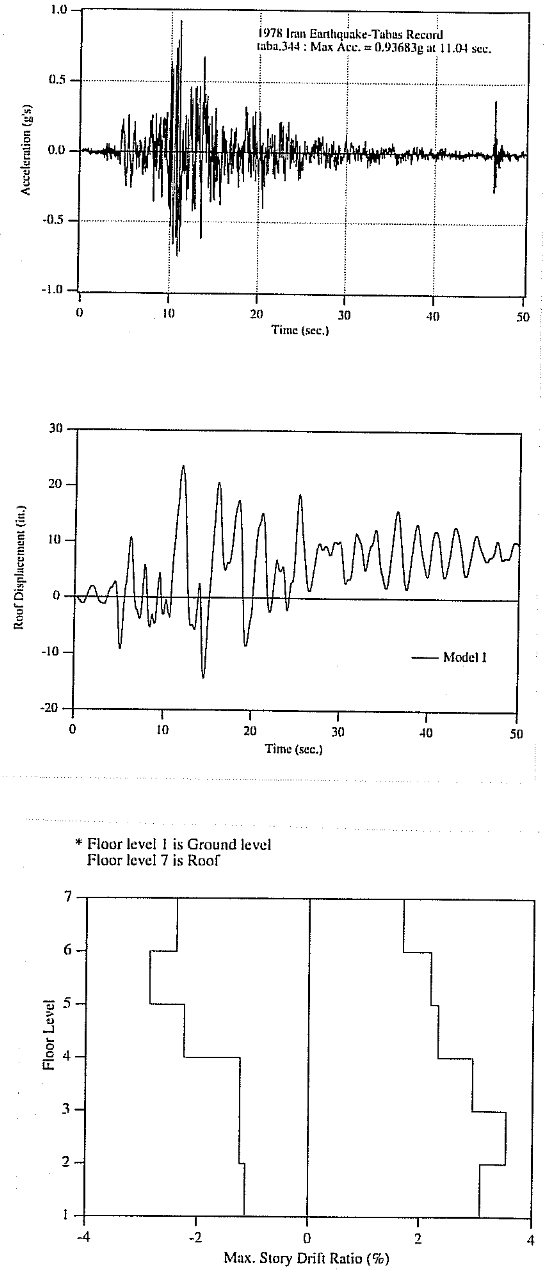
(a) smch.045 record

(b) sylm.000 record

FIGURE 15 - INELASTIC DYNAMIC ANALYSIS - ROOF DISPLACEMENTS AND MAXIMUM STORY DRIFT RATIOS

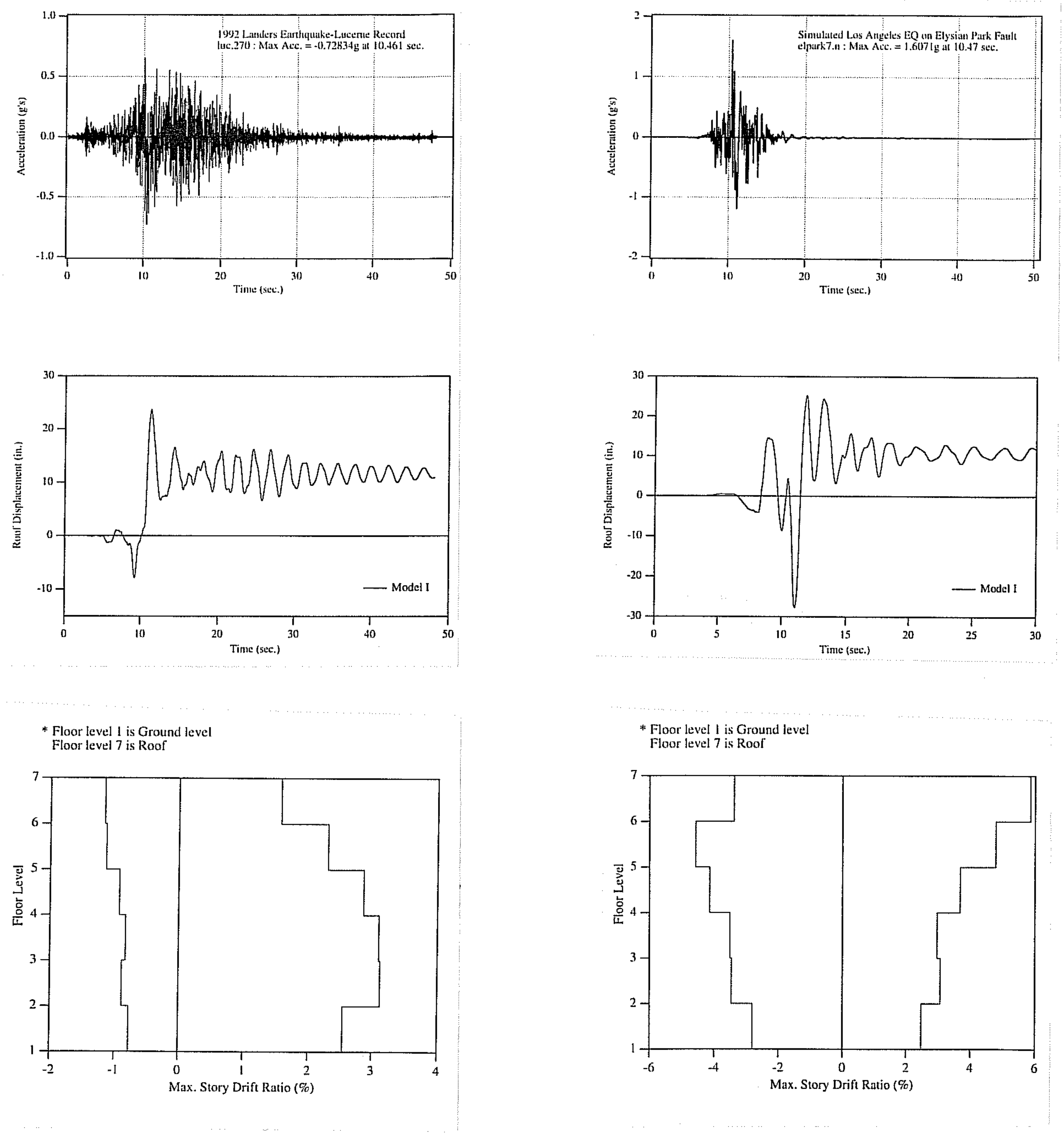


(c) newh.000 record



(d) taba.344 record

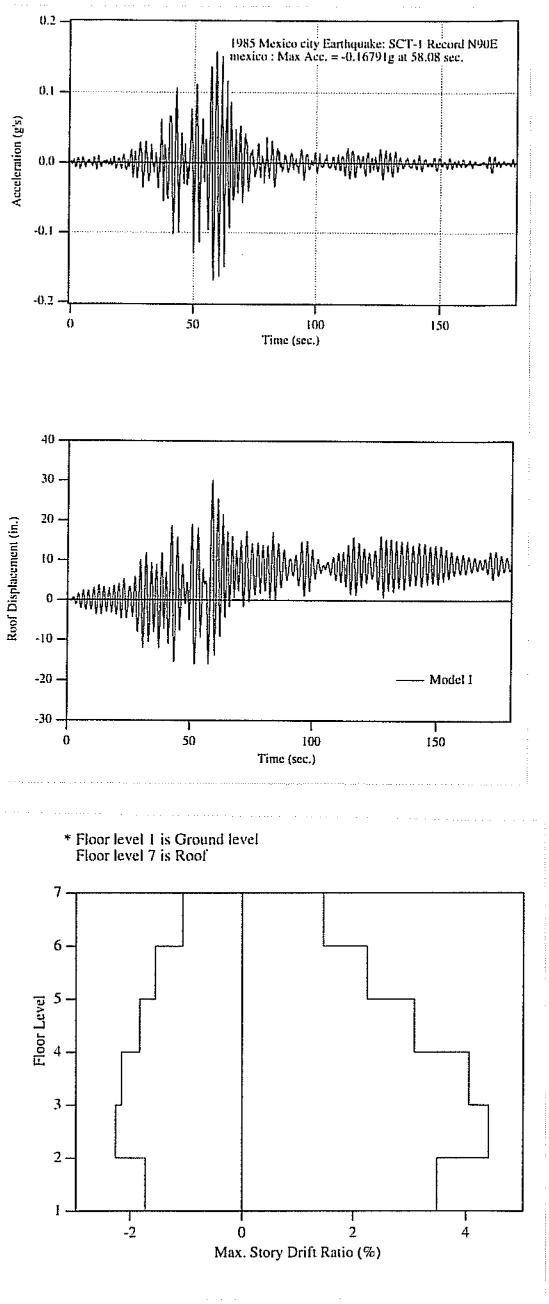
FIGURE 15 - INELASTIC DYNAMIC ANALYSIS - ROOF DISPLACEMENTS AND MAXIMUM STORY DRIFT RATIOS (CONT.)



(e) luc.270 record

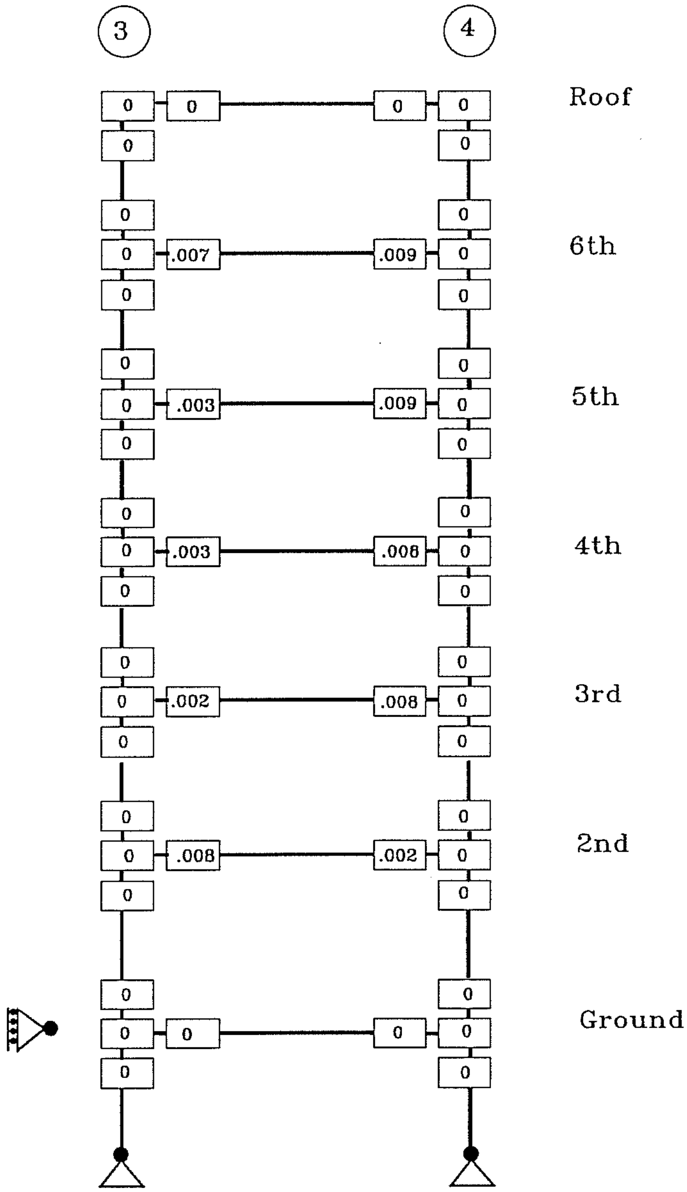
(f) elpark7.n record

FIGURE 15 - INELASTIC DYNAMIC ANALYSIS - ROOF DISPLACEMENTS AND MAXIMUM STORY DRIFT RATIOS (CONT.)



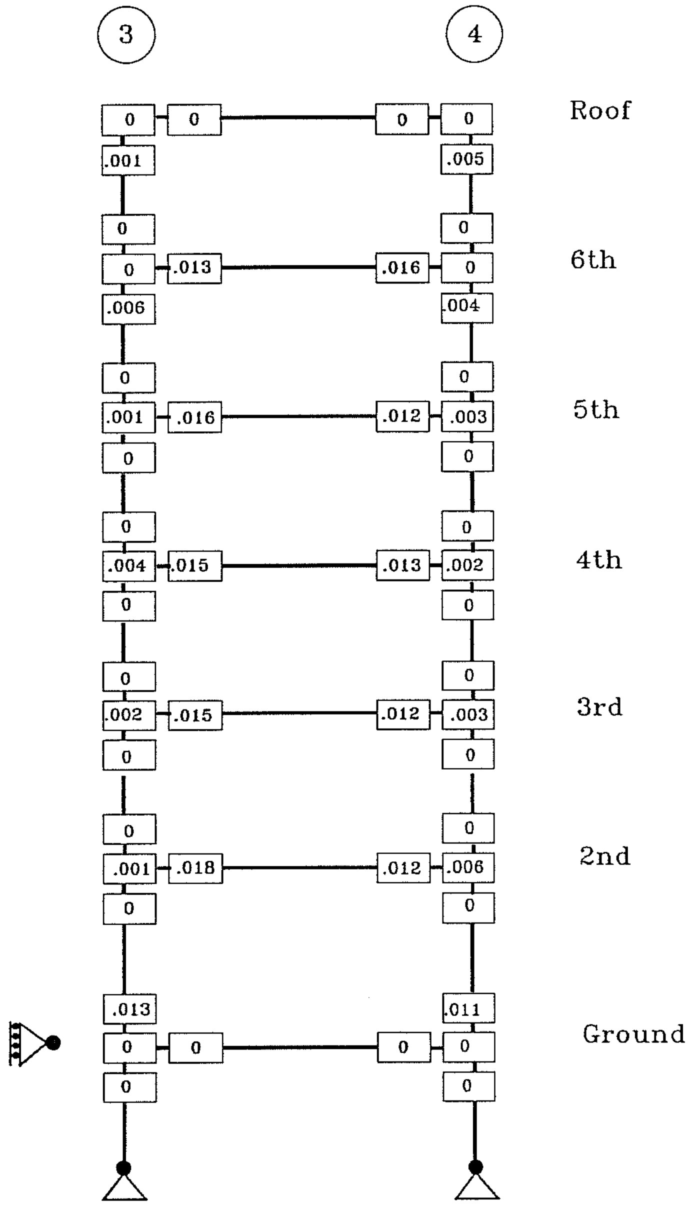
(g) mexico record

FIGURE 15 - INELASTIC DYNAMIC ANALYSIS - ROOF DISPLACEMENTS AND MAXIMUM STORY DRIFT RATIOS (CONT.)



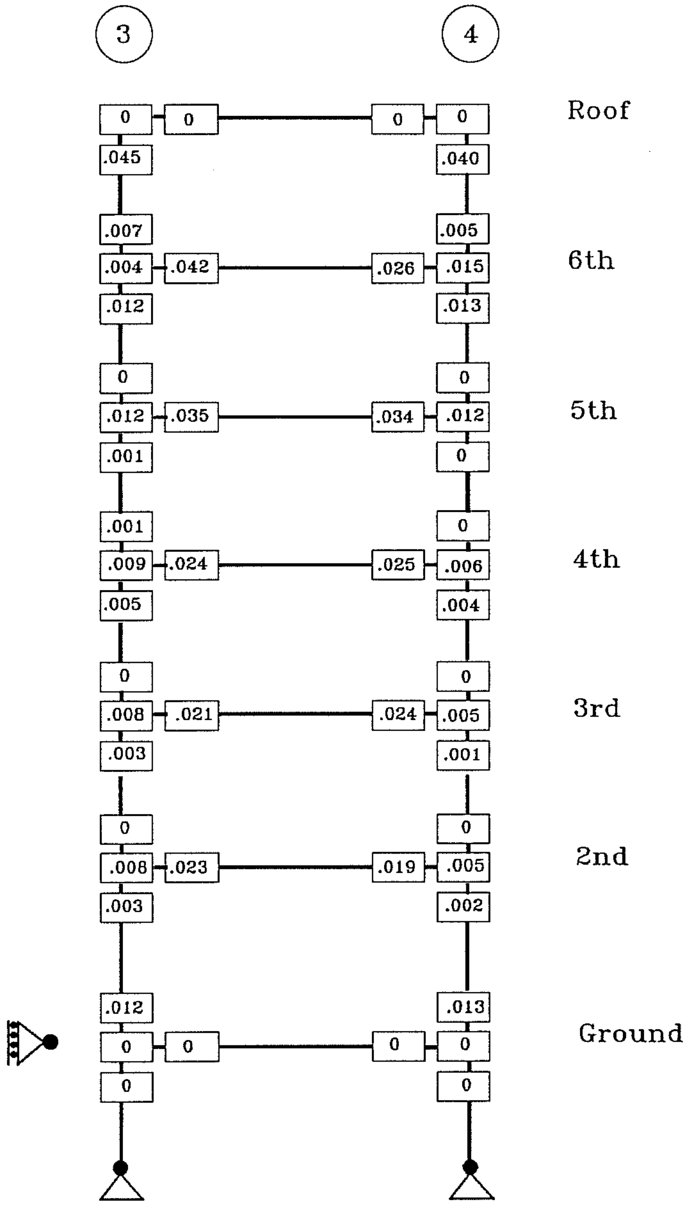
(a) smch.045 record

FIGURE 16 - INELASTIC DYNAMIC ANALYSIS - PLASTIC ROTATIONS FOR JOINTS IN FRAME ON LINE G



(b) sylm.000 record

FIGURE 16 - INELASTIC DYNAMIC ANALYSIS - PLASTIC ROTATIONS FOR JOINTS IN FRAME ON LINE G (CONT.)

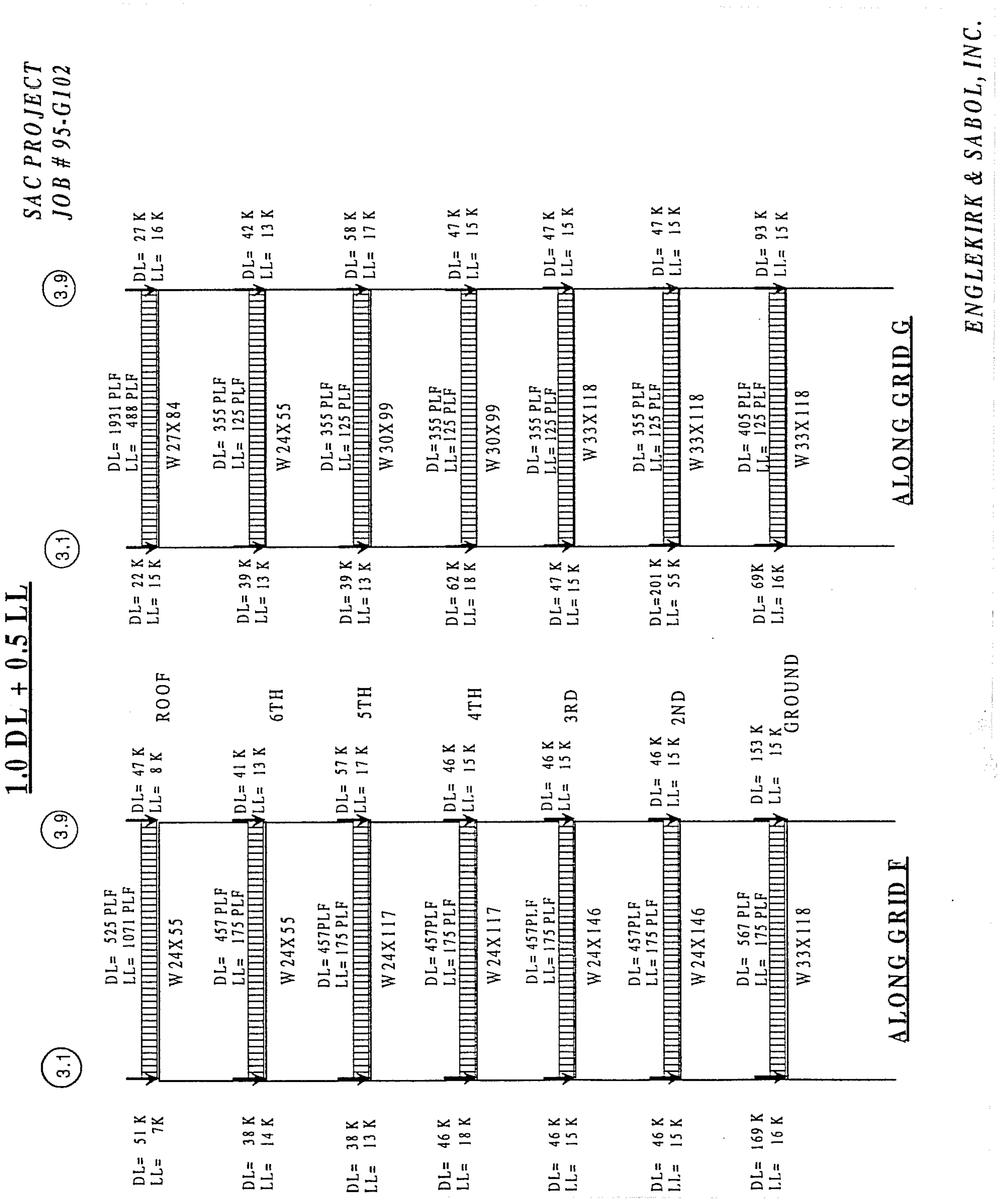


(c) elpark7.n record

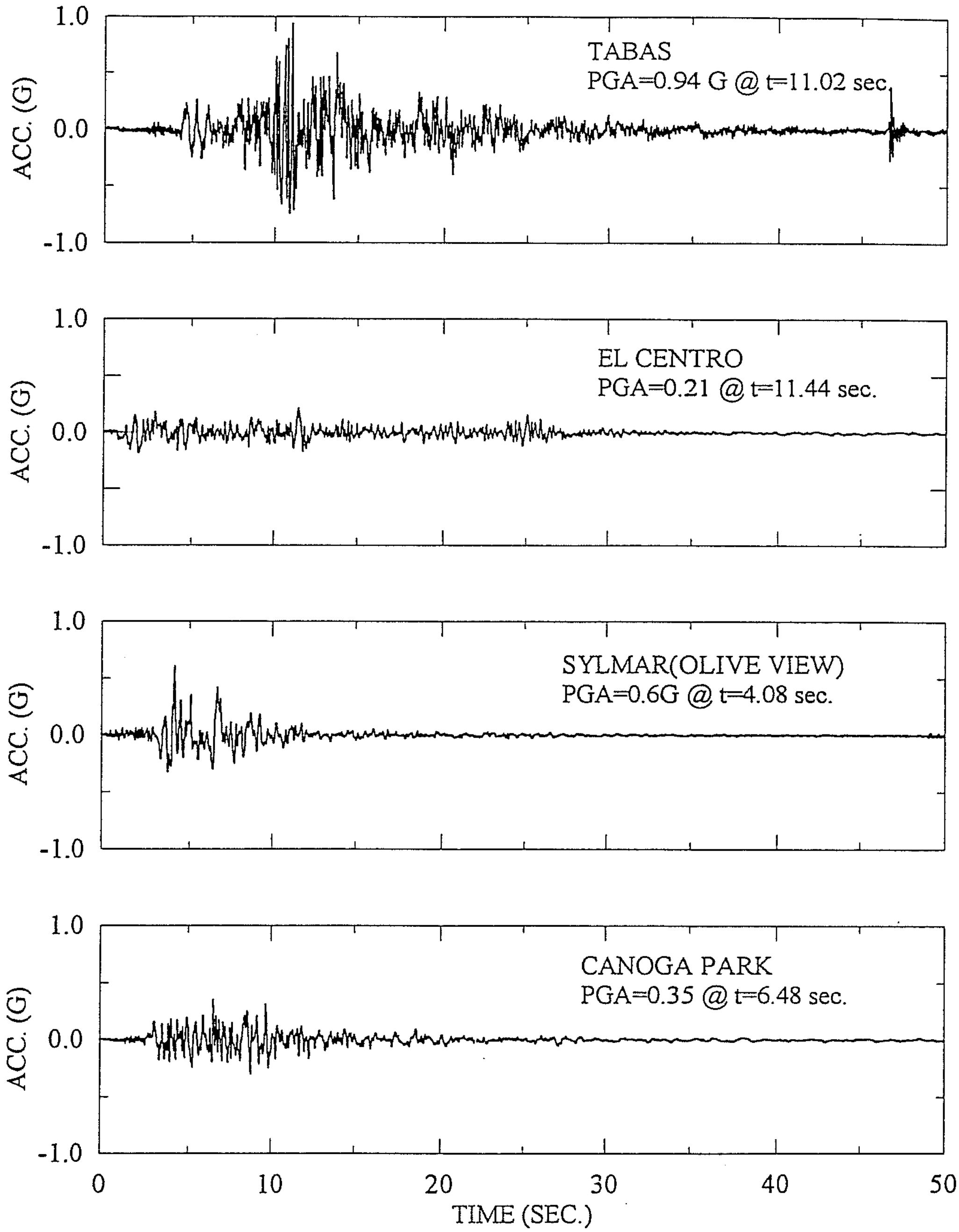
FIGURE 16 - INELASTIC DYNAMIC ANALYSIS - PLASTIC ROTATIONS FOR JOINTS IN FRAME ON LINE G (CONT.)

APPENDIX A - TWO-DIMENSIONAL ELASTIC ANALYSIS

This appendix contains additional data generated in the 2-D elastic analysis. The model assumptions and loading cases considered for the 2-D elastic analysis are described in Section 5.2.



Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)



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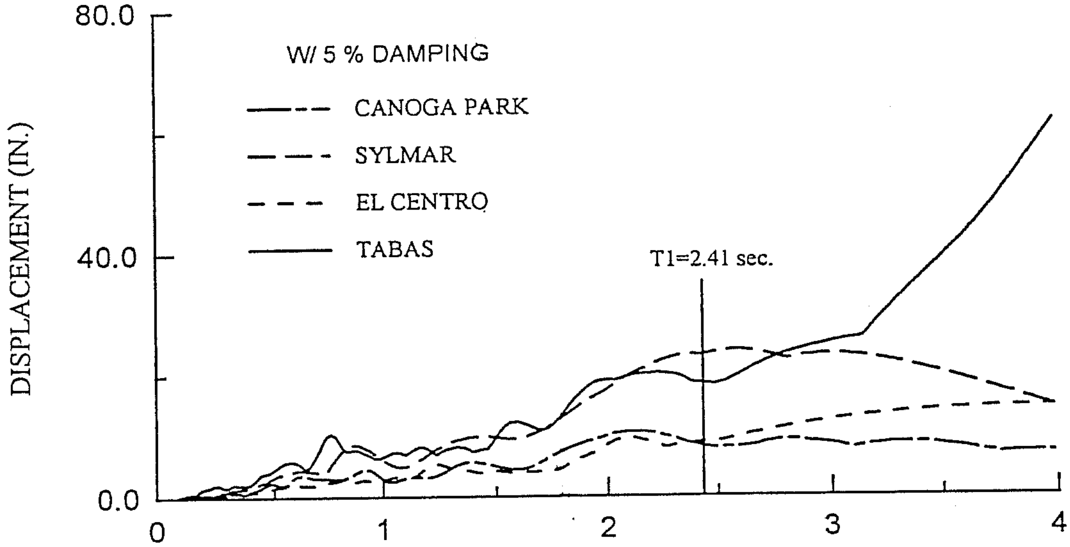
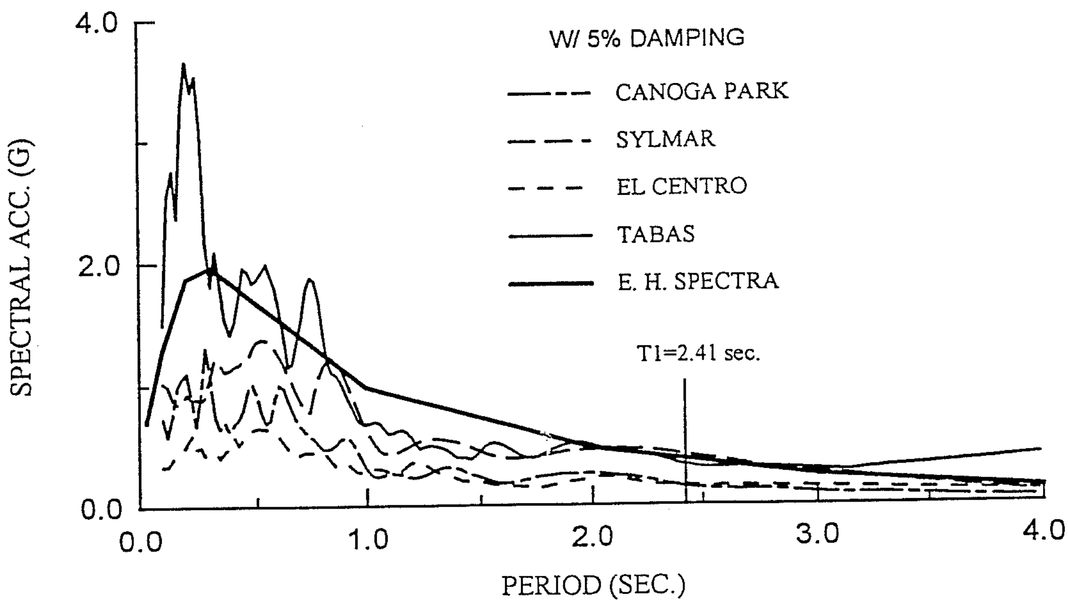
SAC PROJECT
JOB # 95-G102





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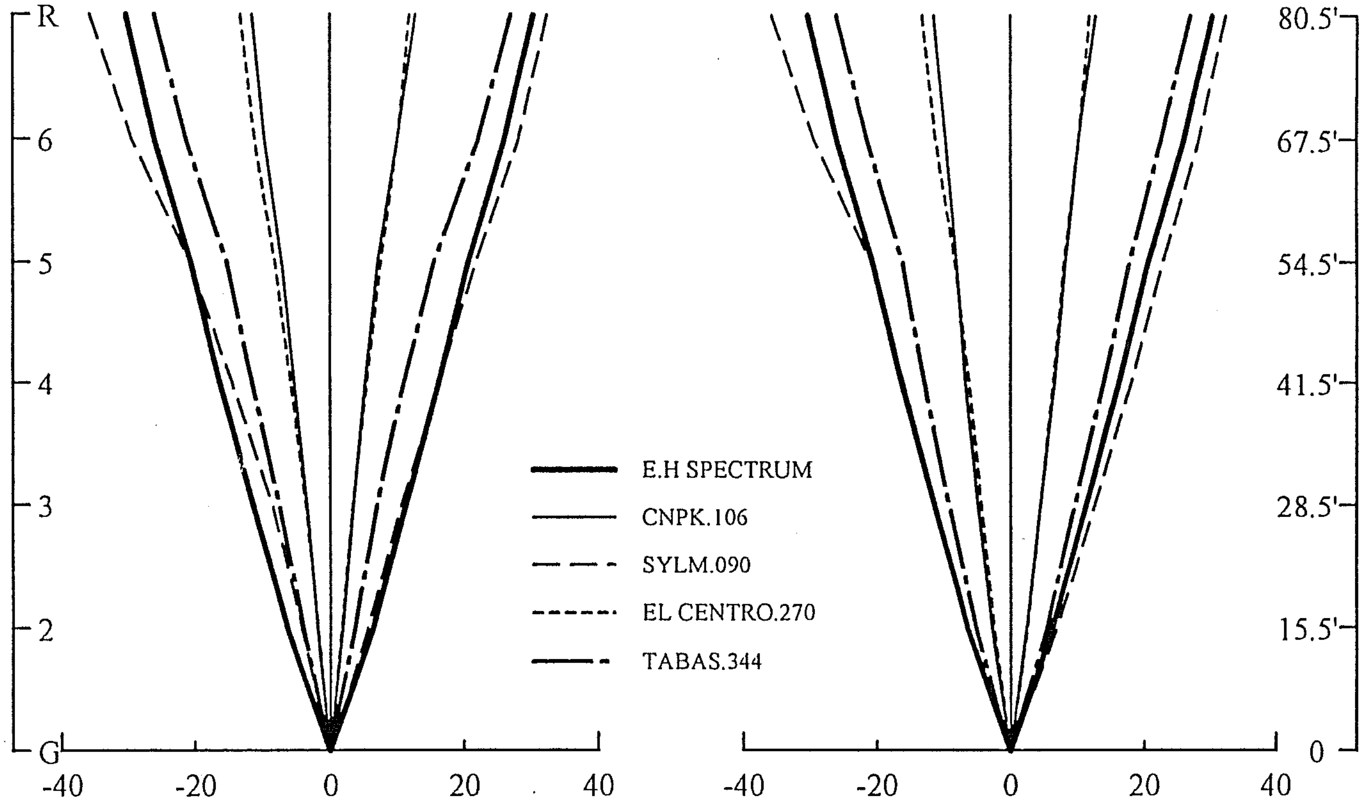
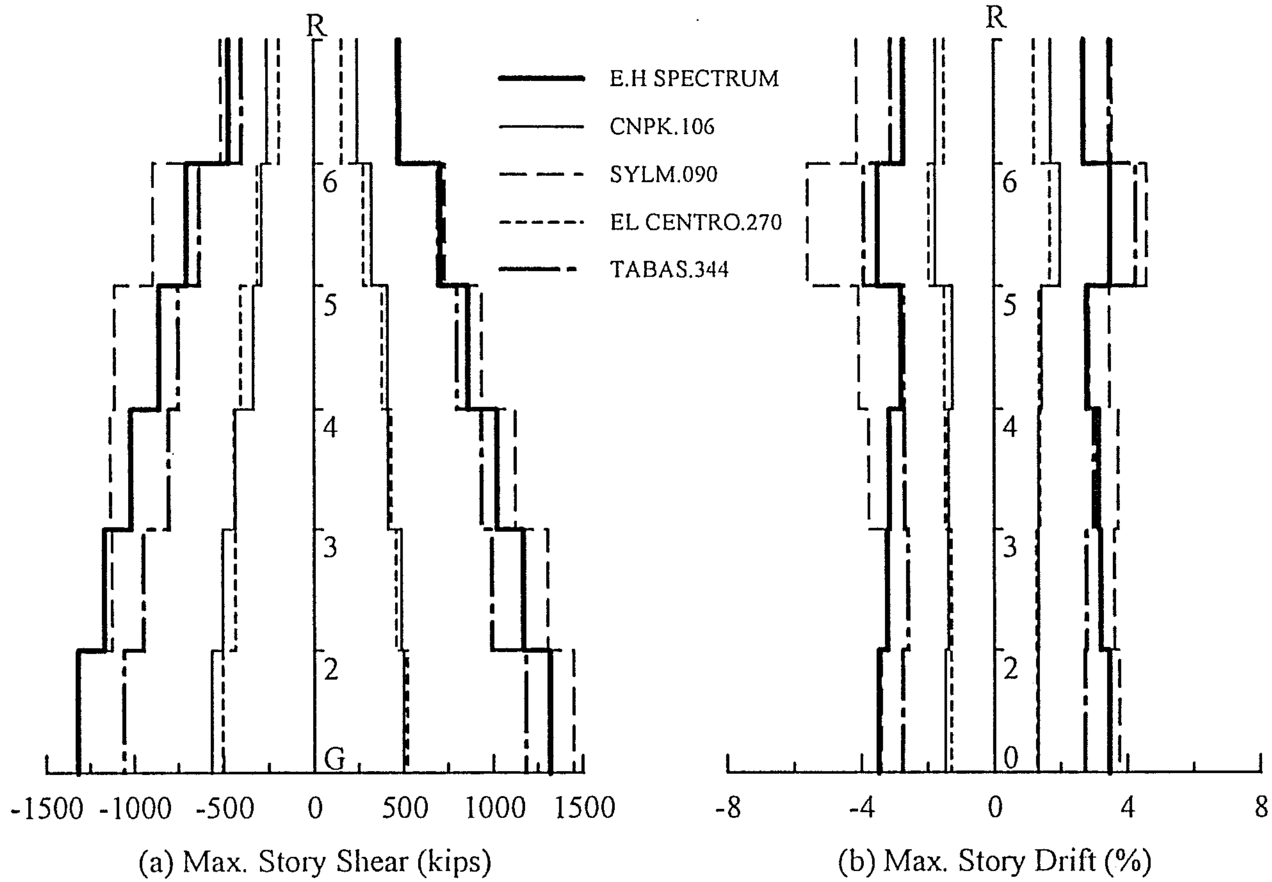
MAXIMUM DISPLACEMENTS - BENCH MARK ANALYSIS



(a) Floor Displacements at Peak Roof Displacement (in.)

(b) Max. Floor Displacements (in.)

MAXIMUM STORY SHEAR AND DRIFT - BENCH MARK ANALYSIS



ELASTIC ANALYSIS RESULTS

BEAM END MOMENTS (FT-KIPS)

MEMBER #	CAP	GRAV	E.H.S	CNPK	SYLM	IVIR	TABAS
F.G.3	1636	38	2043	878	2243	806	1812
F.G.4	1636	33	2043	878	2243	806	1812
F.2.3	1648	41	3923	1698	4357	1555	3305
F.2.4	1648	29	3923	1698	4357	1555	3305
F.3.3	1648	41	3443	1431	3861	1365	2973
F.3.4	1648	27	3443	1431	3861	1365	2973
F.4.3	1289	41	2831	1133	3452	1303	2618
F.4.4	1289	29	2831	1133	3452	1303	2618
F.5.3	1289	36	2589	1223	3411	1215	2469
F.5.4	1289	27	2589	1223	3411	1215	2469
F.6.3	564	43	1550	733	1918	674	1541
F.6.4	564	38	1550	733	1918	674	1541
F.R.3	564	74	852	452	974	359	849
F.R.4	564	71	852	452	974	359	849
G.G.3	1636	22	2088	898	2293	824	1849
G.G.4	1636	29	2089	898	2293	824	1849
G.2.3	1636	18	4459	1929	4953	1768	3758
G.2.4	1636	32	4459	1929	4953	1768	3758
G.3.3	1636	18	3933	1634	4410	1556	3395
G.3.4	1636	31	3933	1634	4410	1556	3395
G.4.3	1230	21	3054	1254	3723	1406	2824
G.4.4	1230	32	3054	1254	3723	1406	2824
G.5.3	1230	19	2730	1291	3598	1283	2603
G.5.4	1230	27	2730	1291	3598	1283	2603
G.6.3	564	31	1591	754	1965	691	1583
G.6.4	564	35	1591	754	1965	691	1583
G.R.3	962	80	1082	575	1233	455	1078
G.R.4	962	83	1082	575	1233	455	1078

DEMAND-CAPCITY RATIO BEAM END MOEMNTS							
MEMBER #	CAP('K)	GRAV	E.H.S	CNPK	SYLM	IVIR	TABAS
F.G.3	1636	0.02	1.25	0.54	1.37	0.49	1.11
F.G.4	1636	0.02	1.25	0.54	1.37	0.49	1.11
F.2.3	1648	0.02	2.38	1.03	2.64	0.94	2.01
F.2.4	1648	0.02	2.38	1.03	2.64	0.94	2.01
F.3.3	1648	0.02	2.09	0.87	2.34	0.83	1.80
F.3.4	1648	0.02	2.09	0.87	2.34	0.83	1.80
F.4.3	1289	0.03	2.20	0.88	2.68	1.01	2.03
F.4.4	1289	0.02	2.20	0.88	2.68	1.01	2.03
F.5.3	1289	0.03	2.01	0.95	2.65	0.94	1.92
F.5.4	1289	0.02	2.01	0.95	2.65	0.94	1.92
F.6.3	564	0.08	2.75	1.30	3.40	1.19	2.73
F.6.4	564	0.07	2.75	1.30	3.40	1.19	2.73
F.R.3	564	0.13	1.51	0.80	1.73	0.64	1.51
F.R.4	564	0.13	1.51	0.80	1.73	0.64	1.51
G.G.3	1636	0.01	1.28	0.55	1.40	0.50	1.13
G.G.4	1636	0.02	1.28	0.55	1.40	0.50	1.13
G.2.3	1636	0.01	2.73	1.18	3.03	1.08	2.30
G.2.4	1636	0.02	2.73	1.18	3.03	1.08	2.30
G.3.3	1636	0.01	2.40	1.00	2.70	0.95	2.08
G.3.4	1636	0.02	2.40	1.00	2.70	0.95	2.08
G.4.3	1230	0.02	2.48	1.02	3.03	1.14	2.30
G.4.4	1230	0.03	2.48	1.02	3.03	1.14	2.30
G.5.3	1230	0.02	2.22	1.05	2.92	1.04	2.12
G.5.4	1230	0.02	2.22	1.05	2.92	1.04	2.12
G.6.3	564	0.05	2.82	1.34	3.48	1.23	2.81
G.6.4	564	0.06	2.82	1.34	3.48	1.23	2.81
G.R.3	962	0.08	1.12	0.60	1.28	0.47	1.12
G.R.4	962	0.09	1.12	0.60	1.28	0.47	1.12

MAX. STORY SHEAR W/ BENCH MARK EARTHQUAKES

STORY	E.H SPECTRUM		CNPK.106		SYLM.090		EL CENTRO 270		TABAS.344	
LEVEL	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)
ROOF	-474	474	-262	241	-515	464	-194	153	-401	468
6TH	-707	707	-292	322	-894	734	-314	275	-632	693
5TH	-861	861	-338	408	-1111	936	-408	378	-750	799
4TH	-1020	1020	-437	413	-1138	1125	-441	429	-807	935
3RD	-1170	1170	-508	488	-1129	1307	-438	456	-948	994
2ND	-1318	1318	-566	499	-1317	1447	-507	521	-1057	1185

MAX. STORY DRIFT(%) W/ BENCH MARK EARTHQUAKES

STORY	E. H. SPECTRUM		CNPK.106		SYLM.090		EL CENTRO 270		TABAS.344	
LEVEL	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)
ROOF	-2.71	2.71	-1.74	1.71	-4.11	3.56	-1.49	1.21	-3.08	3.47
6TH	-3.49	3.49	-1.76	1.99	-5.57	4.60	-1.95	1.69	-3.89	4.27
5TH	-2.79	2.79	-1.24	1.44	-4.06	3.47	-1.49	1.36	-2.70	2.87
4TH	-3.15	3.15	-1.36	1.34	-3.77	3.73	-1.46	1.39	-2.69	3.01
3RD	-3.21	3.21	-1.38	1.35	-3.16	3.63	-1.28	1.28	-2.59	2.79
2ND	-3.48	3.48	-1.46	1.32	-3.39	3.76	-1.29	1.34	-2.74	2.96

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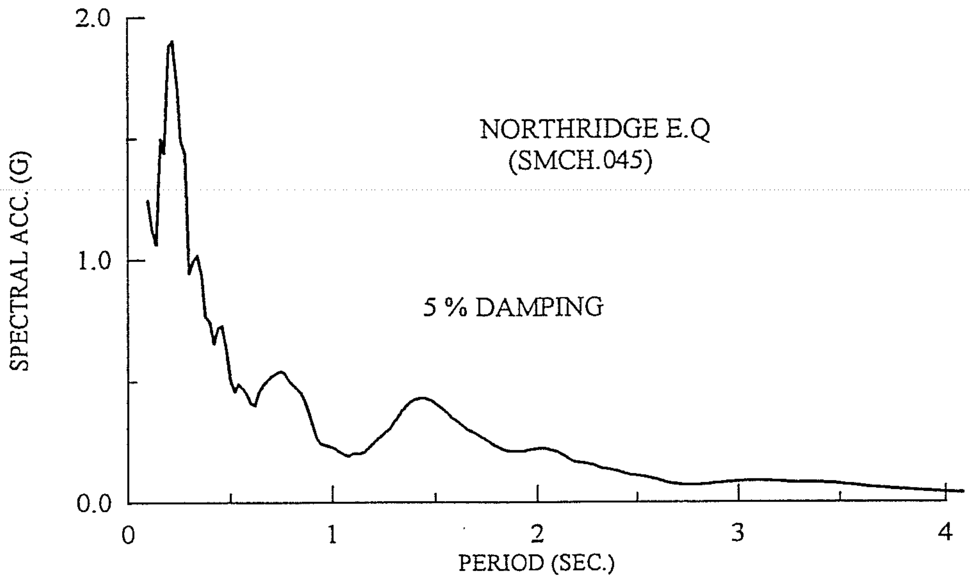
MAX. FLOOR DISPL. @MAX. ROOF DISPL. W/ BENCH MARK EARTHQUAKES

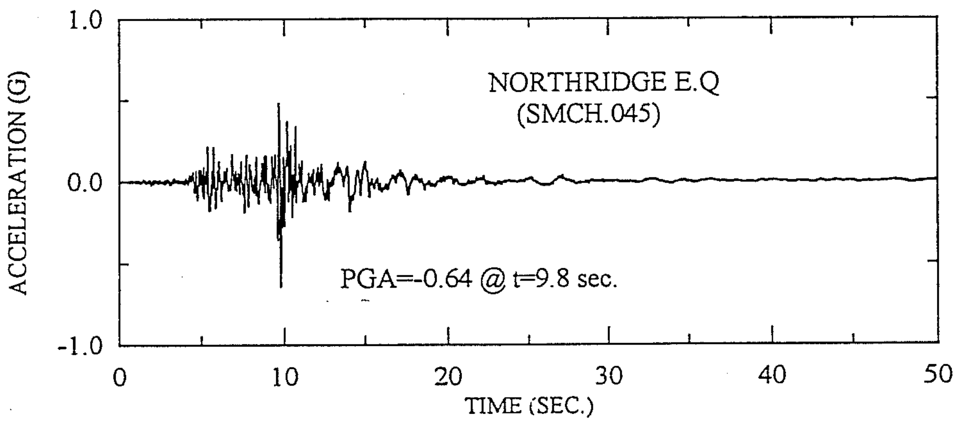
FLOOR	E. H. SPECTRUM		CNPK 106		SYLM 090		EL CENTRO 270		TABAS 344	
LEVEL	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)
ROOF	-30.28	30.28	-11.50	12.80	-35.70	32.30	-13.20	11.84	-26.00	27.00
6TH	-26.06	26.06	-9.65	10.13	-29.40	28.10	-11.10	10.10	-21.30	22.10
5TH	-20.61	20.61	-7.11	7.11	-20.70	21.70	-8.10	7.54	-15.20	15.48
4TH	-16.25	16.25	-5.22	5.27	-14.40	16.26	-5.80	5.45	-11.40	11.05
3RD	-11.33	11.33	-3.44	3.46	-8.50	10.60	-3.56	3.36	-7.46	7.00
2ND	-6.33	6.33	-1.90	1.83	-4.18	5.54	-1.76	1.68	-3.86	3.68

FLOOR DISPL. ENVELOP W/ BENCH MARK EARTHQUAKES

FLOOR	E. H. SPECTRUM		CNPK 106		SYLM 090		EL CENTRO 270		TABAS 344	
LEVEL	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)
ROOF	-30.28	30.28	-11.50	12.80	-35.70	32.30	-13.20	11.80	-26.00	27.00
6TH	-26.06	26.06	-9.66	10.18	-29.40	28.30	-11.20	10.20	-21.40	22.60
5TH	-20.61	20.61	-8.03	8.27	-20.80	22.80	-8.30	8.00	-16.10	17.80
4TH	-16.25	16.25	-6.70	6.60	-15.90	18.10	-6.10	6.40	-12.80	14.00
3RD	-11.33	11.33	-4.87	4.55	-11.25	12.60	-4.30	4.50	-9.10	9.60
2ND	-6.33	6.33	-2.72	2.46	-6.30	7.00	-2.40	2.50	-5.10	5.50



(a) RESPONSE SPECTRUM



(b) TIME HISTORY RECORD OF GROUND MOTION

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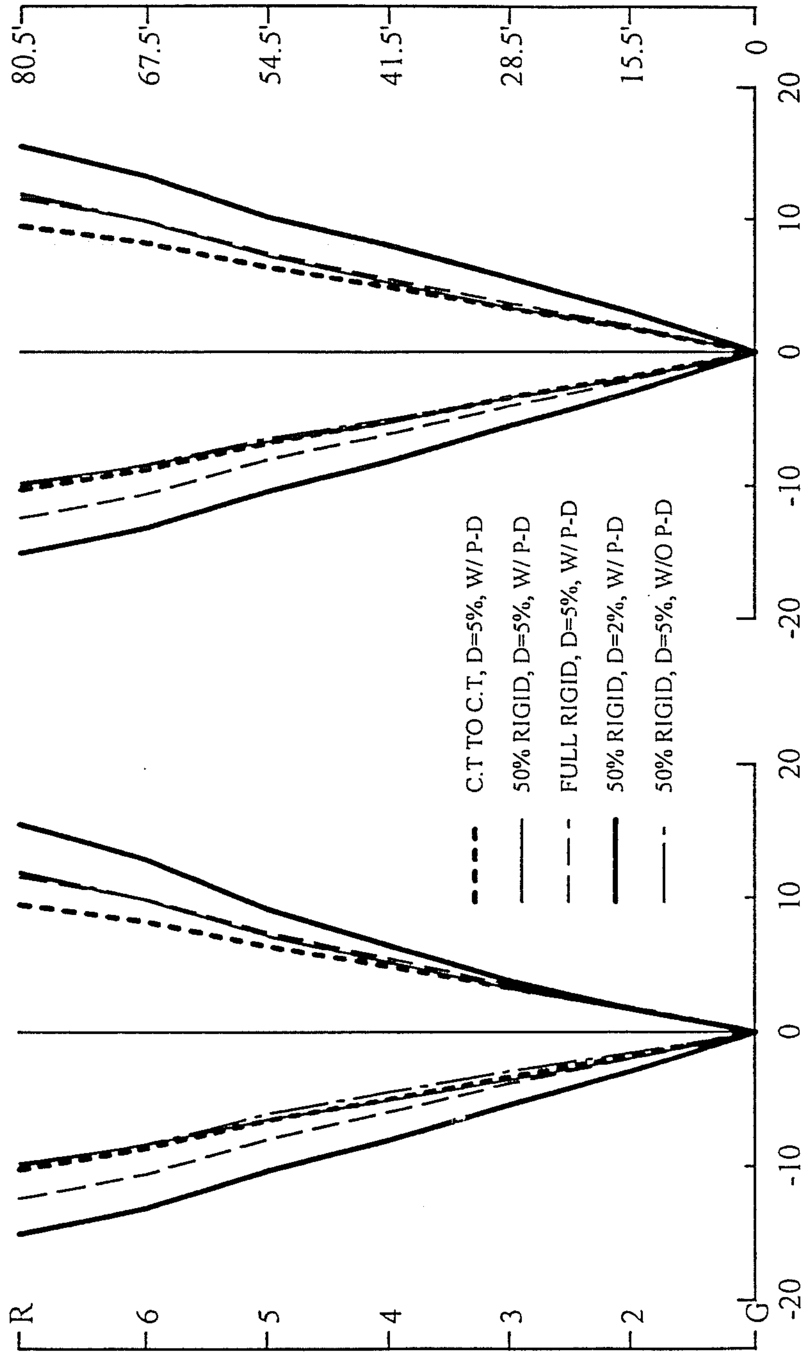
MAXIMUM DISPLACEMENTS - W/ NORTHRIDGE EARTHQUAKE

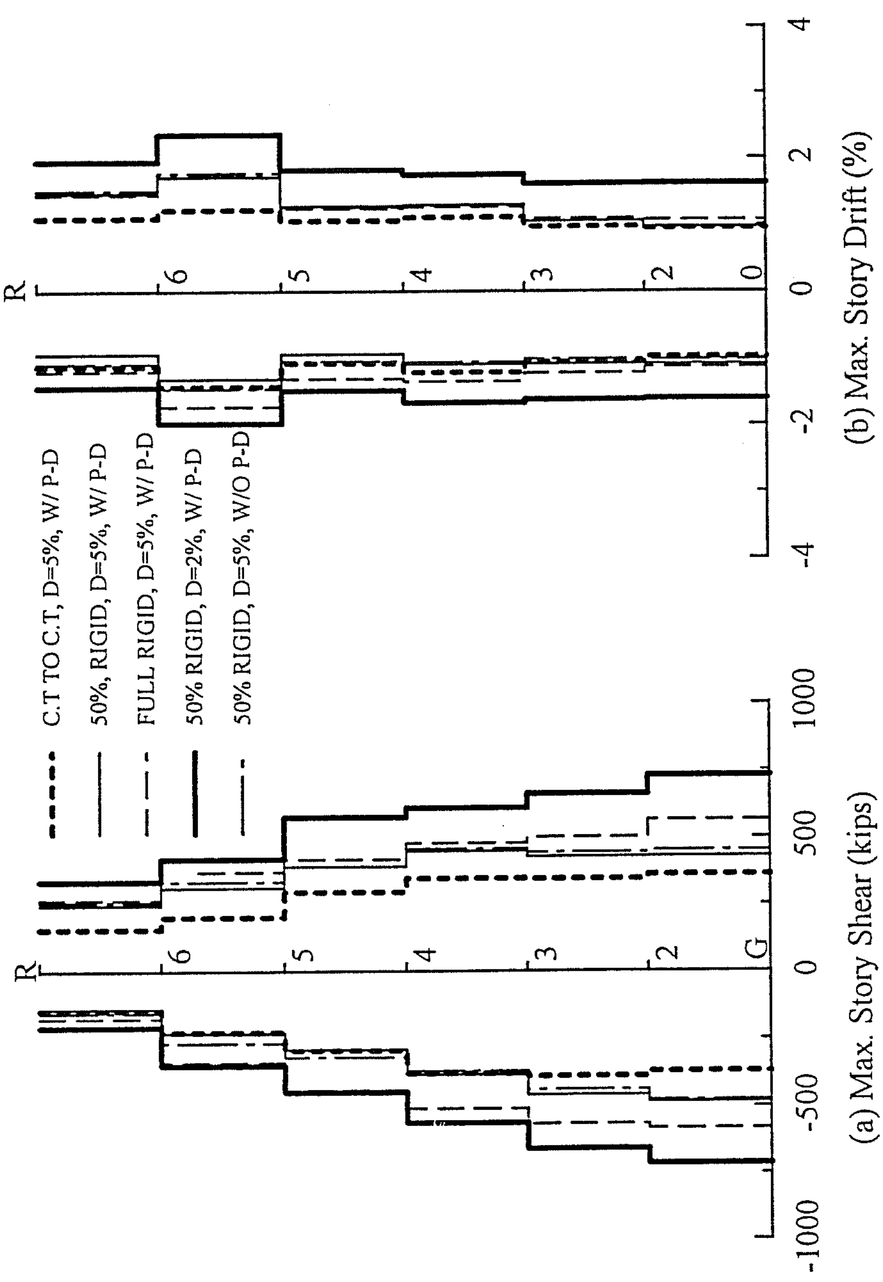


(a) Floor Displacements at Peak Roof Displacement (in.)

(b) Max. Floor Displacements (in.)

ENGLEKIRK & SABOL, INC.

MAXIMUM STORY SHEAR AND DRIFT - W/ NORTHRIDGE EARTHQUAKE



MAX. FLOOR DISPL. @MAX. ROOF DISPL. W/ NORTHRIDGE EARTHQUAKE

FLOOR	CASE1		CASE2		CASE3		CASE4		CASE5	
LEVEL	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)
ROOF	-10.29	9.45	-9.79	11.76	-12.42	11.49	-15.09	15.47	-10.00	11.91
6TH	-8.66	8.17	-8.42	9.76	-10.59	9.85	-13.15	12.80	-8.35	9.82
5TH	-6.54	6.36	-6.53	7.13	-7.97	7.40	-10.36	9.13	-6.09	7.08
4TH	-4.98	4.88	-5.10	5.20	-5.94	5.47	-8.04	6.43	-4.47	5.14
3RD	-3.33	3.28	-3.54	3.24	-3.84	3.51	-5.44	3.77	-2.89	3.25
2ND	-1.76	1.77	-1.96	1.61	-1.99	1.81	-2.93	1.75	-1.52	1.64

FLOOR DISPL. ENVELOP W/ NORTHRIDGE EARTHQUAKE

FLOOR	CASE1		CASE2		CASE3		CASE4		CASE5	
LEVEL	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)	NEG(IN)	POS(IN)
ROOF	-10.29	9.45	-9.79	11.76	-12.42	11.49	-15.09	15.47	-10.00	11.91
6TH	-8.75	8.17	-8.43	9.78	-10.59	9.85	-13.15	13.20	-8.36	9.85
5TH	-6.78	6.36	-6.68	7.19	-8.02	7.40	-10.42	10.13	-6.46	7.18
4TH	-5.15	4.88	-5.30	5.23	-6.08	5.48	-8.12	8.02	-4.96	5.23
3RD	-3.38	3.28	-3.37	3.30	-4.01	3.65	-5.51	5.53	-3.42	3.33
2ND	-1.83	1.77	-2.00	1.79	-2.11	1.99	-2.99	3.02	-1.89	1.80

NOTE: CASE1: C.T TO C.T, D=5%, W/ P-D
CASE2: 50% RIGID, D=5%, W/ P-D
CASE3: FULL RIGID, D=5%, W/ P-D
CASE4: 50% RIGID, D=2%, W/ P-D
CASE5: 50% RIGID, D=5%, W/O P-D

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MAX. STORY SHEAR W/ NORTHRIDGE EARTHQUAKE

STORY	CASE1		CASE2		CASE3		CASE4		CASE5	
LEVEL	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)	NEG(K)	POS(K)
ROOF	-145	157	-140	246	-176	267	-208	335	-155	258
6TH	-227	201	-231	310	-338	370	-349	417	-267	333
5TH	-296	295	-292	390	-441	416	-448	573	-319	389
4TH	-380	345	-384	447	-509	477	-562	608	-370	457
3RD	-390	344	-458	426	-565	499	-658	660	-438	443
2ND	-371	360	-484	428	-582	564	-714	729	-477	451

MAX. STORY DRIFT(%) W/ NORTHRIDGE EARTHQUAKE

STORY	CASE1		CASE2		CASE3		CASE4		CASE5	
LEVEL	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)	NEG(%)	POS(%)
ROOF	-1.12	1.10	-0.94	1.47	-1.20	1.45	-1.44	1.95	-1.08	1.51
6TH	-1.42	1.22	-1.31	1.71	-1.73	1.71	-1.98	2.36	-146.00	1.77
5TH	-1.07	1.06	-0.94	1.28	-1.31	1.24	-1.49	1.82	-1.04	1.26
4TH	-1.21	1.11	-1.09	1.29	-1.35	1.26	-1.67	1.75	-1.06	1.29
3RD	-1.06	0.97	-1.08	1.05	-1.22	1.09	-1.62	1.61	-1.01	1.05
2ND	-0.98	0.95	-1.08	0.96	-1.13	1.07	-1.61	1.62	-1.02	0.97

NOTE: CASE1: C.T TO C.T, D=5%, W/ P-D
CASE2: 50% RIGID, D=5%, W/ P-D
CASE3: FULL RIGID, D=5%, W/ P-D
CASE4: 50% RIGID, D=2%, W/ P-D
CASE5: 50% RIGID, D=5%, W/O P-D

ENGLEKIRK & SABOL, INC.

ELASTIC ANALYSIS RESULTS

BEAM END MOMENTS (FT-KIPS)

MEMBER #	CAP	GRAV	CASE1	CASE2	CASE3	CASE4	CASE5
F.G.3	1636	38	579	628	611	947	591
F.G.4	1636	33	579	628	611	947	591
F.2.3	1648	41	1158	1428	1736	2149	1333
F.2.4	1648	29	1158	1428	1736	2149	1333
F.3.3	1648	41	1189	1355	1668	1943	1353
F.3.4	1648	27	1189	1355	1668	1943	1353
F.4.3	1289	41	1017	1268	1440	1767	1248
F.4.4	1289	29	1017	1268	1440	1767	1248
F.5.3	1289	36	823	1151	1344	1660	1166
F.5.4	1289	27	823	1151	1344	1660	1166
F.6.3	564	43	505	706	818	927	730
F.6.4	564	38	505	706	818	927	730
F.R.3	564	74	279	442	493	593	454
F.R.4	564	71	279	442	493	593	454
G.G.3	1636	22	591	655	647	987	616
G.G.4	1636	29	591	655	647	987	616
G.2.3	1636	18	1317	1678	2100	2526	1568
G.2.4	1636	32	1317	1678	2100	2526	1568
G.3.3	1636	18	1356	1593	2031	2297	1591
G.3.4	1636	31	1356	1593	2031	2297	1591
G.4.3	1230	21	1097	1393	1608	1943	1373
G.4.4	1230	32	1097	1393	1608	1943	1373
G.5.3	1230	19	867	1236	1467	1786	1248
G.5.4	1230	27	867	1236	1467	1786	1248
G.6.3	564	31	518	730	845	952	754
G.6.4	564	35	518	730	845	952	754
G.R.3	962	80	355	595	707	799	613
G.R.4	962	83	355	595	707	799	613

NOTE:
CASE1 : C.T TO C.T, D=5%, W/ P-D — T1=2.41 SEC.
CASE2 : 50% RIGID, D=5%, W/ P-D — T1=2.27 SEC.
CASE3 : FULL RIGID, D=5%, W/ P-D — T1=2.13 SEC.
CASE4 : 50% RIGID, D=2%, W/ P-D — T1=2.27 SEC.
CASE5 : 50% RIGID, D=5%, W/O P-D — T1=2.22 SEC.

ENGLEKIRK & SABOL, INC.

DEMAND-CAPCITY RATIO BEAM END MOEMNTS							
MEMBER #	CAP('K)	GRAV	CASE1	CASE2	CASE3	CASE4	CASE5
F.G.3	1636	0.02	0.35	0.38	0.37	0.58	0.36
F.G.4	1636	0.02	0.35	0.38	0.37	0.58	0.36
F.2.3	1648	0.02	0.70	0.87	1.05	1.30	0.81
F.2.4	1648	0.02	0.70	0.87	1.05	1.30	0.81
F.3.3	1648	0.02	0.72	0.82	1.01	1.18	0.82
F.3.4	1648	0.02	0.72	0.82	1.01	1.18	0.82
F.4.3	1289	0.03	0.79	0.98	1.12	1.37	0.97
F.4.4	1289	0.02	0.79	0.98	1.12	1.37	0.97
F.5.3	1289	0.03	0.64	0.89	1.04	1.29	0.90
F.5.4	1289	0.02	0.64	0.89	1.04	1.29	0.90
F.6.3	564	0.08	0.90	1.25	1.45	1.64	1.29
F.6.4	564	0.07	0.90	1.25	1.45	1.64	1.29
F.R.3	564	0.13	0.50	0.78	0.87	1.05	0.81
F.R.4	564	0.13	0.50	0.78	0.87	1.05	0.81
G.G.3	1636	0.01	0.36	0.40	0.40	0.60	0.38
G.G.4	1636	0.02	0.36	0.40	0.40	0.60	0.38
G.2.3	1636	0.01	0.80	1.03	1.28	1.54	0.96
G.2.4	1636	0.02	0.80	1.03	1.28	1.54	0.96
G.3.3	1636	0.01	0.83	0.97	1.24	1.40	0.97
G.3.4	1636	0.02	0.83	0.97	1.24	1.40	0.97
G.4.3	1230	0.02	0.89	1.13	1.31	1.58	1.12
G.4.4	1230	0.03	0.89	1.13	1.31	1.58	1.12
G.5.3	1230	0.02	0.70	1.00	1.19	1.45	1.01
G.5.4	1230	0.02	0.70	1.00	1.19	1.45	1.01
G.6.3	564	0.05	0.92	1.29	1.50	1.69	1.34
G.6.4	564	0.06	0.92	1.29	1.50	1.69	1.34
G.R.3	962	0.08	0.37	0.62	0.74	0.83	0.64
G.R.4	962	0.09	0.37	0.62	0.74	0.83	0.64

NOTE: CASE1 : C.T TO C.T, D=5%, W/ P-D T1=2.41 SEC.
CASE2 : 50% RIGID, D=5%, W/ P-D T1=2.27 SEC.
CASE3 : FULL RIGID, D=5%, W/ P-D T1=2.13 SEC.
CASE4 : 50% RIGID, D=2%, W/ P-D T1=2.27 SEC.
CASE5 : 50% RIGID, D=5%, W/O P-D T1=2.22 SEC.

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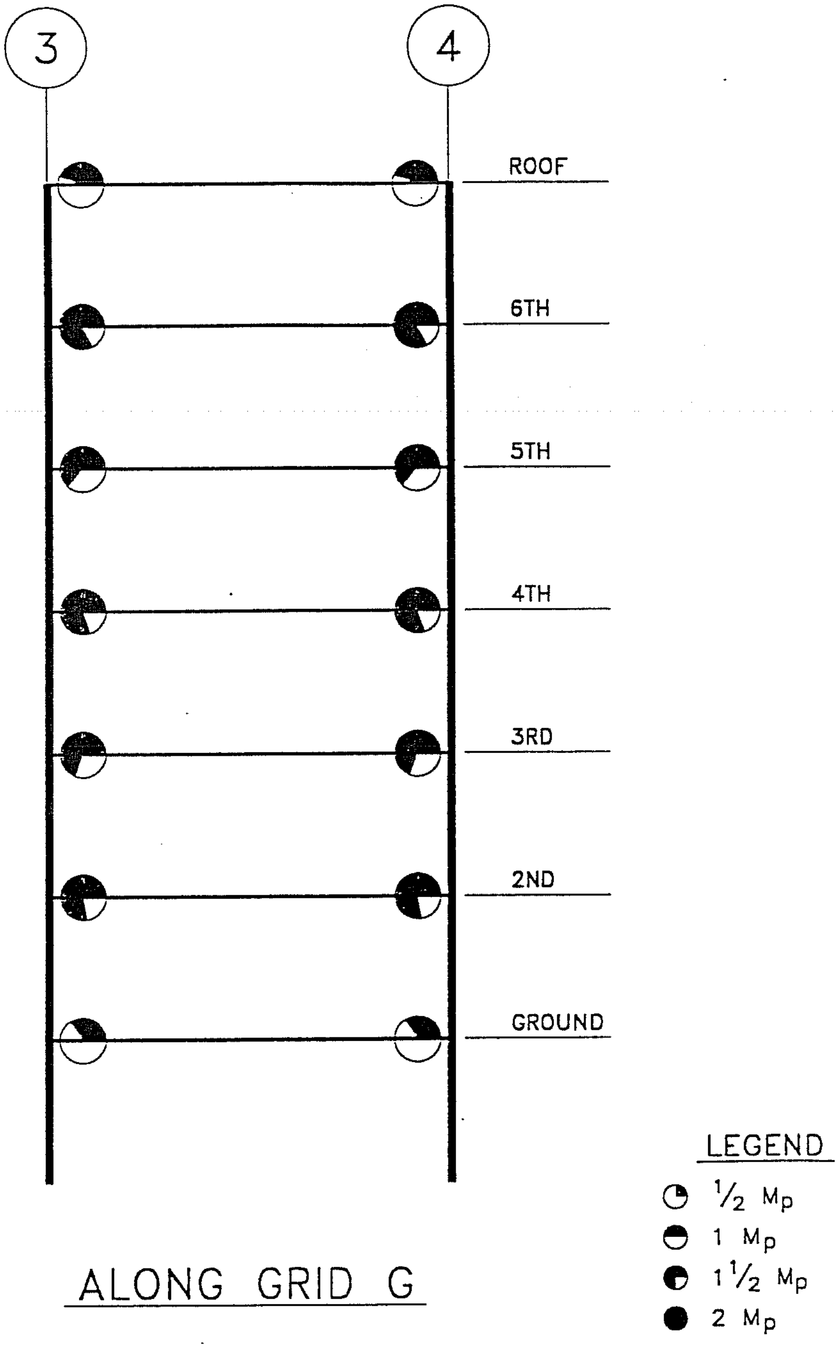
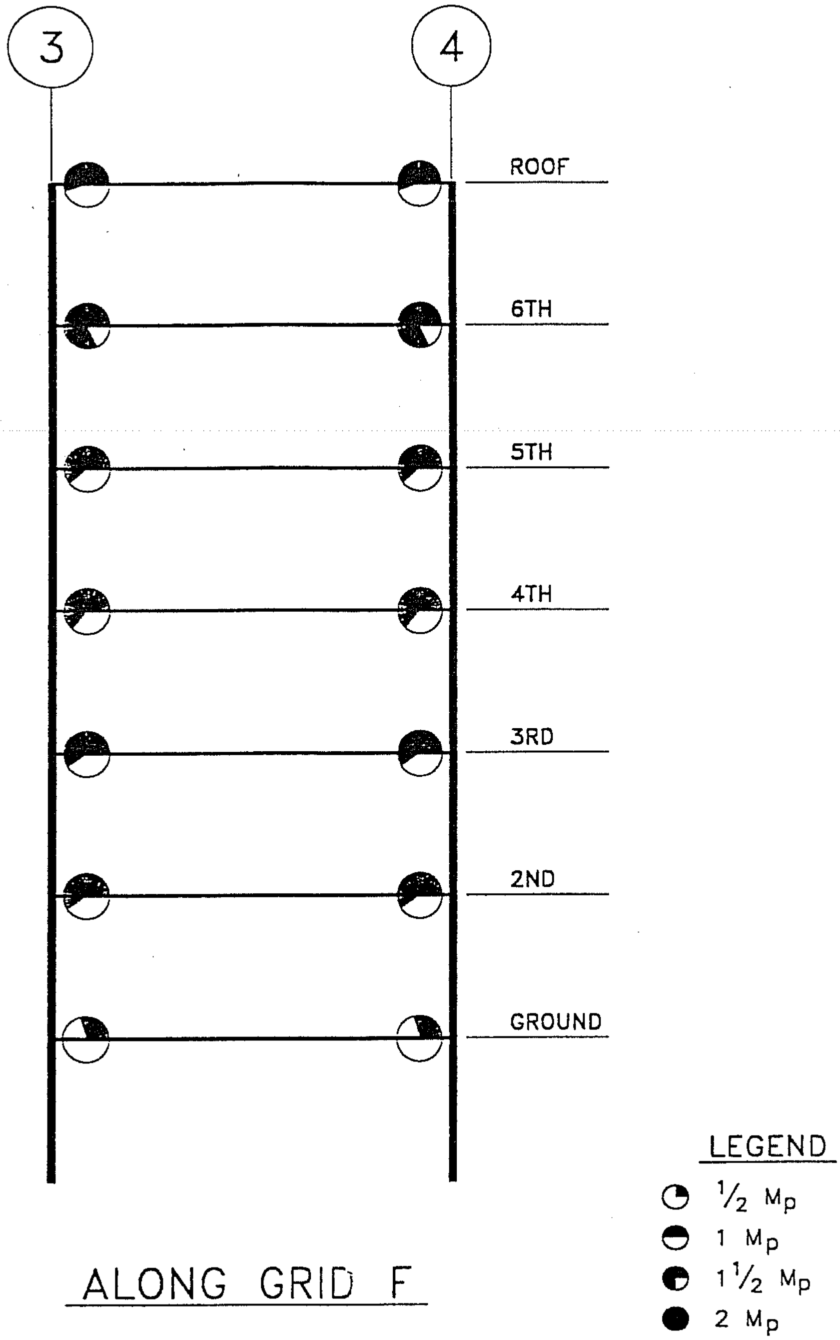


ALONG GRID G

CASE 4 (SMCH.045)



ALONG GRID F

CASE 4 (SMCH.045)

APPENDIX B - THREE-DIMENSIONAL ELASTIC ANALYSIS

This appendix contains additional data generated in the 3-D elastic analysis. The model assumptions and loading cases considered for the 3-D elastic analysis are described in Section 5.3.

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Three Dimensional Elastic Time History Analysis

Modeling

The building is modeled using SAP 90. Only the moment frames in the building have been modeled. Rigid diaphragms are assumed at each story level. The columns are assumed pinned at the basement, and a roller guide has been applied at the ground level to fix the lateral movements at that level. The boundary conditions are shown in Figure 1. Model schematics are attached in the appendix.

Earthquake Record

The Santa Monica City Hall (SMCH) ground motion record from the Northridge earthquake is used as the seismic input. The record provided by Woodward Clyde is for a duration of 60 seconds. However, the first 38 seconds of the record is used due to SAP90 memory usage limitations. The maximum values of element forces occur in the general range of 10-20 seconds.

Trial Analyses

Five different trial analyses were performed. The differences between these trials are explained below and highlighted in the *Trial Comparison Matrix*. Only the beams in frames on lines F and G have been examined. The D/C ratios are calculated as the ratio of the demand moment from the analyses to the nominal plastic moment of the beams. Axial loads in the beams have been neglected.

Trial 1

The first trial used approximate mass moments of inertia based on a rectangular plan for each story. The gravity loads used are *expected* loads i.e. 1.0D+0.08L on the structure. Gravity loads for beams are only applied to moment frames on lines F and G. Vertical acceleration component of the earthquake is not used. Damping is 5% and the building is modeled along centerlines without any rigid-end offsets.

Trial 2

More precisely calculated mass moments of inertia are used for each story. The gravity loads were also modified to the benchmark combination of 1.0D+0.5L. Vertical acceleration is included. The results indicated non-representative 12 second periods of vibration for the vertical modes. This occurred because only moment frames have been modeled, which makes for a very flexible structure supporting the total mass. As such, the vertical modes of vibration did not have any significant effects on the beam D/C ratios because the demand is much less in the range of high time periods.

Trial 3

This trial is exactly the same as trial 2 except that the vertical acceleration is not applied.

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Three Dimensional Elastic Time History Analysis (cont.)

Trial 4

In this trial, an attempt was made to use more realistic modeling assumptions than the benchmark assumptions. As such, rigid offsets are used at beam-column joints with 50% rigidity of joints. The damping is reduced to 2% as more realistic for a steel framed building. The gravity load remained at benchmark 1.0D+0.5L because the effect of these loads on the demand to capacity ratios of beams is negligible.

Trial 5

This trial is same as trial 4 except that torsion is locked in the analysis. This was done in an effort to find an explanation for the discrepancy between the three dimensional analysis results and the two dimensional analysis results. This trial yielded D/C ratios in good agreement with the ratios from the two dimensional analysis.

TRIAL COMPARISON MATRIX

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Mass Moment of Inertia	Approximate	Calculated	Calculated	Calculated	Calculated
Gravity Loads	Expected	Benchmark	Benchmark	Benchmark	Benchmark
Modeling	Centerline	Centerline	Centerline	Rigid Offsets	Rigid Offsets
Vertical Component	No	Yes	No	No	No
Torsional Lock	No	No	No	No	Yes
Damping	5%	5%	5%	2%	2%

PROJECT	SAC TASKS 3.1 & 3.5		
JOB NO.	9501.020		
DATE	03/95	BY	ASH
SHEET	4	OF	

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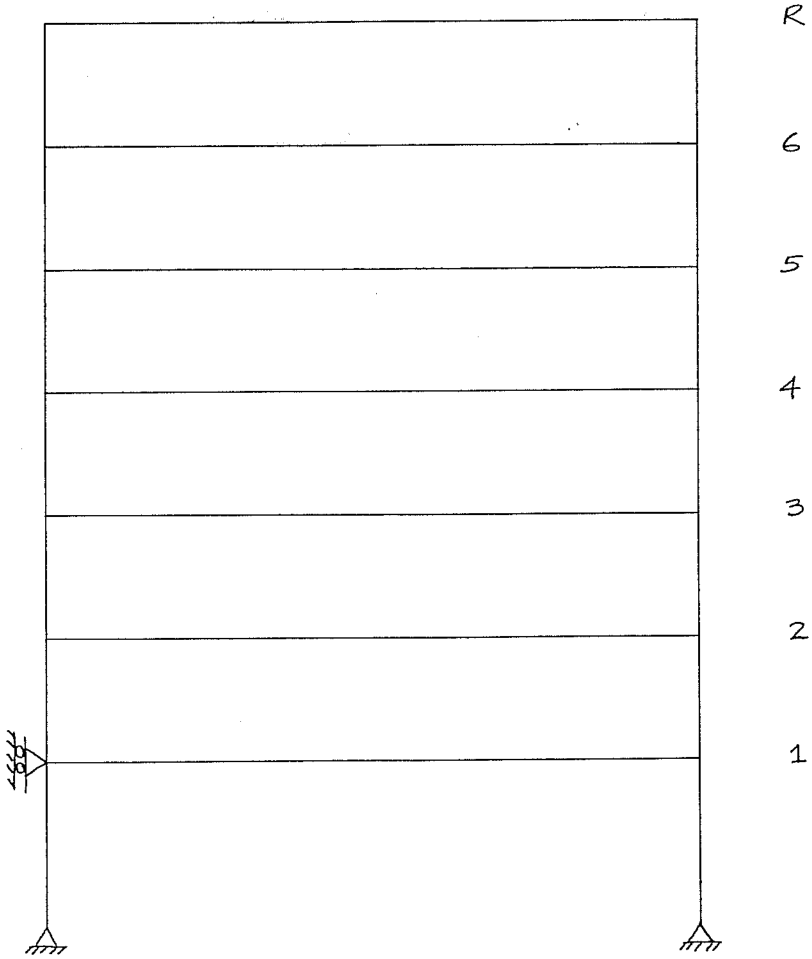
BOUNDARY CONDITIONS



TYPICAL MOMENT FRAME

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9501.020 "SAC TASKS 3.1 & 3.5" 3-D ELASTIC ANALYSIS

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.412761E+01	.203165E+01	.323347	3.092649
2	.660061E+01	.256917E+01	.408895	2.445613
3	.757374E+01	.275204E+01	.438001	2.283099
4	.194619E+02	.441157E+01	.702123	1.424253
5	.470677E+02	.686059E+01	1.091897	.915838
6	.528122E+02	.726720E+01	1.156610	.864595
7	.886912E+02	.941760E+01	1.498858	.667175
8	.152540E+03	.123507E+02	1.965675	.508731
9	.194498E+03	.139463E+02	2.219618	.450528
10	.252231E+03	.158818E+02	2.527662	.395622

B A S E   F O R C E   R E A C T I O N   F A C T O R S

MODE #	PERIOD (sec)	X DIRECTION	Y DIRECTION	Z DIRECTION	X MOMENT	Y MOMENT	Z MOMENT
1	3.093	-.103E+01	.103E+01	.000E+00	-.862E+03	-.933E+03	.397E+04
2	2.446	.369E+01	.556E+00	.000E+00	-.432E+03	.306E+04	-.110E+04
3	2.283	-.162E+00	.382E+01	.000E+00	-.314E+04	-.125E+03	.198E+04
4	1.424	.153E+01	-.458E+00	.000E+00	.191E+03	.770E+03	-.216E+04
5	.916	.963E+00	.113E+01	.000E+00	-.208E+03	.103E+03	.337E+03
6	.865	.639E+00	-.108E+01	.000E+00	.134E+03	.287E+02	-.845E+03
7	.667	-.561E+00	.240E+00	.000E+00	-.660E+02	-.699E+02	.749E+03
8	.509	.161E+00	.748E+00	.000E+00	-.142E+03	.646E+02	.546E+03
9	.451	.983E+00	-.103E+00	.000E+00	.127E+02	.125E+03	-.719E+03
10	.396	-.859E-01	-.712E+00	.000E+00	.953E+02	.277E+02	-.276E+03

P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	5.349	5.352	00.000	5.349	5.352	00.000
2	68.254	1.544	00.000	73.603	6.897	00.000
3	.132	73.137	00.000	73.735	80.034	00.000
4	11.732	1.049	00.000	85.466	81.083	00.000
5	4.639	6.367	00.000	90.105	87.450	00.000
6	2.043	5.888	00.000	92.148	93.338	00.000
7	1.575	.288	00.000	93.723	93.626	00.000
8	.130	2.797	00.000	93.853	96.423	00.000
9	4.838	.053	00.000	98.691	96.476	00.000
10	.037	2.537	00.000	98.727	99.013	00.000

TRIAL-1

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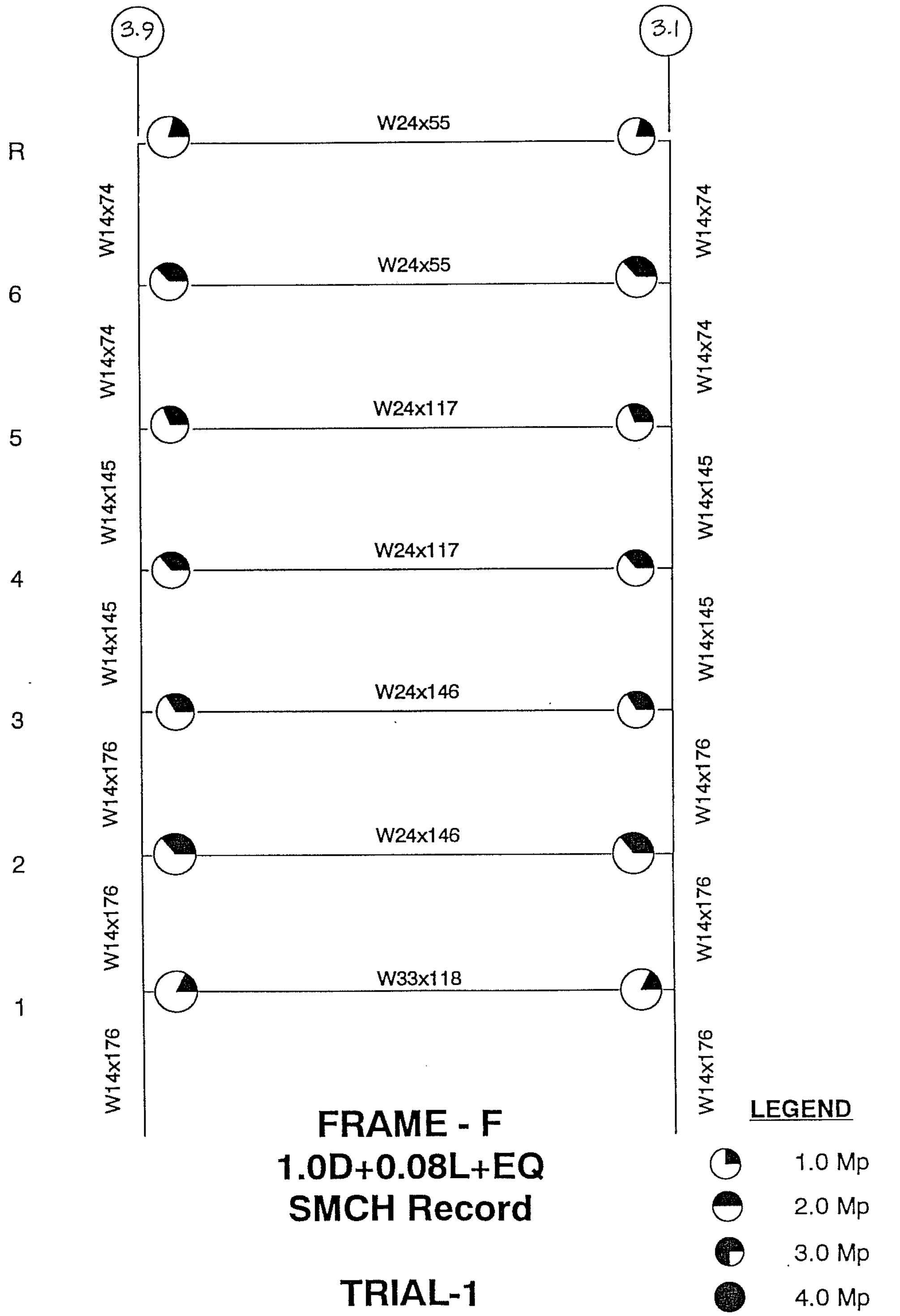

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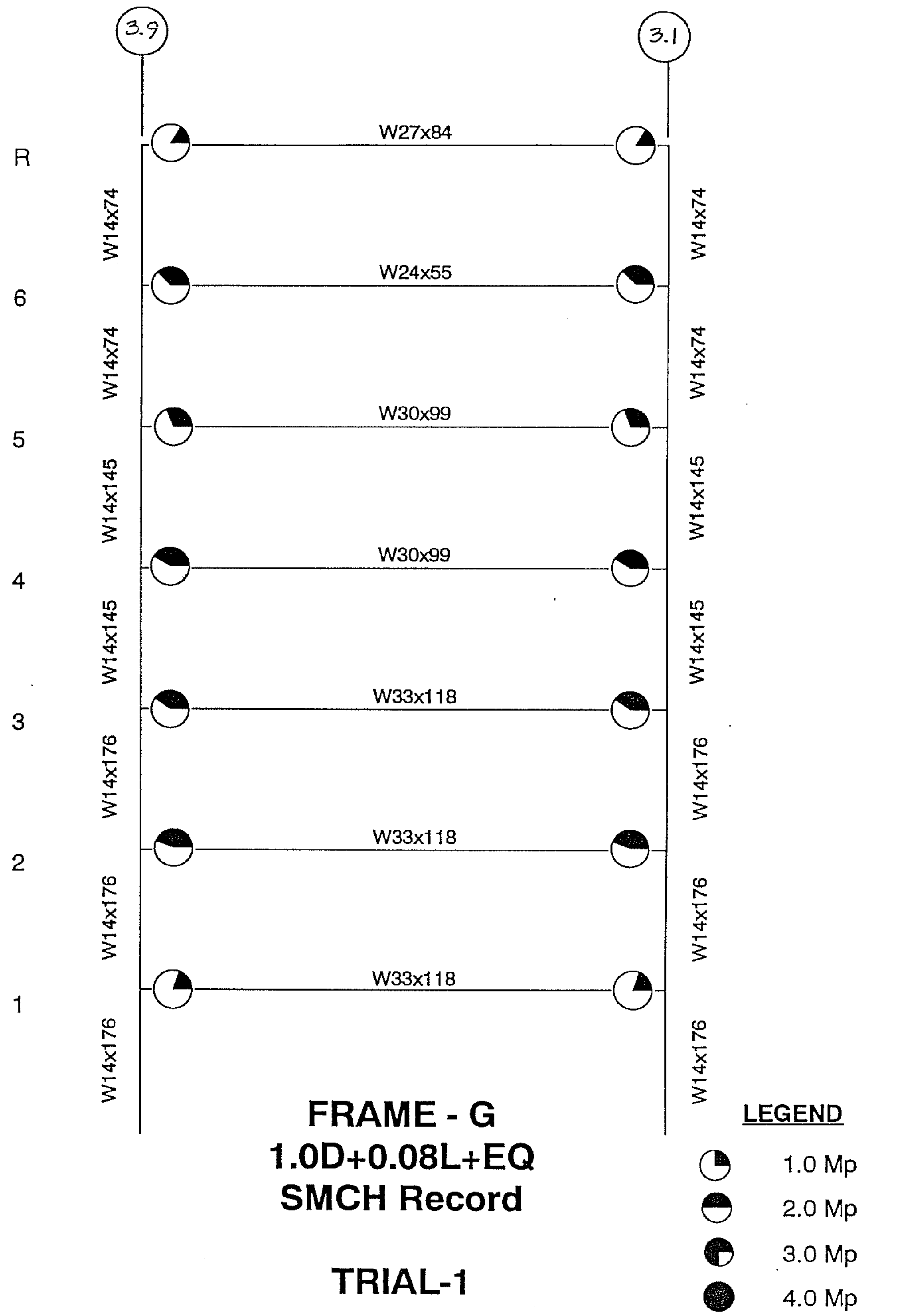

FRAME - G
1.0D+0.08L+EQ
SMCH Record

TRIAL-1

Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)

SOH & Associates, Structural Engineers

9501.020 "SAC TASKS 3.1 & 3.5" 3-D ELASTIC ANALYSIS

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.273891E+00	.523346E+00	.083293	12.005803
2	.289324E+00	.537888E+00	.085608	11.681204
3	.319536E+00	.565275E+00	.089966	11.115270
4	.353665E+00	.594697E+00	.094649	10.565347
5	.476679E+01	.218330E+01	.347483	2.877840
6	.670986E+01	.259034E+01	.412265	2.425622
7	.772998E+01	.278028E+01	.442496	2.259908
8	.216045E+02	.464807E+01	.739763	1.351784
9	.485057E+02	.696461E+01	1.108452	.902159
10	.604989E+02	.777811E+01	1.237924	.807804

B A S E   F O R C E   R E A C T I O N   F A C T O R S

MODE #	PERIOD (sec)	X DIRECTION	Y DIRECTION	Z DIRECTION	X MOMENT	Y MOMENT	Z MOMENT
1	12.006	.000E+00	.000E+00	.278E+01	.183E+04	-.200E+04	.000E+00
2	11.681	.000E+00	.000E+00	.217E+01	.143E+04	-.156E+04	.000E+00
3	11.115	.000E+00	.000E+00	-.211E+01	-.139E+04	.152E+04	.000E+00
4	10.565	.000E+00	.000E+00	.177E+01	.117E+04	-.127E+04	.000E+00
5	2.878	.145E+01	-.133E+01	.000E+00	.110E+04	.128E+04	-.409E+04
6	2.426	.354E+01	.113E+01	.000E+00	-.907E+03	.293E+04	-.552E+03
7	2.260	.485E+00	-.360E+01	.000E+00	.296E+04	.382E+03	-.176E+04
8	1.352	.154E+01	-.475E+00	.000E+00	.182E+03	.709E+03	-.208E+04
9	.902	.719E+00	.148E+01	.000E+00	-.252E+03	.840E+02	.629E+03
10	.808	.106E+01	-.659E+00	.000E+00	.623E+02	.617E+02	-.830E+03

P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	00.000	00.000	38.609	00.000	00.000	38.609
2	00.000	00.000	23.536	00.000	00.000	62.145
3	00.000	00.000	22.186	00.000	00.000	84.330
4	00.000	00.000	15.670	00.000	00.000	100.000
5	10.590	8.813	00.000	10.590	8.813	100.000
6	62.776	6.404	00.000	73.366	15.217	100.000
7	1.177	64.836	00.000	74.543	80.053	100.000
8	11.930	1.131	00.000	86.472	81.183	100.000
9	2.590	10.991	00.000	89.062	92.174	100.000
10	5.617	2.170	00.000	94.679	94.345	100.000

TRIAL-2

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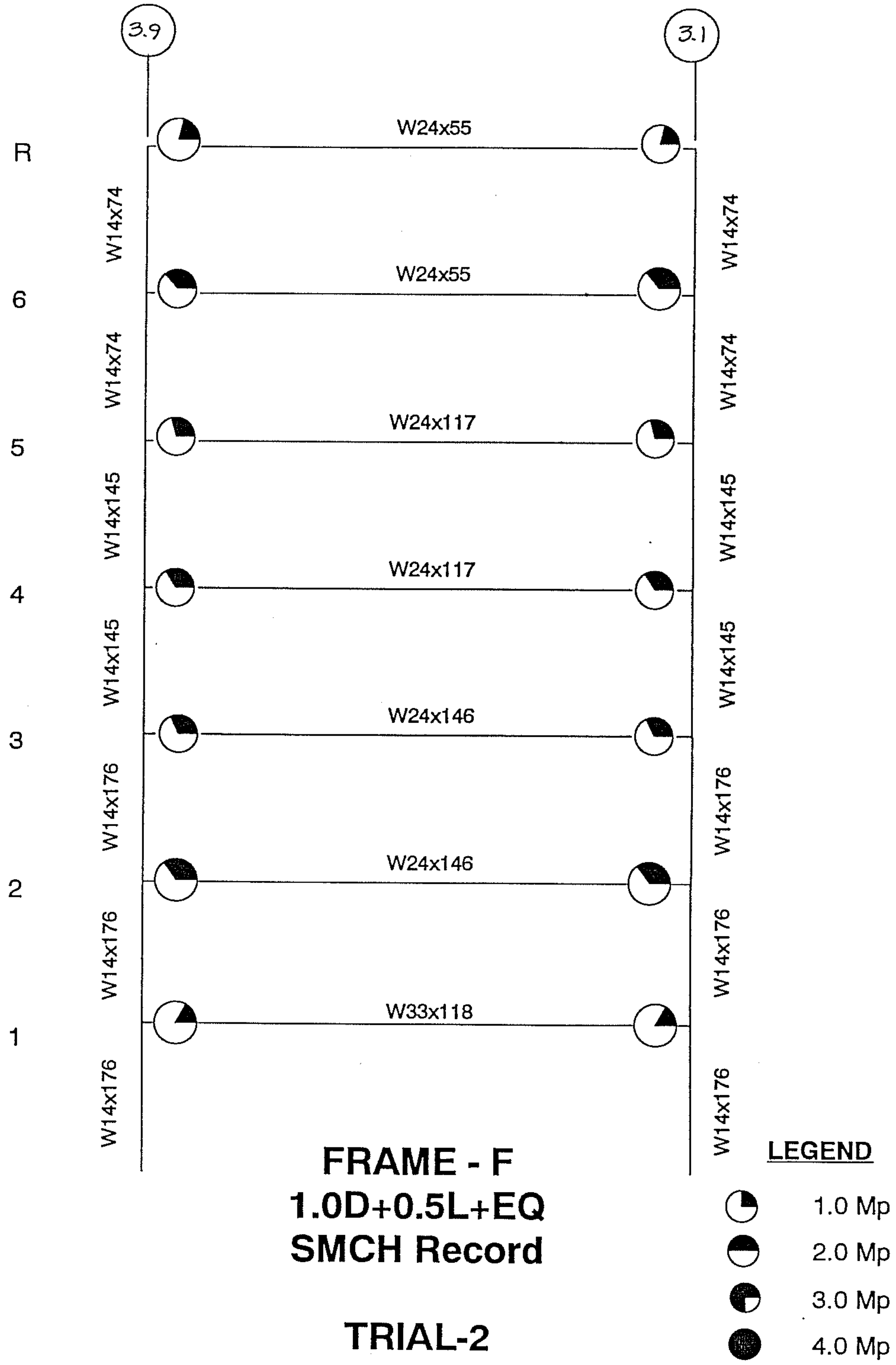

FRAME - F
1.0D+0.5L+EQ
SMCH Record

TRIAL-2

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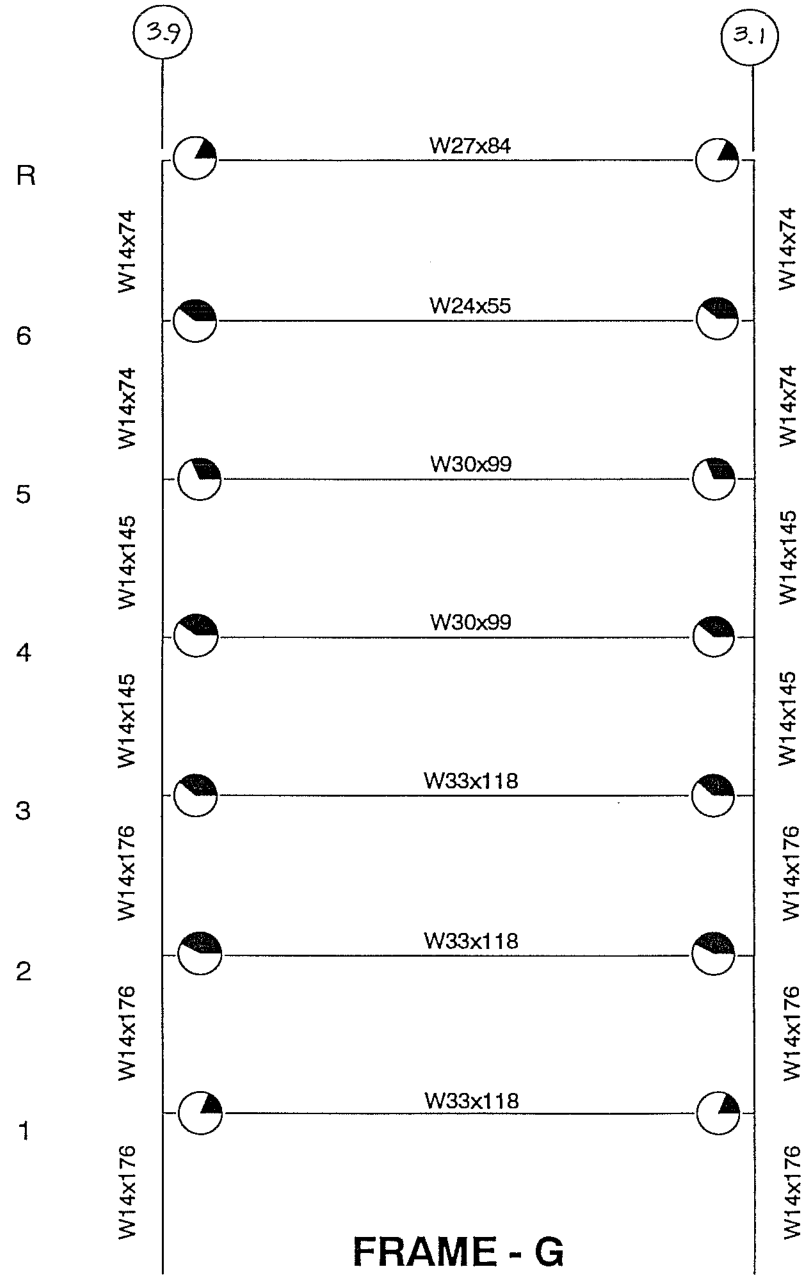
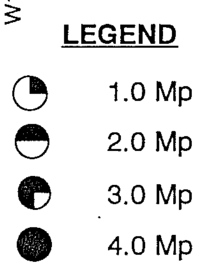


FRAME - G
1.0D+0.5L+EQ
SMCH Record

TRIAL-2



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9501.020 "SAC TASKS 3.1 & 3.5" 3-D ELASTIC ANALYSIS

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.476679E+01	.218330E+01	.347483	2.877840
2	.670986E+01	.259034E+01	.412265	2.425623
3	.772996E+01	.278028E+01	.442495	2.259911
4	.215822E+02	.464567E+01	.739381	1.352483
5	.476851E+02	.690544E+01	1.099035	.909889
6	.555453E+02	.745287E+01	1.186161	.843056
7	.978891E+02	.989389E+01	1.574662	.635057
8	.158044E+03	.125716E+02	2.000826	.499793
9	.202091E+03	.142159E+02	2.262526	.441984
10	.291333E+03	.170685E+02	2.716534	.368116

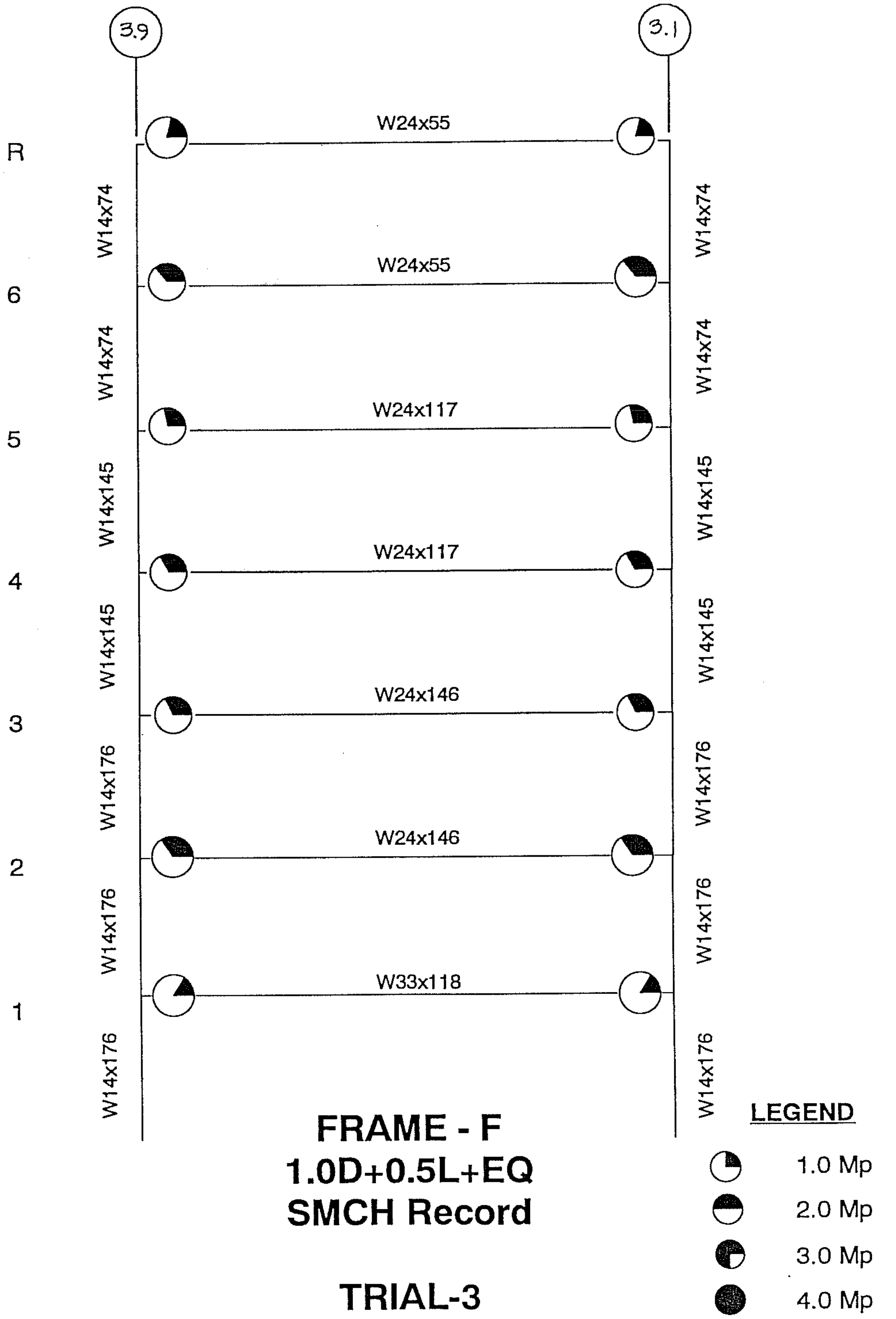
B A S E   F O R C E   R E A C T I O N   F A C T O R S

MODE #	PERIOD (sec)	X DIRECTION	Y DIRECTION	Z DIRECTION	X MOMENT	Y MOMENT	Z MOMENT
1	2.878	.146E+01	-.133E+01	.000E+00	.110E+04	.128E+04	-.409E+04
2	2.426	.354E+01	.113E+01	.000E+00	-.907E+03	.293E+04	-.552E+03
3	2.260	.485E+00	-.360E+01	.000E+00	.296E+04	.382E+03	-.176E+04
4	1.352	.154E+01	-.471E+00	.000E+00	.182E+03	.710E+03	-.208E+04
5	.910	.788E+00	.133E+01	.000E+00	-.234E+03	.822E+02	.530E+03
6	.843	.734E+00	-.820E+00	.000E+00	.833E+02	.207E+02	-.682E+03
7	.635	.590E+00	-.250E+00	.000E+00	.643E+02	.750E+02	-.747E+03
8	.500	.204E+00	.811E+00	.000E+00	-.149E+03	.634E+02	.526E+03
9	.442	.952E+00	-.124E+00	.000E+00	.159E+02	.120E+03	-.710E+03
10	.368	.871E-01	.650E+00	.000E+00	-.828E+02	-.277E+02	.244E+03

P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	10.590	8.812	00.000	10.590	8.812	00.000
2	62.775	6.403	00.000	73.365	15.215	00.000
3	1.176	64.843	00.000	74.541	80.058	00.000
4	11.930	1.111	00.000	86.472	81.168	00.000
5	3.103	8.820	00.000	89.574	89.989	00.000
6	2.698	3.365	00.000	92.272	93.354	00.000
7	1.739	.312	00.000	94.012	93.666	00.000
8	.208	3.287	00.000	94.220	96.953	00.000
9	4.533	.078	00.000	98.753	97.031	00.000
10	.038	2.110	00.000	98.791	99.141	00.000

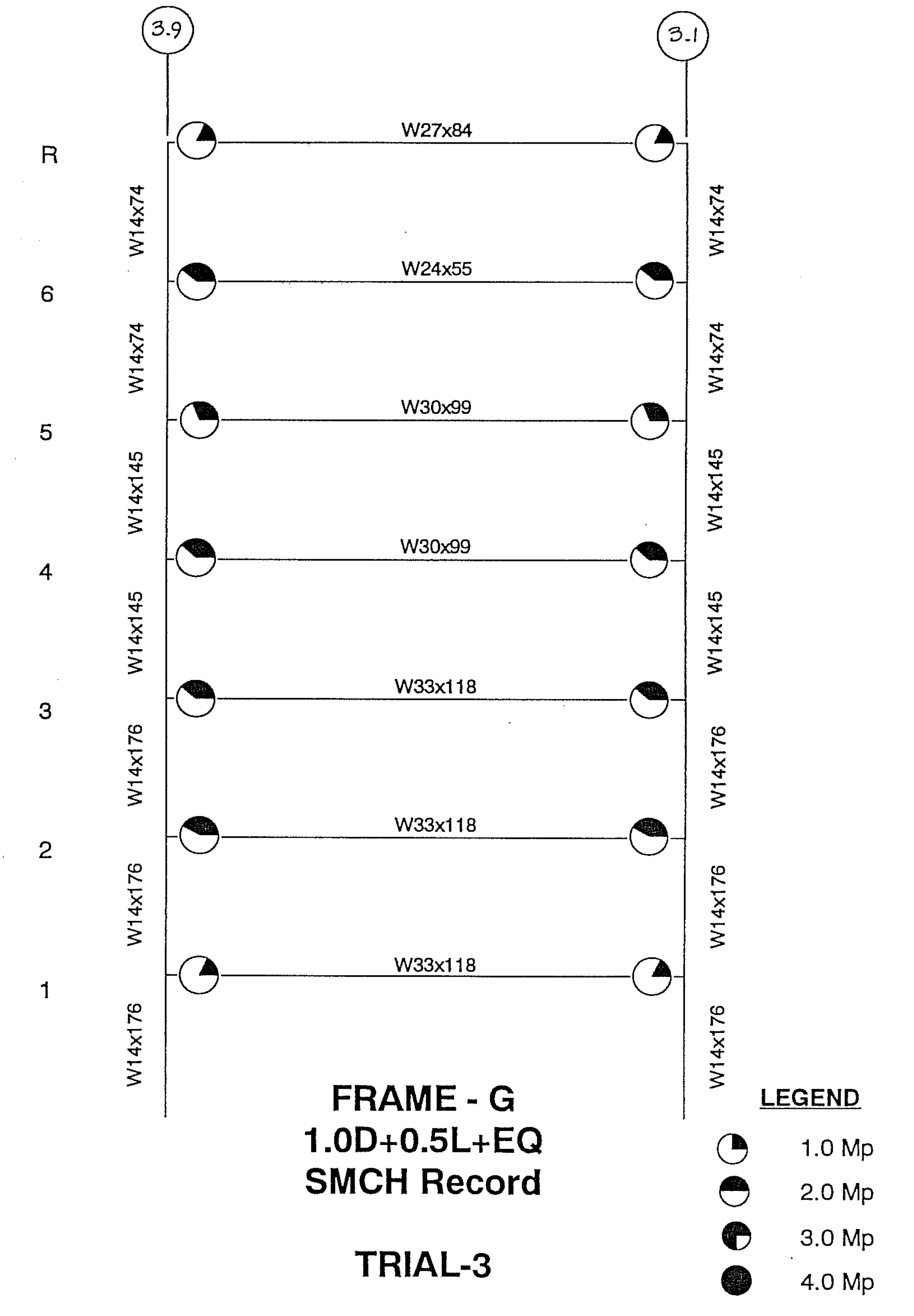
TRIAL-3



FRAME - F
1.0D+0.5L+EQ
SMCH Record

TRIAL-3

SOH & Associates
Structural Engineers



FRAME - G
1.0D+0.5L+EQ
SMCH Record

TRIAL-3

Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)

SOH & Associates, Structural Engineers

9501.020 "SAC TASKS 3.1 & 3.5" 3-D ELASTIC ANALYSIS

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.553462E+01	.235258E+01	.374424	2.670767
2	.782894E+01	.279802E+01	.445319	2.245579
3	.885933E+01	.297646E+01	.473719	2.110958
4	.252808E+02	.502801E+01	.800232	1.249638
5	.551730E+02	.742786E+01	1.182180	.845895
6	.640083E+02	.800052E+01	1.273322	.785347
7	.116794E+03	.108071E+02	1.720007	.581393
8	.185932E+03	.136357E+02	2.170188	.460789
9	.239244E+03	.154675E+02	2.461733	.406218
10	.347468E+03	.186405E+02	2.966729	.337072

B A S E   F O R C E   R E A C T I O N   F A C T O R S

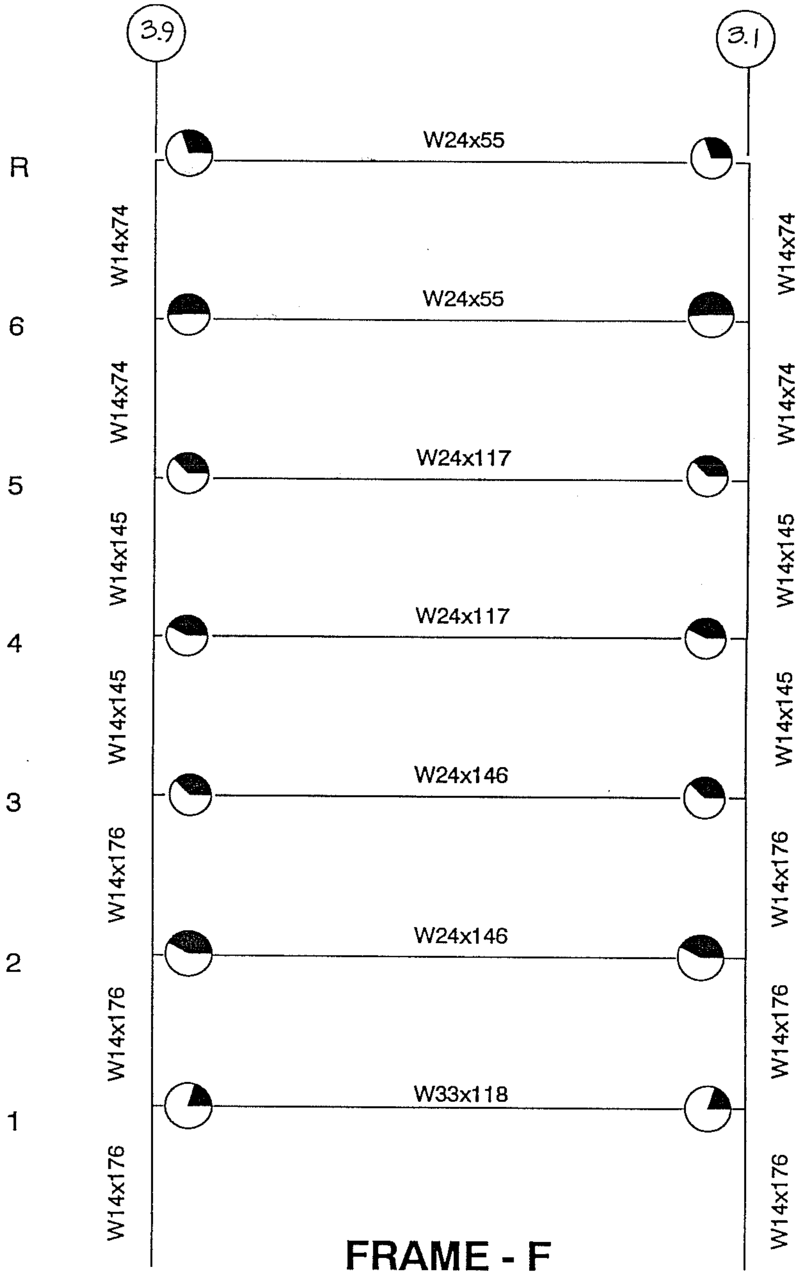
MODE #	PERIOD (sec)	X DIRECTION	Y DIRECTION	Z DIRECTION	X MOMENT	Y MOMENT	Z MOMENT
1	2.671	.139E+01	-.135E+01	.000E+00	.113E+04	.123E+04	-.408E+04
2	2.246	.356E+01	.117E+01	.000E+00	-.940E+03	.295E+04	-.606E+03
3	2.111	.545E+00	-.357E+01	.000E+00	.293E+04	.433E+03	-.175E+04
4	1.250	.152E+01	-.483E+00	.000E+00	.191E+03	.698E+03	-.207E+04
5	.846	.792E+00	.135E+01	.000E+00	-.247E+03	.887E+02	.531E+03
6	.785	.759E+00	-.817E+00	.000E+00	.913E+02	.305E+02	-.699E+03
7	.581	.593E+00	-.265E+00	.000E+00	.692E+02	.744E+02	-.765E+03
8	.461	.256E+00	.816E+00	.000E+00	-.152E+03	.680E+02	.489E+03
9	.406	.936E+00	-.186E+00	.000E+00	.256E+02	.121E+03	-.732E+03
10	.337	.130E+00	.639E+00	.000E+00	-.832E+02	-.204E+02	.213E+03

P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	9.683	9.169	00.000	9.683	9.169	00.000
2	63.482	6.830	00.000	73.165	16.000	00.000
3	1.487	63.613	00.000	74.652	79.613	00.000
4	11.543	1.166	00.000	86.195	80.778	00.000
5	3.139	9.059	00.000	89.334	89.838	00.000
6	2.883	3.342	00.000	92.217	93.180	00.000
7	1.758	.351	00.000	93.976	93.531	00.000
8	.327	3.328	00.000	94.303	96.859	00.000
9	4.381	.172	00.000	98.684	97.031	00.000
10	.084	2.045	00.000	98.768	99.077	00.000

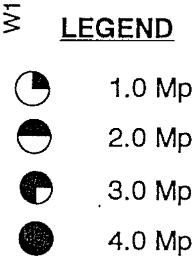
TRIAL-4

SOH & Associates
Structural Engineers



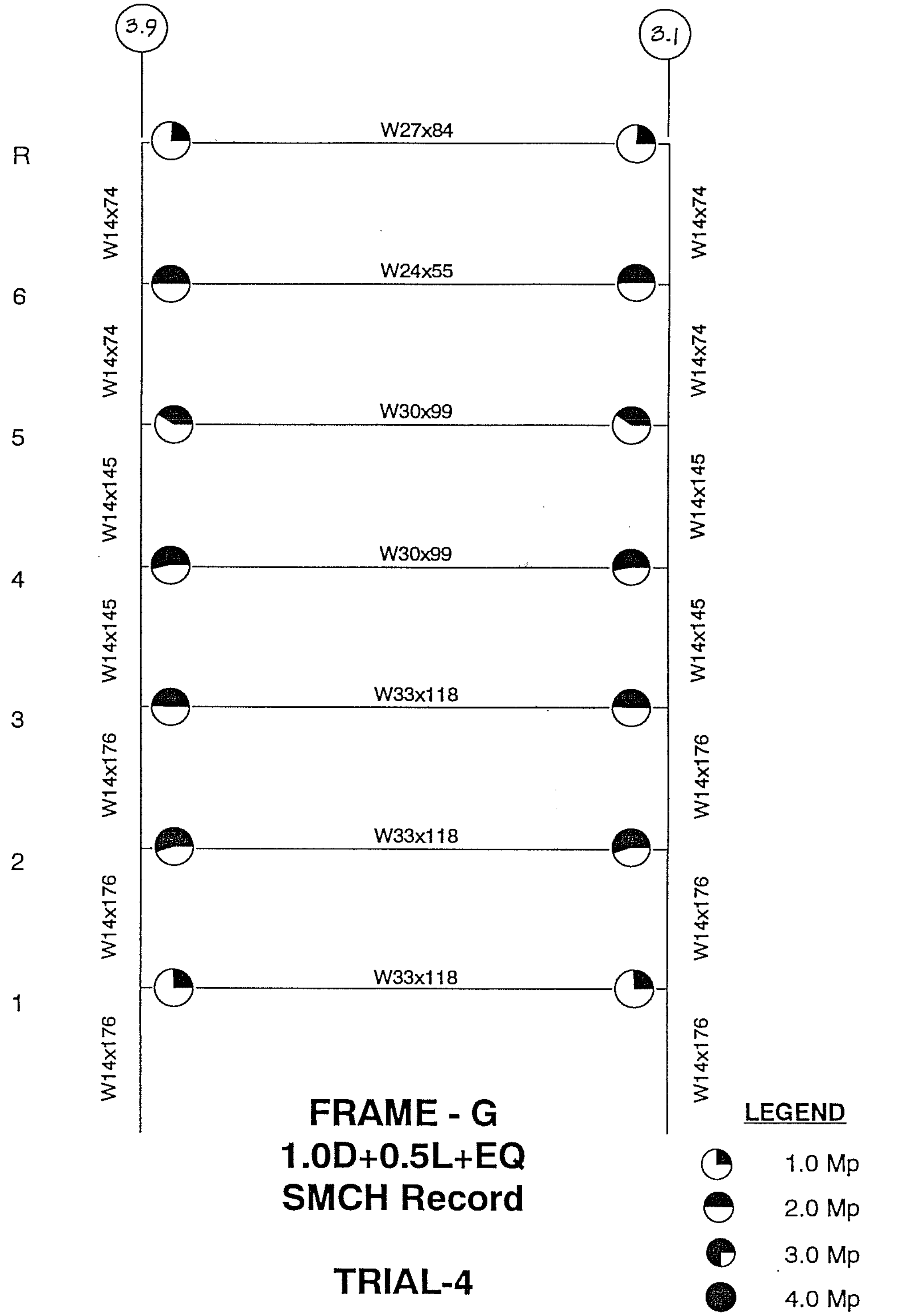
FRAME - F
1.0D+0.5L+EQ
SMCH Record

TRIAL-4



Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)

SOH & Associates
Structural Engineers



FRAME - G
1.0D+0.5L+EQ
SMCH Record

TRIAL-4

SOH & Associates, Structural Engineers

9501.020 "SAC TASKS 3.1 & 3.5" 3-D ELASTIC ANALYSIS

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.819607E+01	.286288E+01	.455641	2.194709
2	.851799E+01	.291856E+01	.464503	2.152837
3	.501978E+02	.708504E+01	1.127619	.886824
4	.573987E+02	.757619E+01	1.205789	.829333
5	.191217E+03	.138281E+02	2.200812	.454378
6	.200835E+03	.141716E+02	2.255485	.443364
7	.375181E+03	.193696E+02	3.082765	.324384
8	.466775E+03	.216050E+02	3.438540	.290821
9	.866468E+03	.294358E+02	4.684859	.213454
10	.942258E+03	.306962E+02	4.885455	.204689

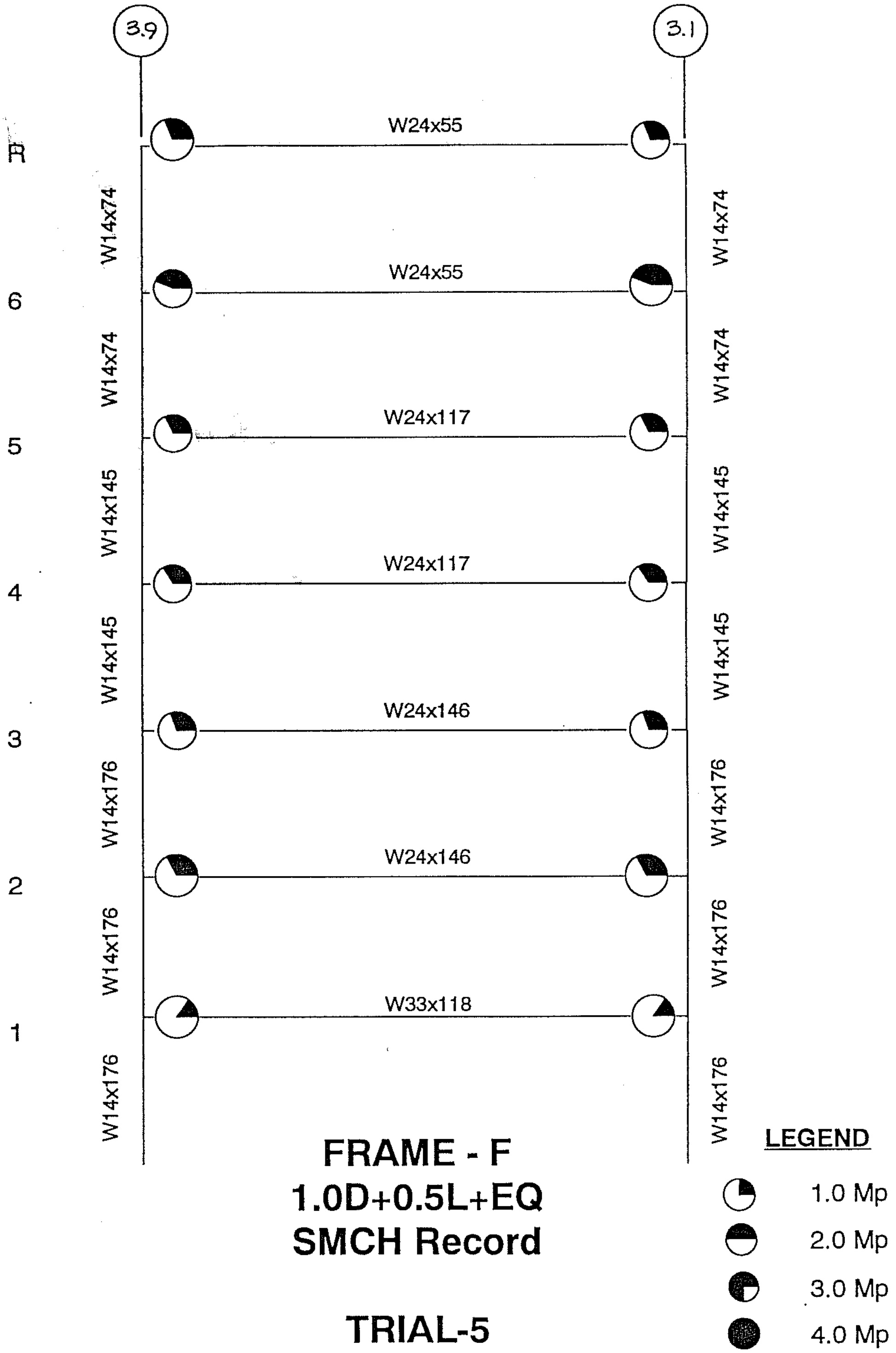
B A S E   F O R C E   R E A C T I O N   F A C T O R S

MODE #	PERIOD (sec)	X DIRECTION	Y DIRECTION	Z DIRECTION	X MOMENT	Y MOMENT	Z MOMENT
1	2.195	.401E+01	-.320E+00	.000E+00	.264E+03	.328E+04	-.288E+04
2	2.153	.327E+00	.398E+01	.000E+00	-.328E+04	.265E+03	.265E+04
3	.887	.153E+01	-.443E-01	.000E+00	.144E+02	.239E+03	-.104E+04
4	.829	.298E-01	.163E+01	.000E+00	-.284E+03	.278E+01	.116E+04
5	.454	.970E+00	-.512E-02	.000E+00	-.745E-01	.141E+03	-.644E+03
6	.443	.576E-02	.930E+00	.000E+00	-.166E+03	.845E+00	.666E+03
7	.324	.461E+00	-.906E-03	.000E+00	.279E+00	.426E+02	-.305E+03
8	.291	.373E-03	.488E+00	.000E+00	-.563E+02	-.206E+00	.351E+03
9	.213	.521E+00	-.351E-03	.000E+00	-.169E-01	.117E+03	-.344E+03
10	.205	.391E-03	.467E+00	.000E+00	-.888E+02	.159E+00	.336E+03

P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	80.465	.512	00.000	80.465	.512	00.000
2	.535	79.398	00.000	81.000	79.910	00.000
3	11.732	.010	00.000	92.732	79.920	00.000
4	.004	13.362	00.000	92.736	93.282	00.000
5	4.706	.000	00.000	97.443	93.282	00.000
6	.000	4.328	00.000	97.443	97.610	00.000
7	1.062	.000	00.000	98.505	97.610	00.000
8	.000	1.189	00.000	98.505	98.799	00.000
9	1.358	.000	00.000	99.863	98.799	00.000
10	.000	1.089	00.000	99.863	99.888	00.000

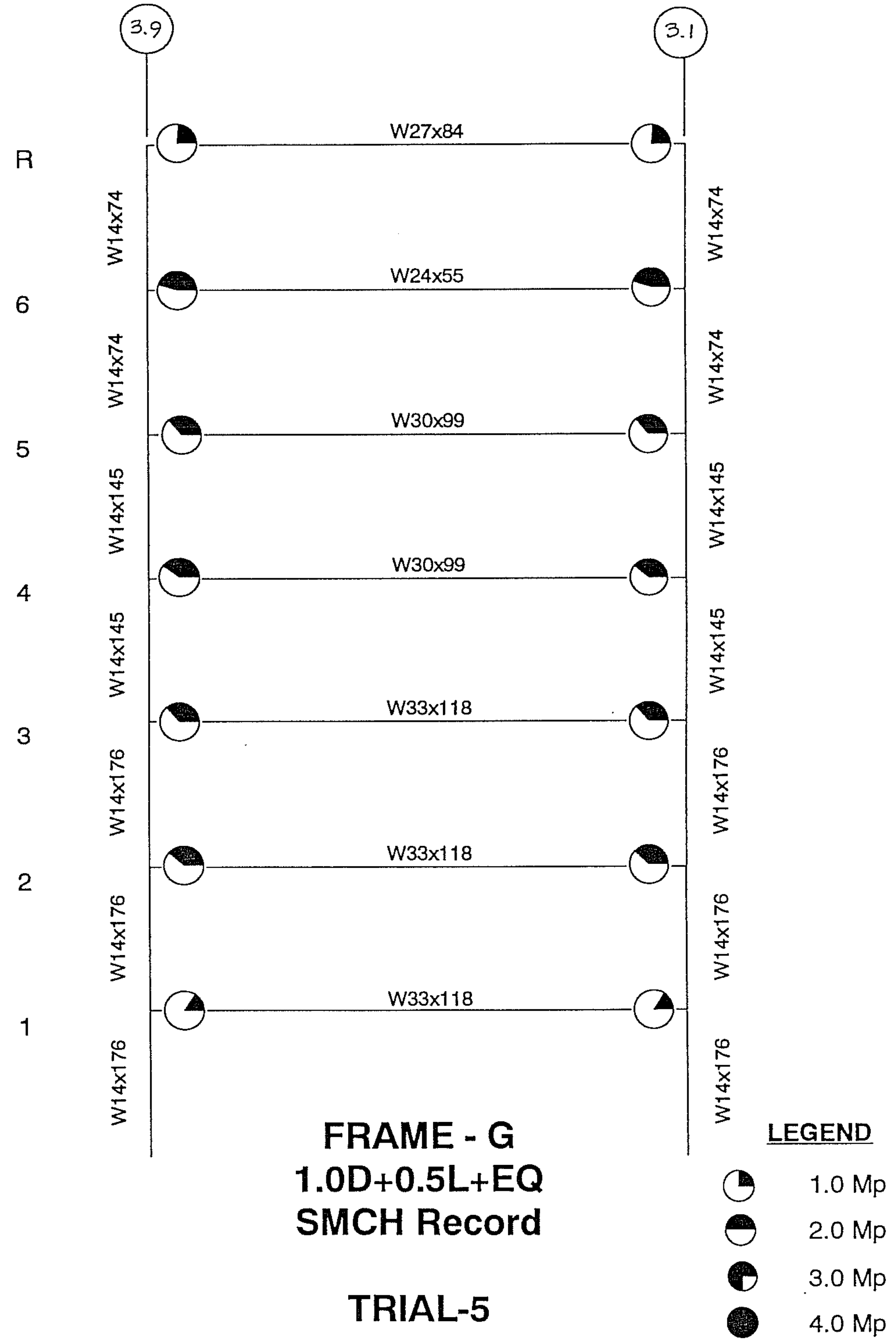
TRIAL-5



FRAME - F
1.0D+0.5L+EQ
SMCH Record

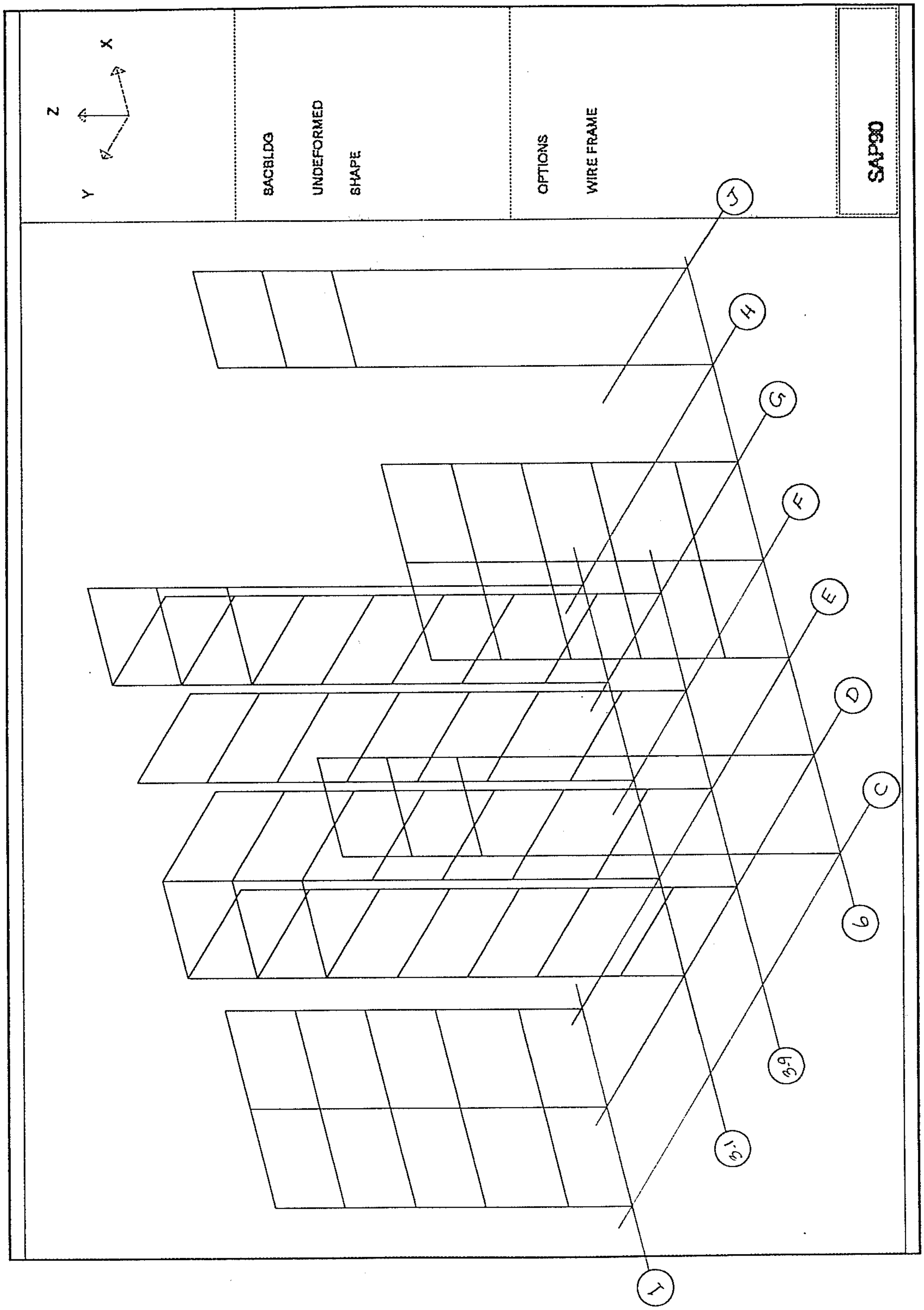
TRIAL-5

SOH & Associates
Structural Engineers



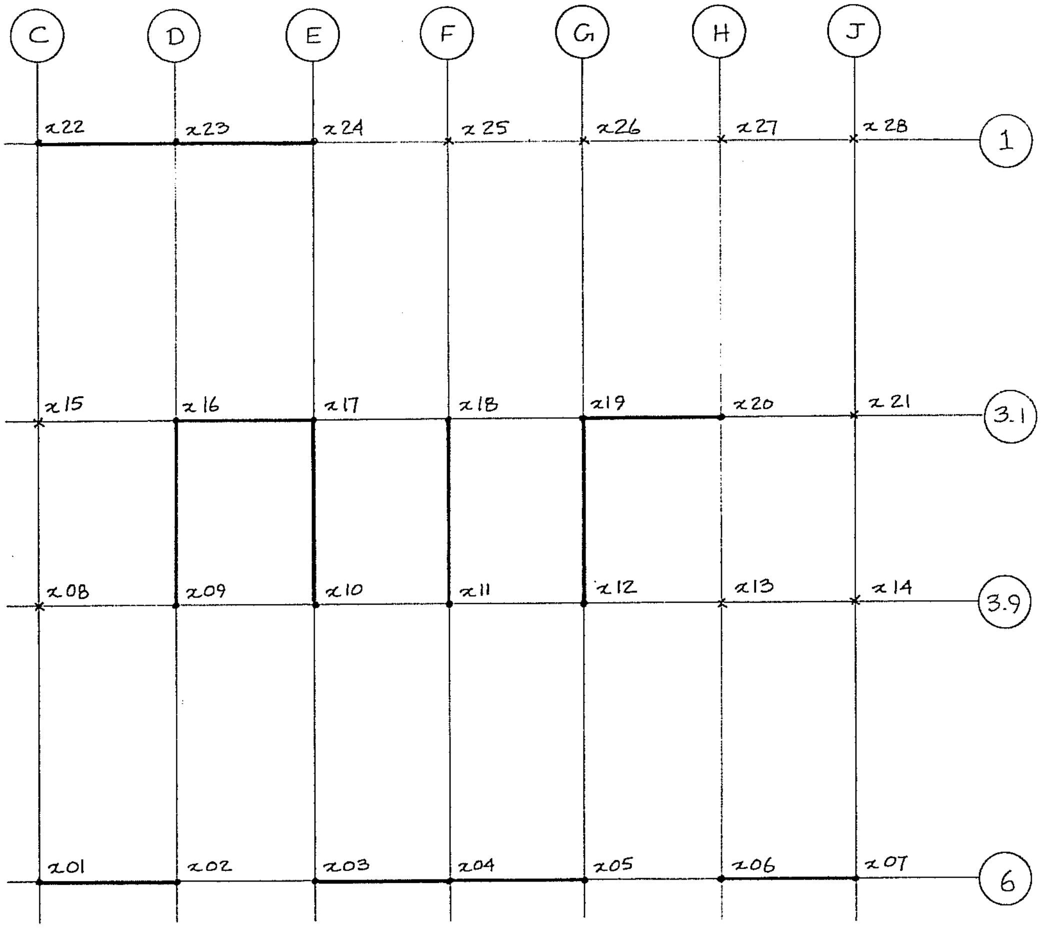
FRAME - G
1.0D+0.5L+EQ
SMCH Record

TRIAL-5



Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)

SOH & Associates
Structural Engineers
303 Second St., Suite 305 South Tower
San Francisco, CA 94107
Tel (415) 882-5533
Fax (415) 882-5445



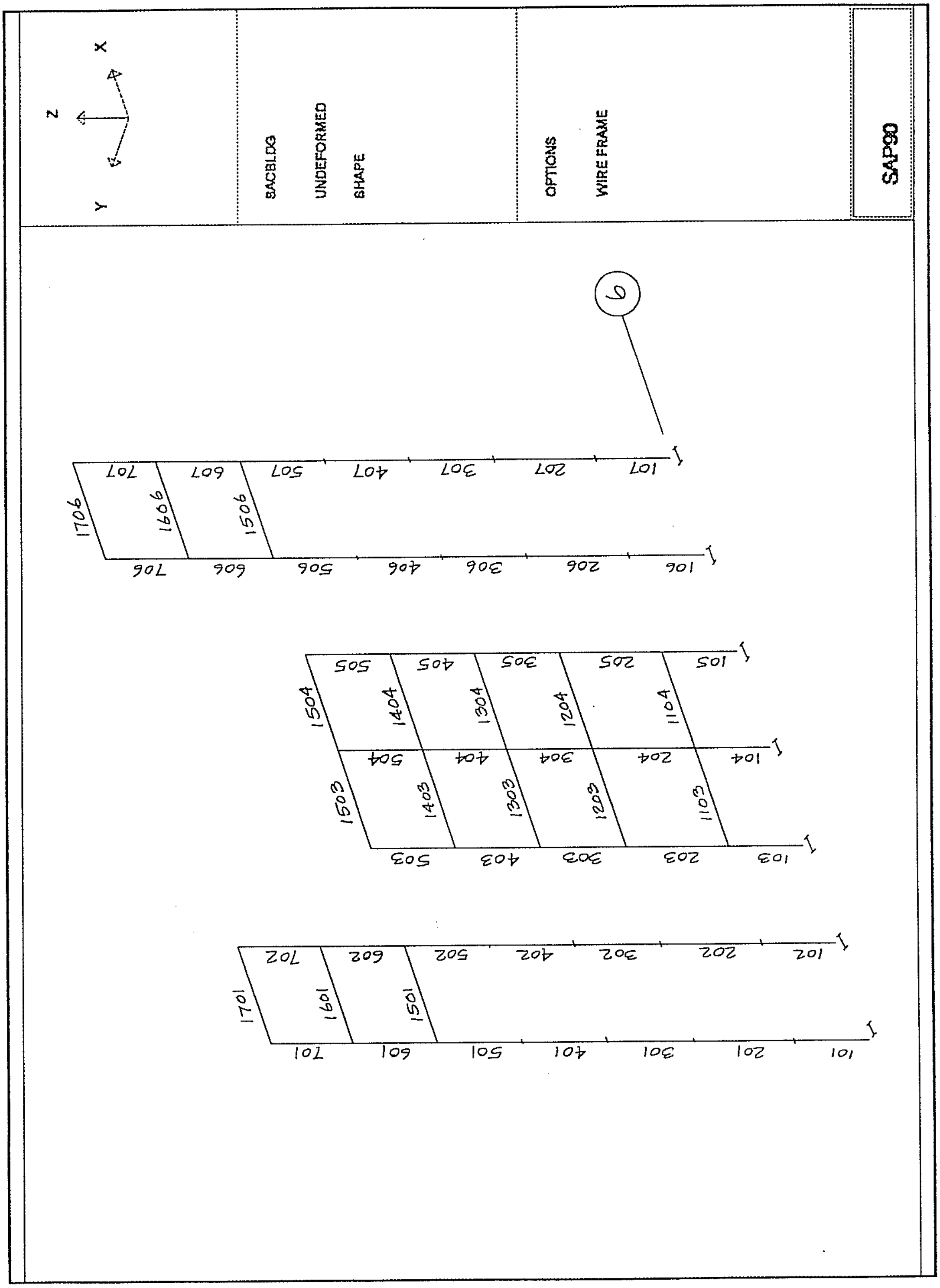
PLAN

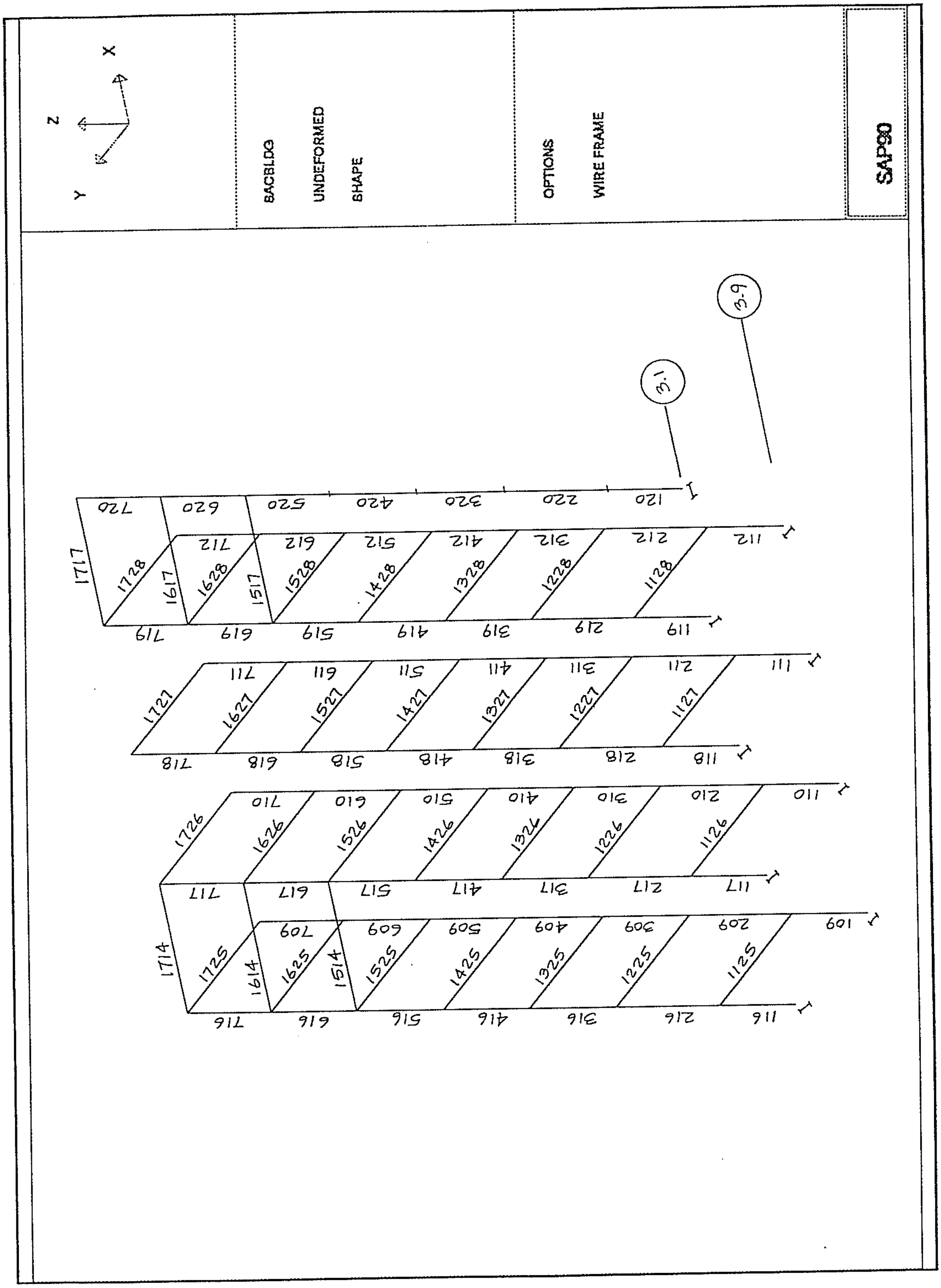
JOINT NUMBERS

- • Frame Joints
- x Extraneous Joints (restrained)

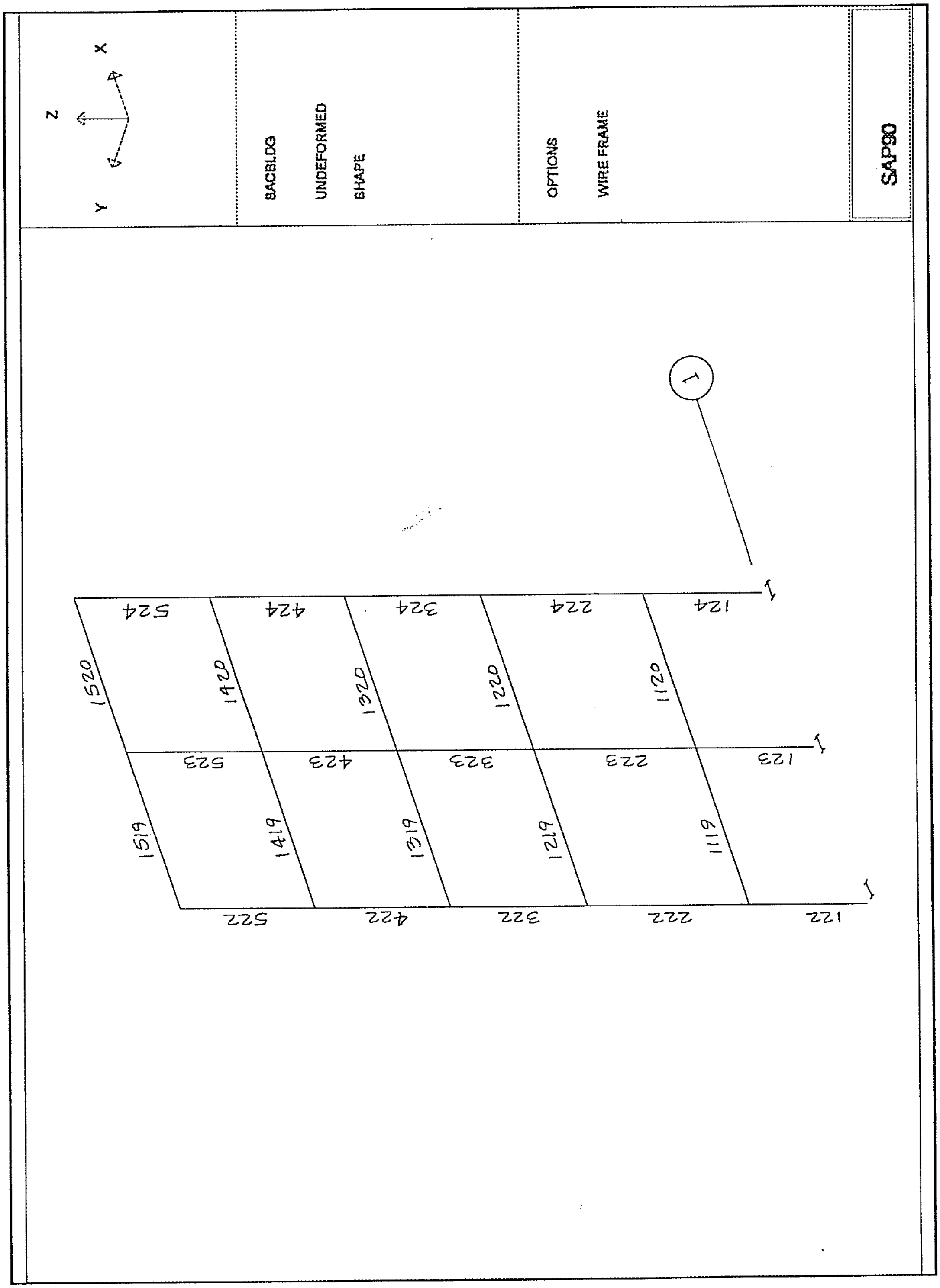
x ≡ 1 for basement to 8 for roof

Hence, joint 210 ≡ Joint @ E/3.9 on ground floor


SACBLDG
UNDEFORMED
SHAPE
OPTIONS
WIRE FRAME
SAP90



X
Y
Z
SACBLDG
UNDEFORMED
SHAPE
OPTIONS
WIRE FRAME
SAP90
3.1
3.9



Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)

MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	DEMAND/CAPACITY		
					D+L	D+L+EQ	D+L-EQ
1127	W33X118	415	47.3	19630	0.02	0.66	0.70
					0.02	0.66	0.71
1128	W33X118	415	47.3	19630	0.02	0.74	0.78
					0.01	0.75	0.77
1227	W24X146	418	47.3	19771	0.02	1.40	1.44
					0.02	1.40	1.44
1228	W33X118	415	47.3	19630	0.02	1.73	1.77
					0.01	1.74	1.76
1327	W24X146	418	47.3	19771	0.02	1.31	1.34
					0.02	1.30	1.35
1328	W33X118	415	47.3	19630	0.02	1.58	1.62
					0.01	1.59	1.61
1427	W24X117	327	47.3	15467	0.02	1.37	1.42
					0.03	1.37	1.43
1428	W30X99	312	47.3	14758	0.03	1.61	1.66
					0.02	1.62	1.65
1527	W24X117	327	47.3	15467	0.02	1.20	1.24
					0.03	1.19	1.25
1528	W30X99	312	47.3	14758	0.02	1.21	1.25
					0.02	1.21	1.24
1627	W24X55	134	50.5	6767	0.07	1.33	1.46
					0.08	1.32	1.47
1628	W24X55	134	50.5	6767	0.05	1.40	1.50
					0.06	1.39	1.51
1727	W24X55	134	50.5	6767	0.13	0.59	0.84
					0.13	0.58	0.85
1728	W27X84	244	47.3	11541	0.08	0.48	0.65
					0.09	0.48	0.65

TRIAL-1

MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	DEMAND/CAPACITY		
					D+L	D+L+EQ	D+L-EQ
1127	W33X118	415	47.3	19630	0.02	0.64	0.68
					0.02	0.64	0.68
1128	W33X118	415	47.3	19630	0.02	0.71	0.74
					0.01	0.71	0.74
1227	W24X146	418	47.3	19771	0.02	1.36	1.40
					0.02	1.35	1.40
1228	W33X118	415	47.3	19630	0.02	1.66	1.70
					0.01	1.67	1.69
1327	W24X146	418	47.3	19771	0.02	1.26	1.29
					0.02	1.25	1.30
1328	W33X118	415	47.3	19630	0.02	1.52	1.56
					0.01	1.53	1.55
1427	W24X117	327	47.3	15467	0.02	1.31	1.36
					0.03	1.31	1.37
1428	W30X99	312	47.3	14758	0.03	1.54	1.59
					0.02	1.55	1.58
1527	W24X117	327	47.3	15467	0.02	1.12	1.17
					0.03	1.12	1.17
1528	W30X99	312	47.3	14758	0.02	1.22	1.26
					0.02	1.22	1.26
1627	W24X55	134	50.5	6767	0.07	1.31	1.45
					0.08	1.30	1.45
1628	W24X55	134	50.5	6767	0.05	1.47	1.58
					0.06	1.46	1.59
1727	W24X55	134	50.5	6767	0.13	0.60	0.85
					0.13	0.59	0.86
1728	W27X84	244	47.3	11541	0.08	0.53	0.70
					0.09	0.53	0.70

TRIAL-2

MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	DEMAND/CAPACITY		
					D+L	D+L+EQ	D+L-EQ
1127	W33X118	415	47.3	19630	0.02	0.62	0.66
					0.02	0.62	0.66
1128	W33X118	415	47.3	19630	0.02	0.68	0.72
					0.01	0.68	0.71
1227	W24X146	418	47.3	19771	0.02	1.34	1.37
					0.02	1.33	1.38
1228	W33X118	415	47.3	19630	0.02	1.64	1.68
					0.01	1.65	1.67
1327	W24X146	418	47.3	19771	0.02	1.26	1.30
					0.02	1.26	1.30
1328	W33X118	415	47.3	19630	0.02	1.52	1.56
					0.01	1.53	1.55
1427	W24X117	327	47.3	15467	0.02	1.27	1.32
					0.03	1.26	1.32
1428	W30X99	312	47.3	14758	0.03	1.49	1.54
					0.02	1.50	1.54
1527	W24X117	327	47.3	15467	0.02	1.09	1.13
					0.03	1.08	1.14
1528	W30X99	312	47.3	14758	0.02	1.21	1.25
					0.02	1.22	1.25
1627	W24X55	134	50.5	6767	0.07	1.30	1.43
					0.08	1.29	1.44
1628	W24X55	134	50.5	6767	0.05	1.45	1.56
					0.06	1.44	1.57
1727	W24X55	134	50.5	6767	0.13	0.59	0.85
					0.13	0.59	0.85
1728	W27X84	244	47.3	11541	0.08	0.56	0.73
					0.09	0.56	0.73

TRIAL-3

MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	DEMAND/CAPACITY		
					D+L	D+L+EQ	D+L-EQ
1127	W33X118	415	47.3	19630	0.02	0.77	0.81
					0.02	0.77	0.81
1128	W33X118	415	47.3	19630	0.02	0.99	1.02
					0.01	0.99	1.02
1227	W24X146	418	47.3	19771	0.02	1.66	1.69
					0.02	1.65	1.69
1228	W33X118	415	47.3	19630	0.02	2.17	2.20
					0.01	2.18	2.19
1327	W24X146	418	47.3	19771	0.02	1.49	1.52
					0.02	1.49	1.53
1328	W33X118	415	47.3	19630	0.02	1.93	1.97
					0.01	1.94	1.96
1427	W24X117	327	47.3	15467	0.02	1.66	1.70
					0.03	1.65	1.71
1428	W30X99	312	47.3	14758	0.02	2.08	2.13
					0.02	2.09	2.12
1527	W24X117	327	47.3	15467	0.02	1.46	1.50
					0.02	1.46	1.50
1528	W30X99	312	47.3	14758	0.02	1.60	1.64
					0.01	1.60	1.63
1627	W24X55	134	50.5	6767	0.06	1.90	2.03
					0.07	1.89	2.04
1628	W24X55	134	50.5	6767	0.05	1.92	2.02
					0.06	1.91	2.03
1727	W24X55	134	50.5	6767	0.11	1.00	1.22
					0.12	0.99	1.23
1728	W27X84	244	47.3	11541	0.07	0.80	0.95
					0.08	0.80	0.95

TRIAL-4

MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	DEMAND/CAPACITY		
					D+L	D+L+EQ	D+L-EQ
1127	W33X118	415	47.3	19630	0.02	0.59	0.63
					0.02	0.59	0.63
1128	W33X118	415	47.3	19630	0.02	0.61	0.64
					0.01	0.61	0.63
1227	W24X146	418	47.3	19771	0.02	1.27	1.31
					0.02	1.27	1.31
1228	W33X118	415	47.3	19630	0.02	1.51	1.55
					0.01	1.52	1.54
1327	W24X146	418	47.3	19771	0.02	1.19	1.23
					0.02	1.19	1.23
1328	W33X118	415	47.3	19630	0.02	1.43	1.46
					0.01	1.44	1.46
1427	W24X117	327	47.3	15467	0.02	1.32	1.36
					0.03	1.31	1.37
1428	W30X99	312	47.3	14758	0.02	1.54	1.59
					0.02	1.55	1.58
1527	W24X117	327	47.3	15467	0.02	1.25	1.29
					0.02	1.25	1.30
1528	W30X99	312	47.3	14758	0.02	1.43	1.46
					0.01	1.43	1.46
1627	W24X55	134	50.5	6767	0.06	1.62	1.75
					0.07	1.61	1.76
1628	W24X55	134	50.5	6767	0.05	1.72	1.82
					0.06	1.71	1.83
1727	W24X55	134	50.5	6767	0.11	1.02	1.24
					0.12	1.01	1.25
1728	W27X84	244	47.3	11541	0.07	0.80	0.95
					0.08	0.80	0.95

TRIAL-5

FLOOR	MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	D/C
FIRST	1103	W33x118	415	47.3	19630	1.91 1.76
	1104	W30x99	312	47.3	14758	1.91 2.07
	1119	W30x99	312	47.3	14758	1.99 1.71
	1120	W30x99	312	47.3	14758	1.71 1.99
	1125	W33x118	415	47.3	19630	1.23 1.23
	1126	W33x118	415	47.3	19630	0.93 0.93
	1127	W33x118	415	47.3	19630	0.79 0.79
	1128	W33x118	415	47.3	19630	1.00 1.00
SECOND	1203	W30x99	312	47.3	14758	2.64 2.81
	1204	W30x99	312	47.3	14758	2.77 2.60
	1219	W30x99	312	47.3	14758	2.76 2.41
	1220	W30x99	312	47.3	14758	2.41 2.76
	1225	W33x118	415	47.3	19630	1.93 1.93
	1226	W33x118	415	47.3	19630	1.78 1.78
	1227	W24x146	418	47.3	19771	1.67 1.67
	1228	W33x118	415	47.3	19630	2.19 2.19
THIRD	1303	W30x99	312	47.3	14758	2.35 2.54
	1304	W30x99	312	47.3	14758	2.56 2.36
	1319	W30x99	312	47.3	14758	2.39 2.12
	1320	W30x99	312	47.3	14758	2.12 2.39
	1325	W33x118	415	47.3	19630	1.73 1.73

FLOOR	MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	D/C
	1326	W33x118	415	47.3	19630	1.66
						1.66
	1327	W24x146	418	47.3	19771	1.51
						1.51
	1328	W33x118	415	47.3	19630	1.95
						1.95
	1403	W27x94	278	47.3	13149	1.98
FOURTH						2.11
	1404	W27x94	278	47.3	13149	2.11
						1.98
	1419	W27x94	278	47.3	13149	1.95
						1.86
	1420	W27x94	278	47.3	13149	1.86
						1.95
	1425	W30x99	312	47.3	14758	1.88
						1.88
	1426	W30x99	312	47.3	14758	1.81
						1.81
	1427	W24x117	327	47.3	15467	1.68
						1.68
	1428	W30x99	312	47.3	14758	2.11
						2.11
	1501	W24x62	153	50.5	7727	1.41
FIFTH						1.41
	1503	W27x94	278	47.3	13149	1.06
						1.13
	1504	W27x94	278	47.3	13149	1.13
						1.06
	1506	W24x62	153	50.5	7727	1.41
						1.41
	1514	W24x76	200	47.3	9460	1.12
						1.12
	1517	W24x76	200	47.3	9460	2.11
						1.74
	1519	W27x94	278	47.3	13149	1.08
						1.19
	1520	W27x94	278	47.3	13149	1.19
						1.08
	1525	W30x99	312	47.3	14758	2.03
						2.03
	1526	W30x99	312	47.3	14758	1.80
						1.80

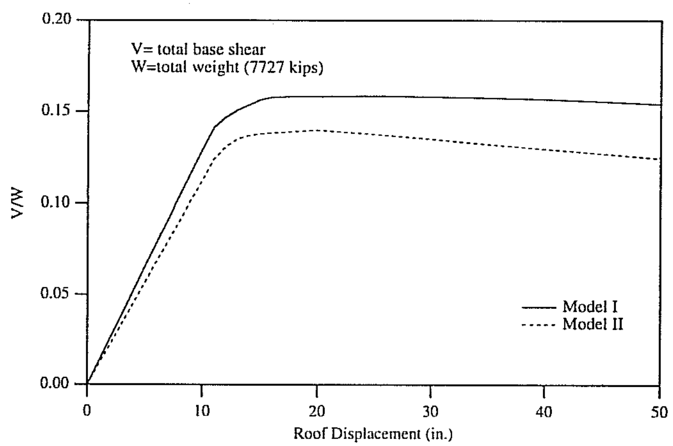
Trial-4 with EQ only - all beams

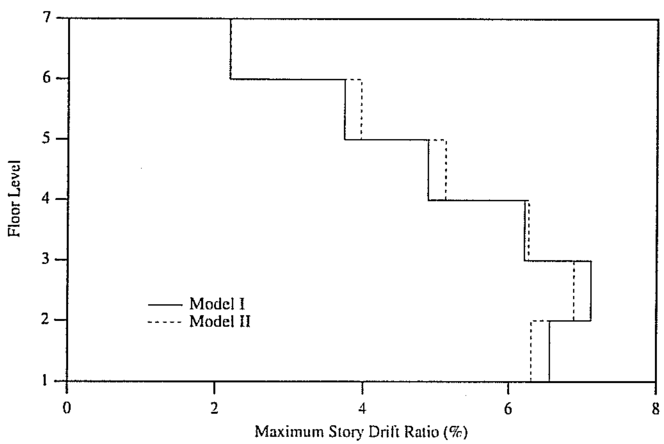
FLOOR	MEMBER	SECTION	Z (in^3)	Fy (ksi)	Mp (k-in)	D/C
	1527	W24x117	327	47.3	15467	1.48
						1.48
	1528	W30x99	312	47.3	14758	1.62
						1.62
SIXTH	1601	W24x62	153	50.5	7727	1.70
						1.70
	1606	W24x62	153	50.5	7727	1.70
						1.70
	1614	W24x76	200	47.3	9460	1.53
						1.53
	1617	W24x76	200	47.3	9460	3.51
						3.20
	1625	W24x55	134	50.5	6767	3.21
						3.21
	1626	W24x55	134	50.5	6767	2.28
						2.28
	1627	W24x55	134	50.5	6767	1.97
						1.97
	1628	W24x55	134	50.5	6767	1.97
						1.97
ROOF	1701	W24x62	153	50.5	7727	0.99
						0.99
	1706	W24x62	153	50.5	7727	0.99
						0.99
	1714	W24x76	200	47.3	9460	0.76
						0.76
	1717	W24x76	200	47.3	9460	1.99
						1.99
	1725	W24x55	134	50.5	6767	1.88
						1.88
	1726	W24x55	134	50.5	6767	1.26
						1.26
	1727	W24x55	134	50.5	6767	1.11
						1.11
	1728	W27x84	244	47.3	11541	0.88
						0.88

APPENDIX C - TWO-DIMENSIONAL INELASTIC ANALYSIS

This appendix contains additional data generated in the 2-D inelastic analysis. The model assumptions and loading cases considered for the 2-D inelastic analysis are described in Sections 6.1 and 6.2.

Static Pushover Analysis





	Model I	Model II
T_1	1.970	2.250
T_2	0.715	0.836
T_3	0.394	0.451
T_4	0.259	0.288
ϖ_1^2	10.175	7.794
ϖ_2^2	77.236	56.424
ϖ_3^2	254.927	194.507
ϖ_4^2	586.980	474.759
β_m	0.11270	0.09899
β_{k_o}	0.00146	0.00127

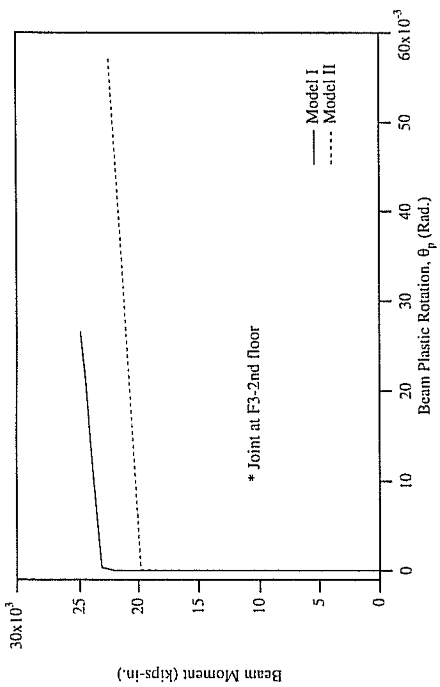
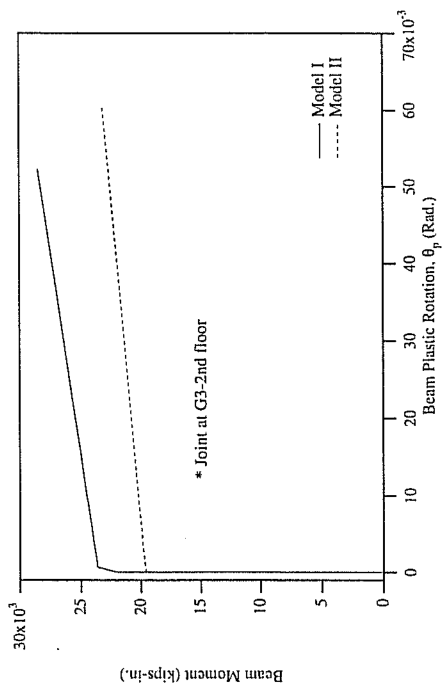
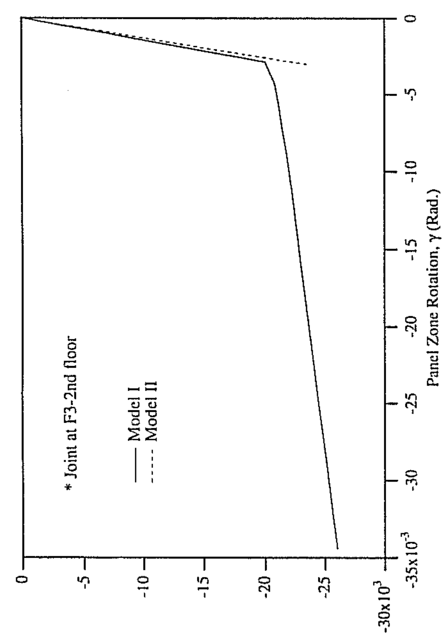
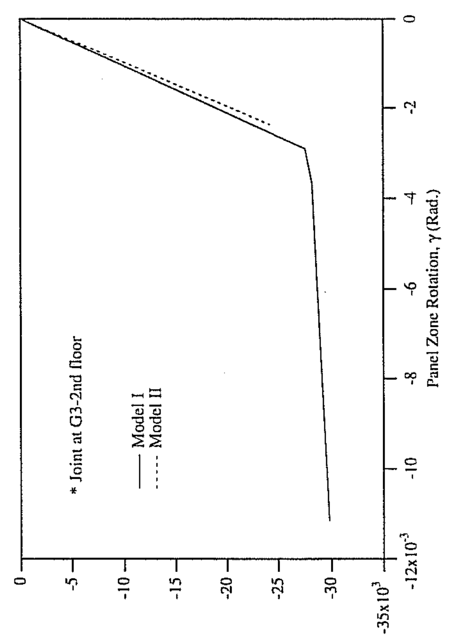
Maximum Story Drift Ratio

Staic Pushover Analysis : Model I

Ground Level-2nd Floor	0.000000	6.55956
2nd Floor-3rd Floor	0.000000	7.12064
3rd Floor-4th Floor	0.000000	6.21419
4th Floor-5th Floor	0.000000	4.89109
5th Floor-6th Floor	0.000000	3.73365
6th Floor-Roof	0.000000	2.18928

Staic Pushover Analysis : Model II

Ground Level-2nd Floor	0.000000	6.30791
2nd Floor-3rd Floor	0.000000	6.89032
3rd Floor-4th Floor	0.000000	6.26679
4th Floor-5th Floor	0.000000	5.13244
5th Floor-6th Floor	0.000000	3.96218
6th Floor-Roof	0.000000	2.19877


* Joint at F3-2nd floor
Model I
Model II
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)

* Joint at G3-2nd floor
Model I
Model II
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)

* Joint at F3-2nd floor
Model I
Model II
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)

* Joint at G3-2nd floor
Model I
Model II
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)

Static Pushover Analysis : Model I

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05768	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05780	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05790	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05761	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05799	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05666	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05868	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05725	0.00000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.05215	0.00000	0.00000	-0.00799	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.02917	0.02756	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.05217	0.00000	0.00000	-0.00799	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.02917	0.02756	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.02656	0.00000	0.00000	-0.03065	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.01179	0.04396	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.05220	0.00000	0.00000	-0.00800	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.02916	0.02755	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04955	0.00000	0.00000	-0.00298	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.02705	0.02234	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.04955	0.00000	0.00000	-0.00298	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.02705	0.02235	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.02321	0.00000	0.00000	-0.02620	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.00960	0.03883	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.04955	0.00000	0.00000	-0.00298	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.02706	0.02236	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04008	0.00000	0.00000	-0.00056	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.02263	0.01553	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.04009	0.00000	0.00000	-0.00056	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.02264	0.01553	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.04039	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.03171	0.00318	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.04011	0.00000	0.00000	-0.00058	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.02266	0.01556	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02661	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01633	0.00844	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.02662	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01633	0.00845	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.02631	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.02240	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.02656	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01626	0.00837	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 6th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01409	0.00000	0.00000	0.00000	0.00075	0.00000	0.00000	0.00000
D4	0.00000	-0.01073	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01410	0.00000	0.00000	0.00000	0.00075	0.00000	0.00000	0.00000
E4	0.00000	-0.01074	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01424	0.00000	0.00000	0.00000	0.00083	0.00000	0.00000	0.00000
F4	0.00000	-0.01099	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01495	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01142	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00056	0.00000	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Static Pushover Analysis : Model II

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05012	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05198	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05041	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05178	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05142	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05160	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05141	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.05141	0.00000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.06050	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.06052	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.05754	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.05754	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.06058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.05584	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.05581	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.05584	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.05582	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.05278	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.05278	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.05583	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.05583	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.04648	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.04649	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.04649	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.04650	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.04549	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.04549	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.04656	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.04656	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

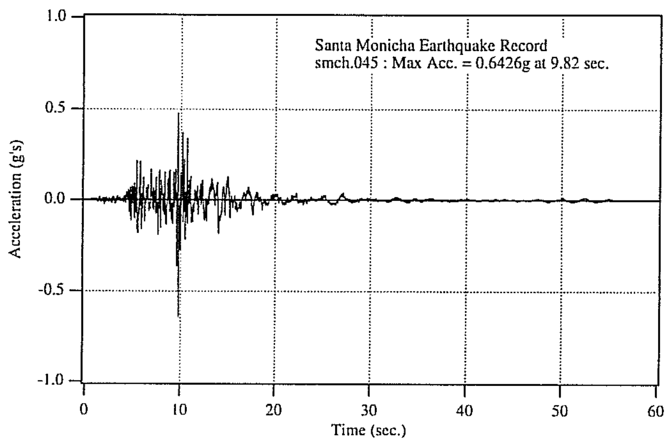
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.03314	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.03313	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.03314	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.03314	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.03212	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.03212	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.03296	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.03296	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

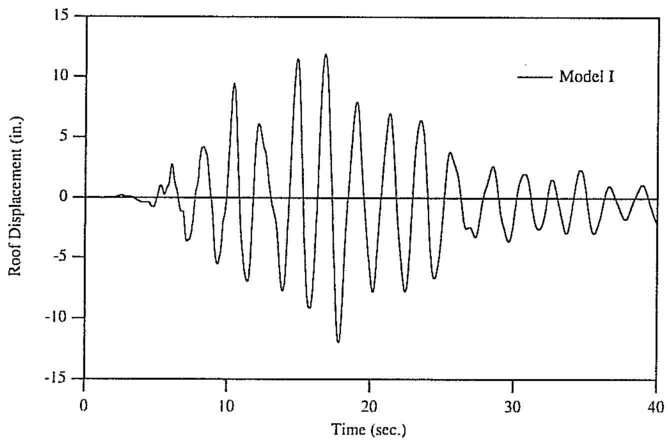
Floor : 6th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	-0.01728	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01729	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	-0.01729	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01729	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	-0.01745	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.01745	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	-0.01812	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01812	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

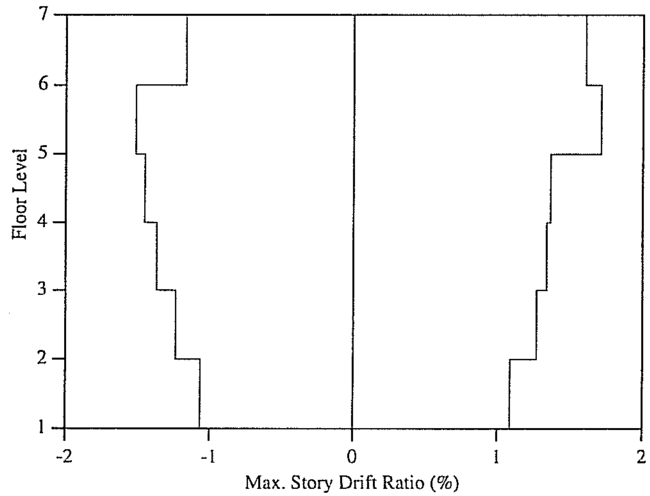
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000





ANALYSIS : SMCH.045

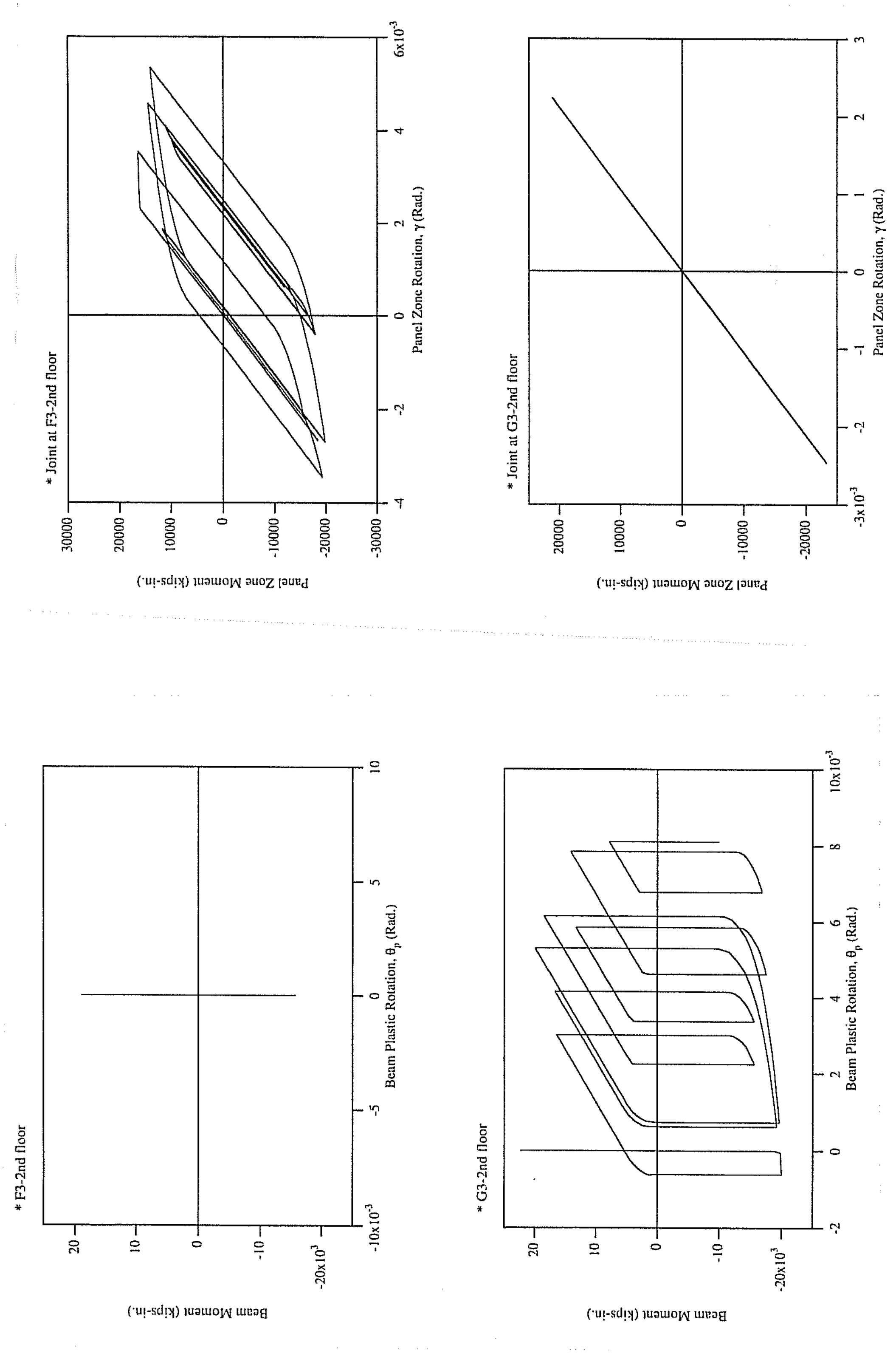
* Floor level 1 is Ground level
Floor level 7 is Roof



Maximum Story Drift Ratio

Analysis of Model I : SMCH.O45

Ground Level-2nd Floor	-0.01065	0.01087
2nd Floor-3rd Floor	-0.01237	0.01268
3rd Floor-4th Floor	-0.01370	0.01337
4th Floor-5th Floor	-0.01454	0.01363
5th Floor-6th Floor	-0.01516	0.01711
6th Floor-Roof	-0.01169	0.01606



* Joint at F3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
* Joint at G3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
* F3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)
* G3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)

Analysis of Model I : SMCH.O45

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00824	-0.00051	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00097	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00821	-0.00054	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00096	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00332	-0.00066	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00202	-0.00106	0.00000	0.00000	0.00000	0.00000
G3	0.00811	-0.00063	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00095	-0.00226	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00096	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00783	-0.00066	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00096	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00783	-0.00067	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00044	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00104	-0.00128	0.00000	0.00000	0.00000	0.00000
G3	0.00096	-0.00224	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00783	-0.00069	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00144	-0.00323	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00813	-0.00107	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00145	-0.00323	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00814	-0.00107	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00639	-0.00011	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00088	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00150	-0.00325	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00813	-0.00110	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

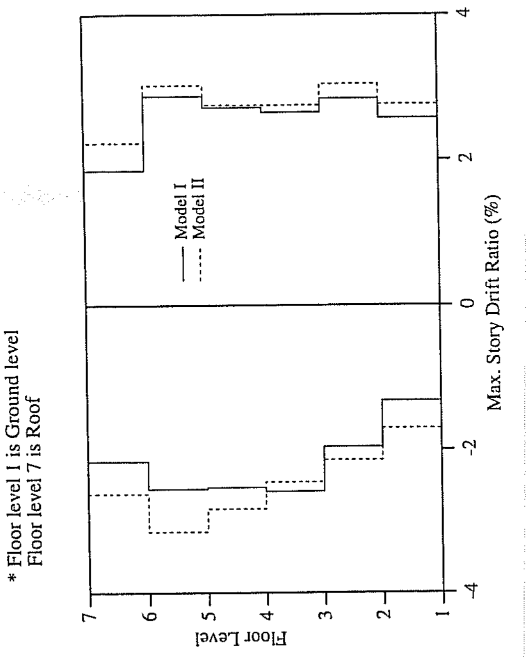
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00263	-0.00171	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00861	-0.00131	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00263	-0.00172	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00861	-0.00131	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00090	-0.00207	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00250	-0.00170	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00868	-0.00119	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 6th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00720	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00996	-0.00360	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00719	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00986	-0.00360	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00742	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00998	-0.00370	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00728	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00944	-0.00425	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00018	-0.00017	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000



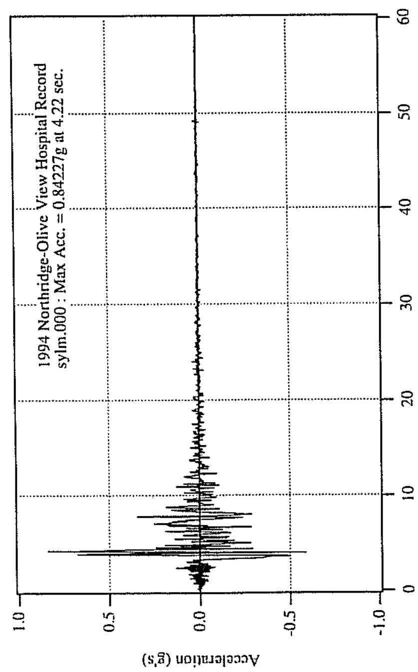
Maximum Story Drift Ratio

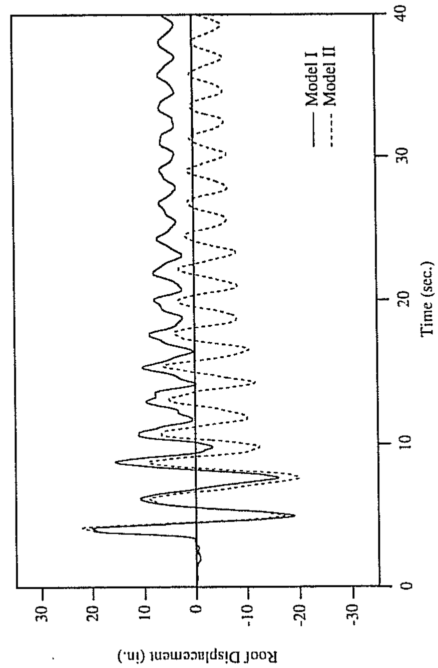
Analysis of Model I : SYLM.000

Ground Level-2nd Floor	-0.01317	0.02575
2nd Floor-3rd Floor	-0.01958	0.02841
3rd Floor-4th Floor	-0.02585	0.02655
4th Floor-5th Floor	-0.02527	0.02720
5th Floor-6th Floor	-0.02549	0.02877
6th Floor-Roof	-0.02159	0.01852

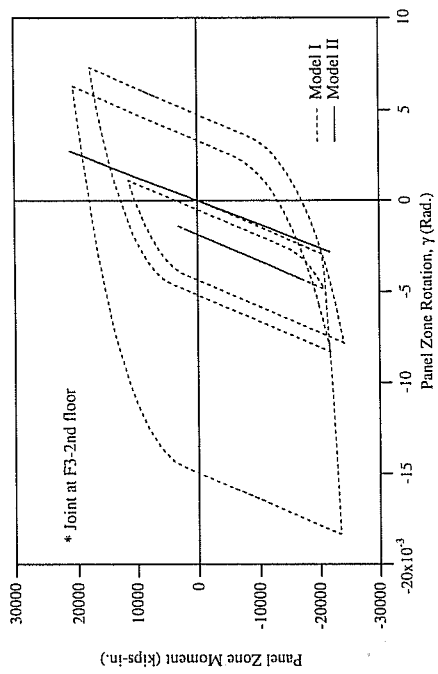
Analysis of Model II : SYLM.000

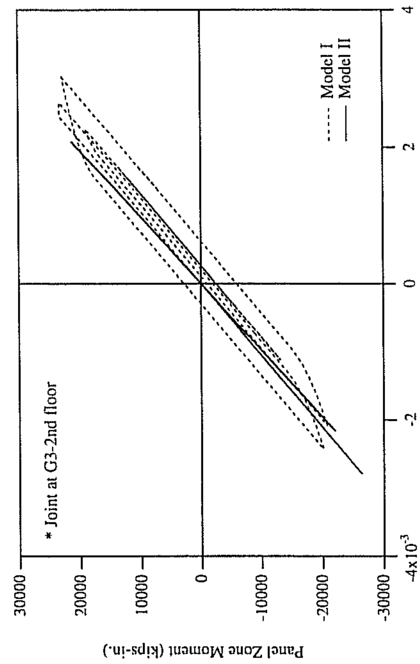
Ground Level-2nd Floor	-0.01699	0.02764
2nd Floor-3rd Floor	-0.02139	0.03045
3rd Floor-4th Floor	-0.02450	0.02754
4th Floor-5th Floor	-0.02828	0.02755
5th Floor-6th Floor	-0.03148	0.03027
6th Floor-Roof	-0.02621	0.02230

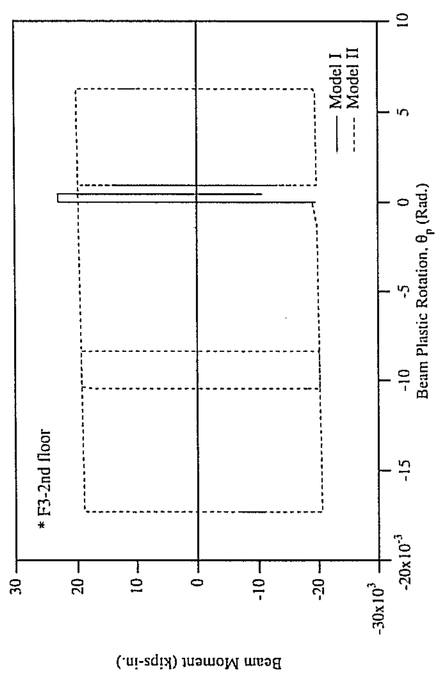


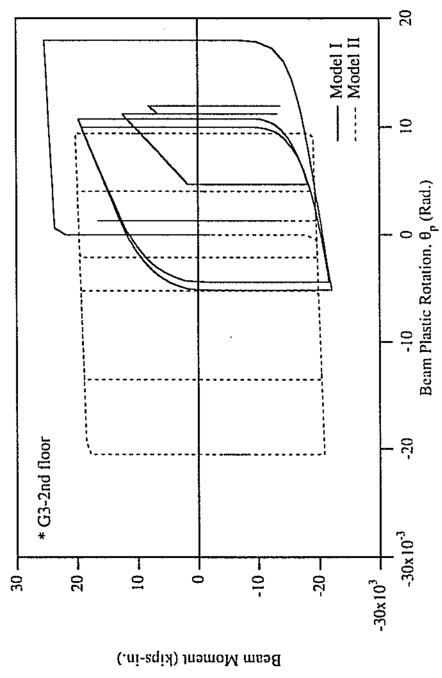


Analysis of a Six Story Steel Moment Frame Building in Santa Monica (SAC Building Site 7)


* Joint at F3-2nd floor
Model I
Model II
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
30000
20000
10000
0
-10000
-20000
-30000
-20x10^{-3}
-15
-10
-5
0
5
10


* Joint at G3-2nd floor
Model I
Model II
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
30000
20000
10000
0
-10000
-20000
-30000
-4x10^{-3}
-2
0
2
4


* F3-2nd floor
Model I
Model II
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)
30
20
10
0
-10
-20
-30x10^{3}
-20x10^{-3}
-15
-10
-5
0
5
10


* G3-2nd floor
Model I
Model II
Beam Moment (kips-in.)
Beam Plastic Rotation, θ_p (Rad.)
30
20
10
0
-10
-20
-30x10^{3}
-30x10^{-3}
-20
-10
0
10
20

Floor : Ground

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01215	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01205	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01235	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01187	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01261	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01110	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01303	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01149	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01153	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01338	-0.00068
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01182	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01319	-0.00049
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01277	-0.00017
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01295	-0.00034
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01275	-0.00007
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01275	-0.00007

Floor : 2nd

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01789	-0.00505	0.00051	-0.00038	0.00000	0.00000	0.00000	0.00000
D4	0.00768	-0.01172	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01791	-0.00507	0.00054	-0.00037	0.00000	0.00000	0.00000	0.00000
E4	0.00767	-0.01172	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00045	0.00000	0.00474	-0.01498	0.00000	0.00000	0.00000	0.00000
F4	0.00736	0.00000	0.01677	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01796	-0.00515	0.00062	-0.00030	0.00000	0.00000	0.00000	0.00000
G4	0.00766	-0.01173	0.00649	0.00000	0.00000	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00932	-0.02041	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00939	-0.02046	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00934	-0.02042	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00939	-0.02046	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00627	-0.01731	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00628	-0.01732	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00940	-0.02047	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00940	-0.02047	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01541	-0.00838	0.00210	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01124	-0.01150	0.00316	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01541	-0.00838	0.00209	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01127	-0.01151	0.00317	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00813	-0.00145	0.00804	-0.01227	0.00000	0.00000	0.00000	0.00000
F4	0.01090	0.00000	0.01513	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01542	-0.00838	0.00208	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01129	-0.01151	0.00317	0.00000	0.00000	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01128	-0.01720	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01128	-0.01722	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01128	-0.01721	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01128	-0.01722	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00829	-0.01418	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00829	-0.01418	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01129	-0.01722	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01129	-0.01722	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01462	-0.00991	0.00358	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01343	-0.01138	0.00230	-0.00083	0.00000	0.00000	0.00000	0.00000
E3	0.01463	-0.00992	0.00359	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01343	-0.01138	0.00230	-0.00082	0.00000	0.00000	0.00000	0.00000
F3	0.01351	-0.01148	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01346	-0.01105	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01466	-0.00995	0.00365	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01330	-0.01140	0.00235	-0.00087	0.00000	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01561	-0.01476	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01564	-0.01475	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01562	-0.01476	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01564	-0.01475	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01440	-0.01359	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01440	-0.01359	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01571	-0.01480	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01571	-0.01480	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01570	-0.00819	0.00104	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00995	-0.01206	0.00259	-0.00019	0.00000	0.00000	0.00000	0.00000
E3	0.01571	-0.00819	0.00104	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00996	-0.01206	0.00259	-0.00019	0.00000	0.00000	0.00000	0.00000
F3	0.01476	-0.00784	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00943	-0.01238	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01568	-0.00816	0.00101	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00995	-0.01204	0.00256	-0.00020	0.00000	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01698	-0.01740	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01697	-0.01741	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01698	-0.01741	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01697	-0.01741	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01581	-0.01619	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01581	-0.01619	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01684	-0.01737	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01684	-0.01737	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 6th

Analysis of Model I : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00832	-0.01244	0.00000	0.00000	0.00713	0.00000	0.00000	0.00000
D4	0.01575	-0.00563	0.00000	0.00000	0.00520	0.00000	0.00000	0.00000
E3	0.00831	-0.01245	0.00000	0.00000	0.00713	0.00000	0.00000	0.00000
E4	0.01576	-0.00563	0.00000	0.00000	0.00520	0.00000	0.00000	0.00000
F3	0.00832	-0.01254	0.00000	0.00000	0.00714	0.00000	0.00000	0.00000
F4	0.01585	-0.00563	0.00000	0.00000	0.00527	0.00000	0.00000	0.00000
G3	0.01029	-0.01302	0.00000	0.00000	0.00644	0.00000	0.00000	0.00000
G4	0.01559	-0.00649	0.00000	0.00000	0.00442	0.00000	0.00000	0.00000

Analysis of Model II : SYLM.000

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01835	-0.01016	0.00000	0.00000	0.00341	0.00000	0.00000	0.00000
D4	0.01836	-0.01016	0.00000	0.00000	0.00341	0.00000	0.00000	0.00000
E3	0.01836	-0.01017	0.00000	0.00000	0.00341	0.00000	0.00000	0.00000
E4	0.01837	-0.01017	0.00000	0.00000	0.00341	0.00000	0.00000	0.00000
F3	0.01843	-0.01011	0.00000	0.00000	0.00349	0.00000	0.00000	0.00000
F4	0.01843	-0.01011	0.00000	0.00000	0.00349	0.00000	0.00000	0.00000
G3	0.01852	-0.01133	0.00000	0.00000	0.00252	0.00000	0.00000	0.00000
G4	0.01852	-0.01133	0.00000	0.00000	0.00252	0.00000	0.00000	0.00000

Floor : Roof

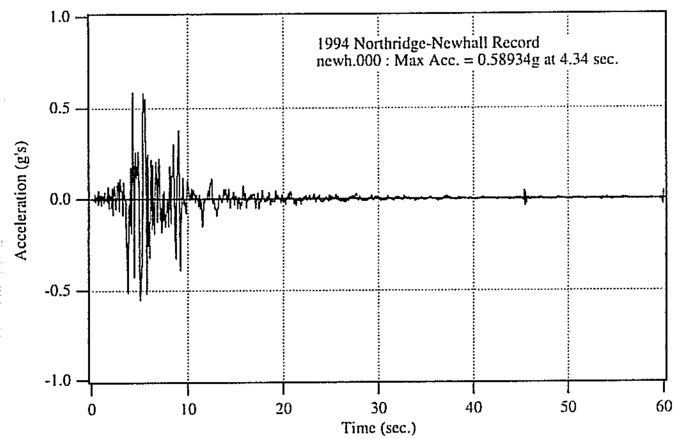
Analysis of Model I : SYLM.000

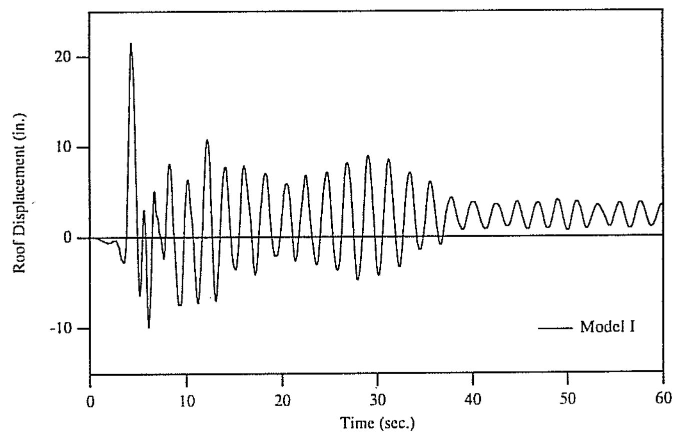
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00176	-0.00114	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00113	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00177	-0.00114	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00114	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00170	-0.00120	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00119	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00508	-0.00118	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00093	-0.00516	0.00000	0.00000

Analysis of Model II : SYLM.000

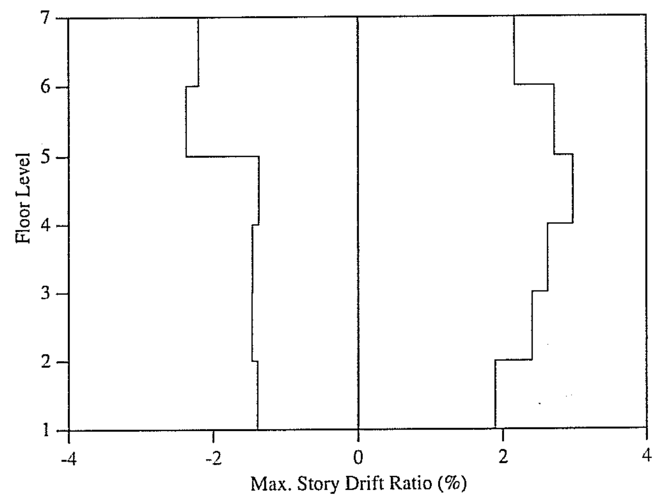
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00478	-0.00268	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00478	-0.00268	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00479	-0.00268	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00479	-0.00268	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00472	-0.00258	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00472	-0.00258	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00516	-0.00800	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00516	-0.00800	0.00000	0.00000

ANALYSIS : NEWH.000





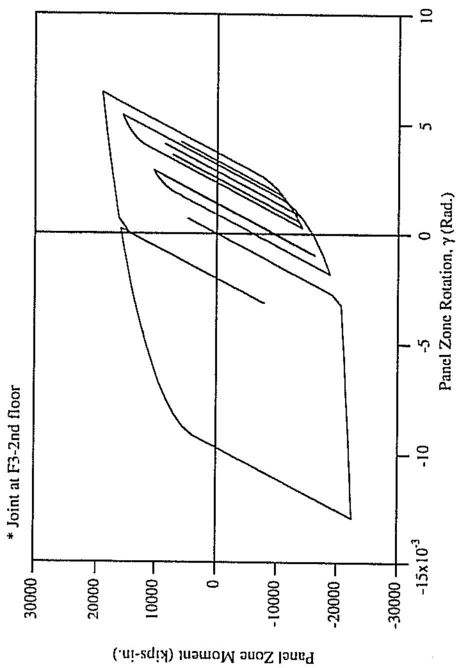
* Floor level 1 is Ground level
Floor level 7 is Roof

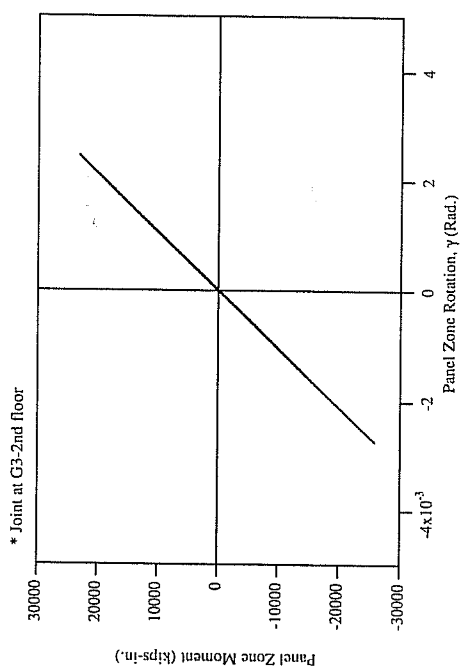


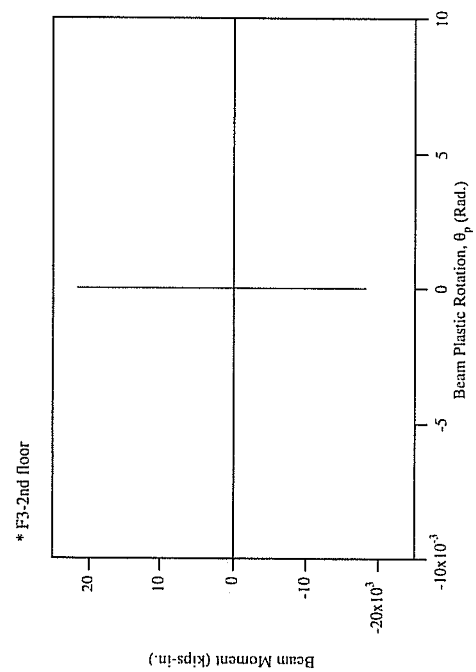
Maximum Story Drift Ratio

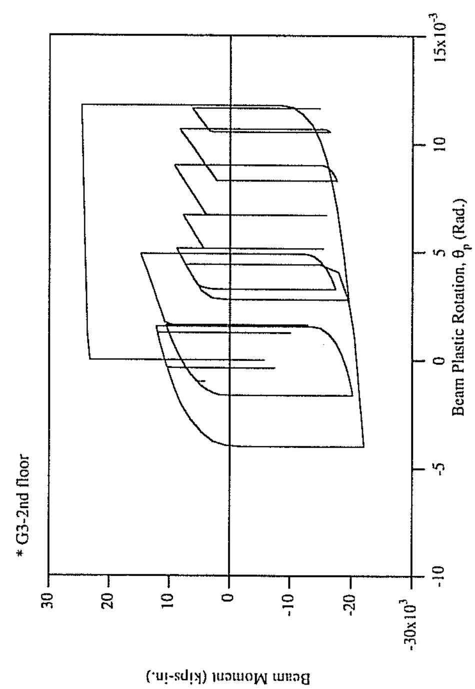
Analysis of Model I : NEWH.000

Ground Level-2nd Floor	-0.01391	0.01894
2nd Floor-3rd Floor	-0.01467	0.02406
3rd Floor-4th Floor	-0.01463	0.02625
4th Floor-5th Floor	-0.01377	0.02977
5th Floor-6th Floor	-0.02374	0.02716
6th Floor-Roof	-0.02197	0.02166


* Joint at F3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)


* Joint at G3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)


* F3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)


* G3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)

Analysis of Model I : NEWH.000

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00449	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00439	-0.00093
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00464	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00422	-0.00097
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00259	-0.00199
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00347	-0.00134
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00532	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00393	-0.00102

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01320	-0.00384	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00703	-0.00903	0.00148	-0.00053	0.00000	0.00000	0.00000	0.00000
E3	0.01173	-0.00386	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00702	-0.00903	0.00148	-0.00053	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00368	-0.00970	0.00000	0.00000	0.00000	0.00000
F4	0.00254	0.00000	0.01100	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01179	-0.00399	0.00002	-0.00001	0.00000	0.00000	0.00000	0.00000
G4	0.00697	-0.00903	0.00149	-0.00052	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01233	-0.00219	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00330	-0.00989	0.00031	-0.00124	0.00000	0.00000	0.00000	0.00000
E3	0.01232	-0.00219	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00331	-0.00989	0.00032	-0.00123	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00266	-0.00903	0.00000	0.00000	0.00000	0.00000
F4	0.00056	0.00000	0.01067	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01224	-0.00217	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00264	-0.00990	0.00033	-0.00122	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01743	-0.00200	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00352	-0.01270	0.00297	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01744	-0.00199	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00352	-0.01270	0.00298	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01648	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00595	-0.01356	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01882	-0.00143	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00355	-0.01273	0.00301	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01536	-0.00564	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00872	-0.01170	0.00201	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01537	-0.00564	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00872	-0.01171	0.00202	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01453	-0.00359	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00493	-0.01217	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01530	-0.00548	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00877	-0.01165	0.00193	0.00000	0.00000	0.00000	0.00000	0.00000

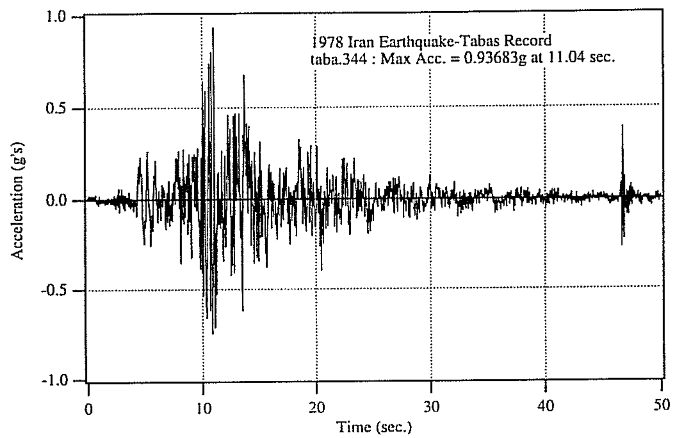
Floor : 6th

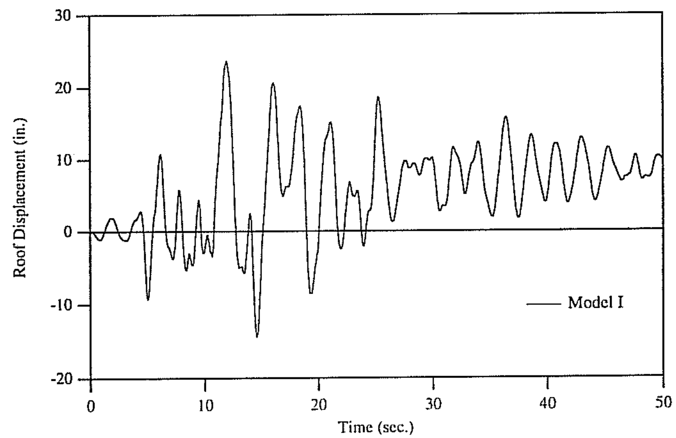
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01161	-0.01230	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01305	-0.00993	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01162	-0.01229	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01304	-0.00993	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01166	-0.01217	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01287	-0.00990	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01145	-0.01346	0.00006	-0.00061	0.00000	0.00000	0.00000	0.00000
G4	0.01316	-0.01068	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

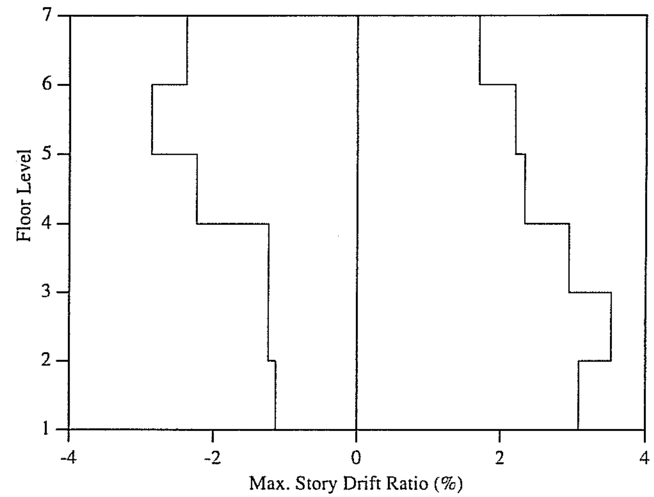
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00476	-0.00023	0.00000	0.00000	0.00511	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00296	-0.00206	0.00000	0.00000
E3	0.00476	-0.00022	0.00000	0.00000	0.00511	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00297	-0.00205	0.00000	0.00000
F3	0.00486	-0.00013	0.00000	0.00000	0.00517	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00305	-0.00196	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00607	-0.00300	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00343	-0.00538	0.00000	0.00000

ANALYSIS : TABA.344





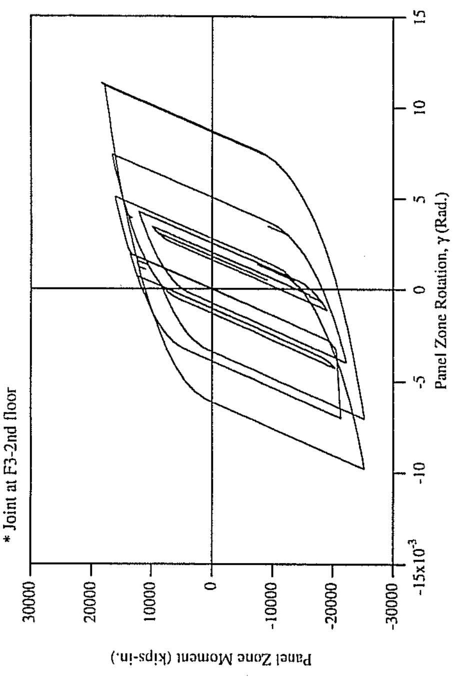
* Floor level 1 is Ground level
Floor level 7 is Roof

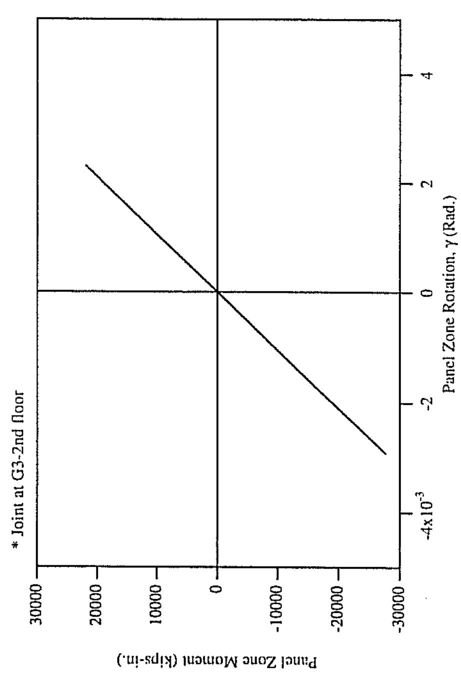


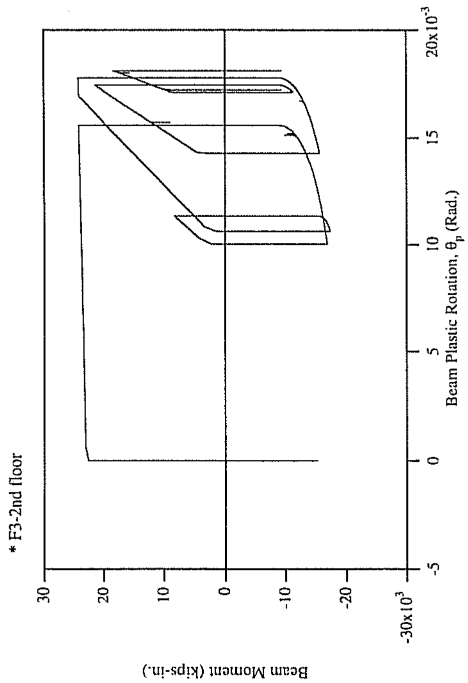
Maximum Story Drift Ratio

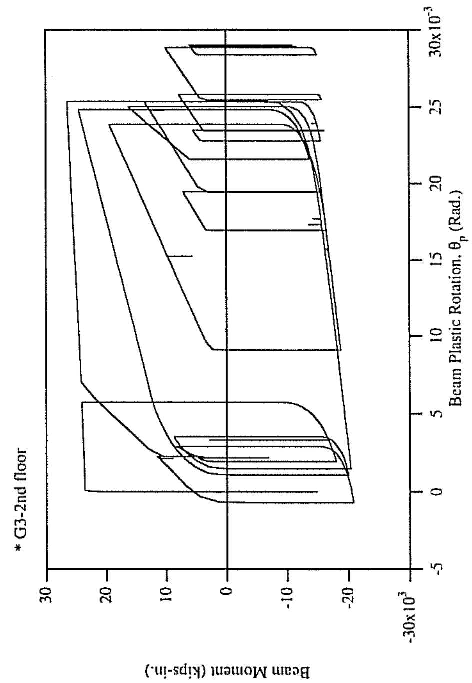
Analysis of Model I : 1978 Iran Earthquake: Taba.344

Ground Level-2nd Floor	-0.01134	0.03088
2nd Floor-3rd Floor	-0.01237	0.03533
3rd Floor-4th Floor	-0.01238	0.02953
4th Floor-5th Floor	-0.02232	0.02336
5th Floor-6th Floor	-0.02861	0.02205
6th Floor-Roof	-0.02375	0.01695


* Joint at F3-2nd floor
Panel Zone Moment (Kips-in.)
Panel Zone Rotation, γ (Rad.)
30000
20000
10000
0
-10000
-20000
-30000
-15x10^-3
-10
-5
0
5
10
15


* Joint at G3-2nd floor
Panel Zone Moment (Kips-in.)
Panel Zone Rotation, γ (Rad.)
30000
20000
10000
0
-10000
-20000
-30000
-4x10^-3
-2
0
2
4


* F3-2nd floor
Beam Moment (Kips-in.)
Beam Plastic Rotation, θp (Rad.)
30
20
10
0
-10
-20
-30x10^3
-5
0
5
10
15
20x10^-3


* G3-2nd floor
Beam Moment (Kips-in.)
Beam Plastic Rotation, θp (Rad.)
30
20
10
0
-10
-20
-30x10^3
-5
0
5
10
15
20
25
30x10^-3

Analysis of Model I : 1978 Iran Earthquake: Taba.344

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01670	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01660	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01692	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01640	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01746	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01587	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01766	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01597	0.00000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02799	-0.00058	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00614	-0.01323	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.02772	-0.00061	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00615	-0.01323	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01814	0.00000	0.00869	-0.00615	0.00000	0.00000	0.00000	0.00000
F4	0.01261	-0.00383	0.02371	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.02904	-0.00071	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00614	-0.01324	0.01265	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02202	-0.00168	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00354	-0.01154	0.00790	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.02202	-0.00169	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00354	-0.01155	0.00790	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00288	-0.00048	0.00000	-0.01636	0.00000	0.00000	0.00000	0.00000
F4	0.00580	-0.00475	0.01455	-0.00091	0.00000	0.00000	0.00000	0.00000
G3	0.02202	-0.00170	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00352	-0.01157	0.00791	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01667	-0.00285	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00443	-0.01122	0.00347	-0.00072	0.00000	0.00000	0.00000	0.00000
E3	0.01685	-0.00286	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00443	-0.01122	0.00346	-0.00071	0.00000	0.00000	0.00000	0.00000
F3	0.01366	-0.00351	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00379	-0.01090	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01669	-0.00290	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00449	-0.01124	0.00349	-0.00070	0.00000	0.00000	0.00000	0.00000

Floor : 5th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01808	-0.00308	0.00307	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01270	-0.00842	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01807	-0.00309	0.00307	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01272	-0.00842	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01052	-0.00839	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01501	-0.00413	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01802	-0.00296	0.00301	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01263	-0.00836	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

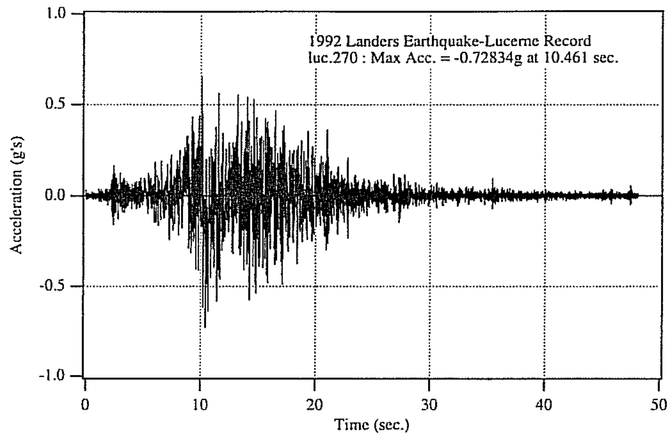
Floor : 6th

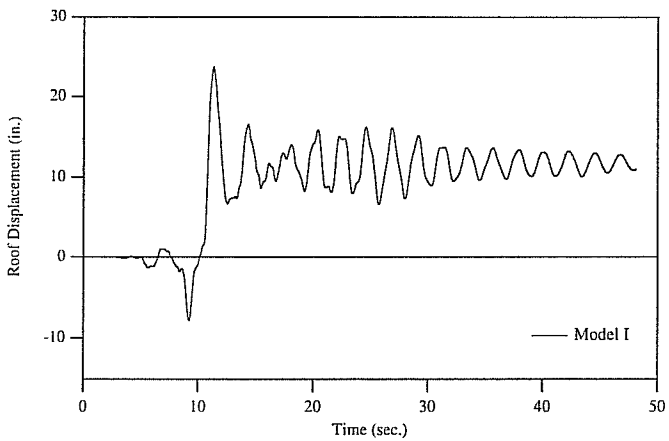
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01047	-0.01305	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.01298	-0.00420	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01047	-0.01306	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.01298	-0.00421	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01086	-0.01297	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.01303	-0.00430	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01082	-0.01392	0.00006	-0.00025	0.00000	0.00000	0.00000	0.00000
G4	0.01698	-0.00422	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

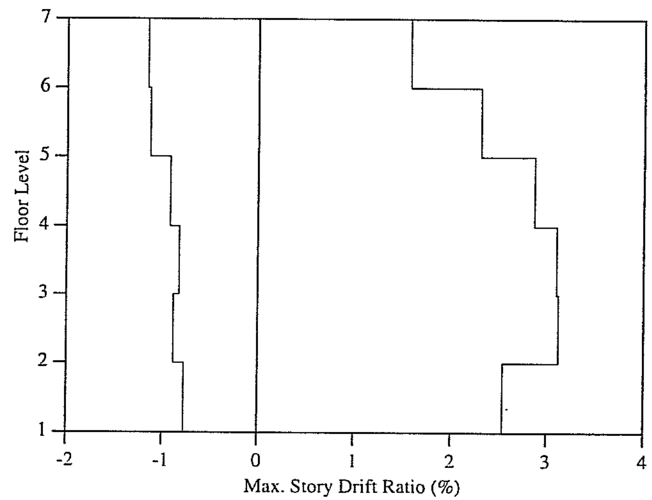
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00824	-0.00123	0.00000	0.00000	0.00096	-0.00359	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00857	0.00000	0.00000
E3	0.00825	-0.00122	0.00000	0.00000	0.00095	-0.00359	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00857	0.00000	0.00000
F3	0.00860	-0.00127	0.00000	0.00000	0.00056	-0.00391	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00882	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00231	-0.00673	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00105	-0.00957	0.00000	0.00000

ANALYSIS : LUC.270





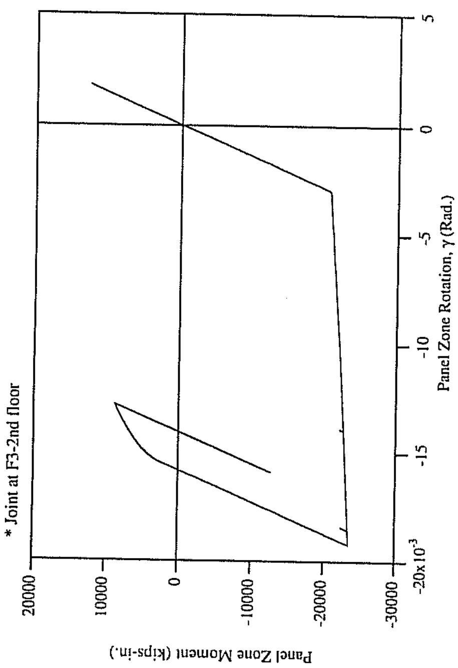
* Floor level 1 is Ground level
Floor level 7 is Roof

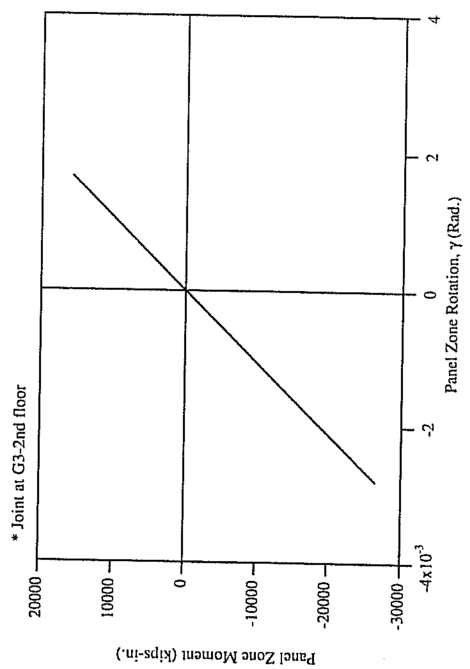


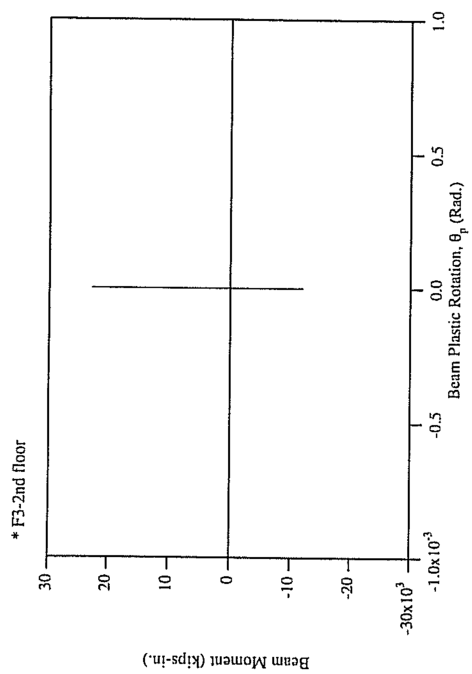
Maximum Story Drift Ratio

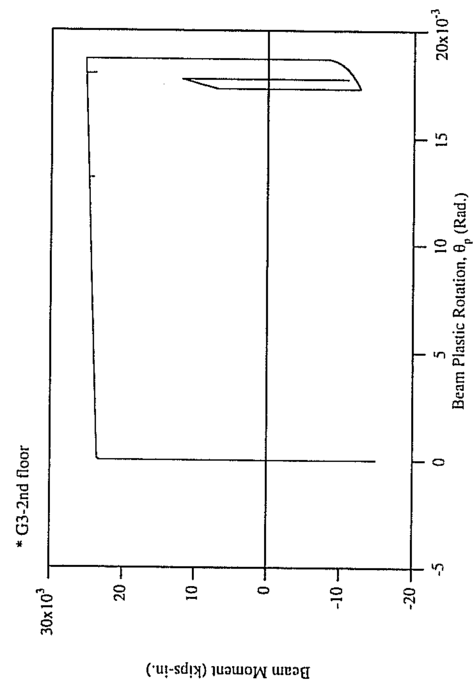
Analysis of Model I : LUC.270

Ground Level-2nd Floor	-0.00769	0.02541
2nd Floor-3rd Floor	-0.00882	0.03121
3rd Floor-4th Floor	-0.00824	0.03102
4th Floor-5th Floor	-0.00922	0.02869
5th Floor-6th Floor	-0.01133	0.02309
6th Floor-Roof	-0.01158	0.01577


* Joint at F3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
20000
10000
0
-10000
-20000
-30000
-20x10^-3
-15
-10
-5
0
5


* Joint at G3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)
20000
10000
0
-10000
-20000
-30000
-4x10^-3
-2
0
2
4


* F3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)
30
20
10
0
-10
-20
-30x10^3
-1.0x10^-3
-0.5
0.0
0.5
1.0


* G3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)
30x10^3
20
10
0
-10
-20
-5
0
5
10
15
20x10^-3

Analysis of Model I : LUC.270

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00999	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00989	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01019	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00971	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01030	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00877	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01087	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00931	0.00000

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01862	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01864	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	-0.01583	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.01723	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01870	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01181	0.00545	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01989	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01321	0.00478	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01989	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01322	0.00478	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00060	0.00000	0.00000	-0.01581	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.01772	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01989	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01322	0.00479	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01803	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01278	0.00350	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01804	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01278	0.00350	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01717	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.01417	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01809	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01280	0.00353	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 5th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01325	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.01082	0.00078	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01326	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.01082	0.00079	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01238	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.01028	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01319	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.01078	0.00074	0.00000	0.00000	0.00000	0.00000	0.00000

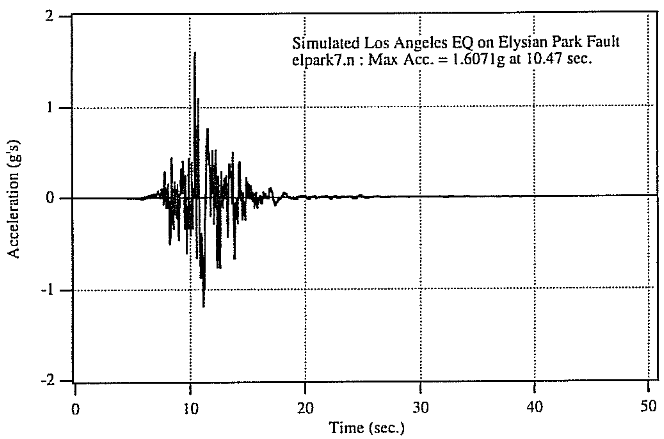
Floor : 6th

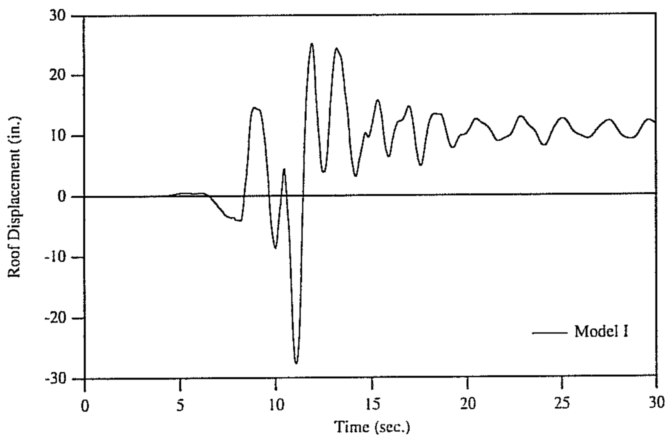
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00581	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	-0.00304	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00582	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	-0.00305	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00598	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	-0.00319	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00669	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	-0.00369	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

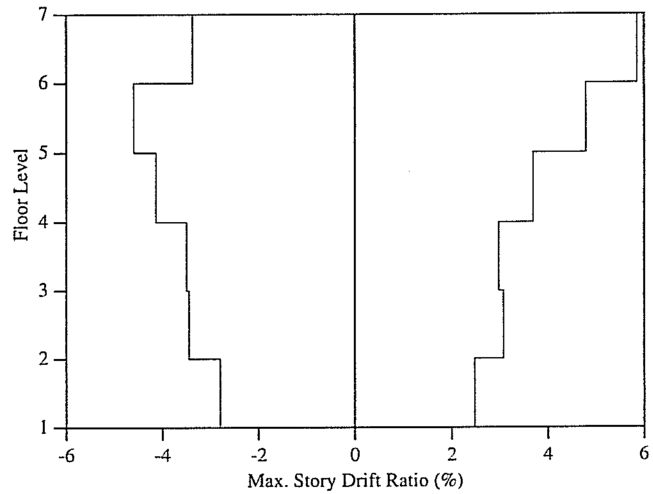
	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

ANALYSIS : Elpark7.N





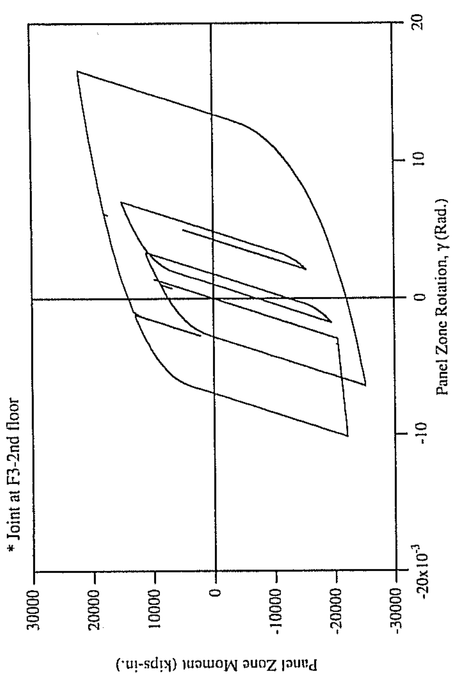
* Floor level 1 is Ground level
Floor level 7 is Roof



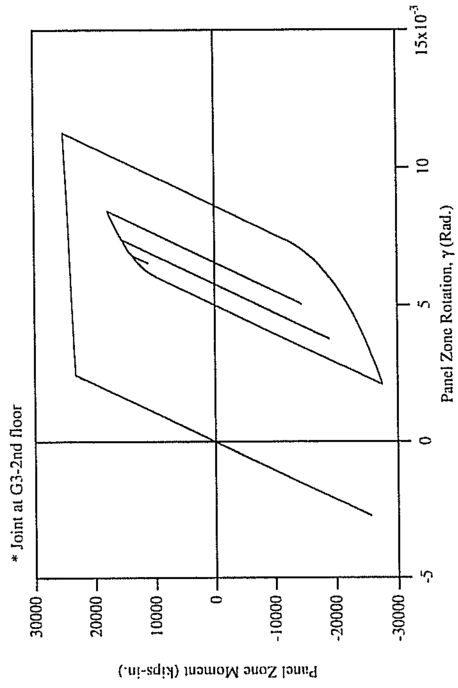
Maximum Story Drift Ratio

Analysis of Model I : ELPARK7.N

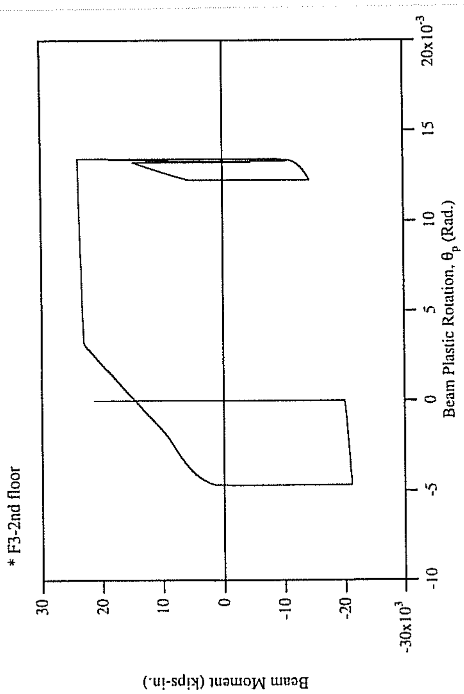
Ground Level-2nd Floor	-0.02802	0.02477
2nd Floor-3rd Floor	-0.03455	0.03070
3rd Floor-4th Floor	-0.03507	0.02981
4th Floor-5th Floor	-0.04136	0.03686
5th Floor-6th Floor	-0.04594	0.04782
6th Floor-Roof	-0.03379	0.05854



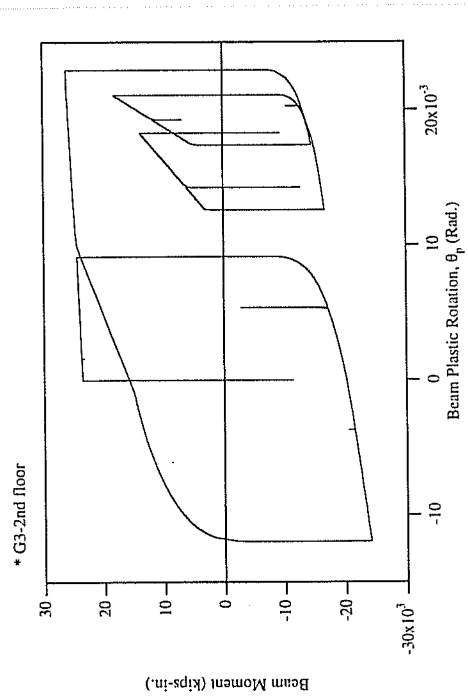
* Joint at F3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)



* Joint at G3-2nd floor
Panel Zone Moment (kips-in.)
Panel Zone Rotation, γ (Rad.)



* F3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)



* G3-2nd floor
Beam Moment (kips-in.)
Beam Plastic Rotation, θp (Rad.)

Analysis of Model I : ELPARK7.N

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00793	-0.01012
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00781	-0.01301
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00814	-0.01033
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00761	-0.01293
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00855	-0.01093
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00715	-0.01277
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00882	-0.01109
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00720	-0.01275

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02278	-0.01189	0.00855	0.00000	0.00231	0.00000	0.00000	0.00000
D4	0.01943	-0.01376	0.00445	-0.00239	0.00189	0.00000	0.00000	0.00000
E3	0.02280	-0.01191	0.00857	0.00000	0.00234	0.00000	0.00000	0.00000
E4	0.01942	-0.01376	0.00445	-0.00239	0.00186	0.00000	0.00000	0.00000
F3	0.01343	-0.00468	0.01341	-0.00696	0.00248	0.00000	0.00000	0.00000
F4	0.01530	0.00000	0.01983	-0.00313	0.00067	0.00000	0.00000	0.00000
G3	0.02295	-0.01194	0.00863	0.00000	0.00246	0.00000	0.00000	0.00000
G4	0.01943	-0.01377	0.00446	-0.00238	0.00179	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.02090	-0.01241	0.00868	0.00000	0.00281	0.00000	0.00000	0.00000
D4	0.02363	-0.01008	0.00468	0.00000	0.00079	0.00000	0.00000	0.00000
E3	0.02090	-0.01241	0.00868	0.00000	0.00281	0.00000	0.00000	0.00000
E4	0.02364	-0.01008	0.00468	0.00000	0.00079	0.00000	0.00000	0.00000
F3	0.01239	-0.00527	0.01477	-0.00371	0.00186	0.00000	0.00000	0.00000
F4	0.01552	0.00000	0.01612	-0.00499	0.00000	0.00000	0.00000	0.00000
G3	0.02062	-0.01242	0.00868	0.00000	0.00284	0.00000	0.00000	0.00000
G4	0.02365	-0.01010	0.00469	0.00000	0.00079	0.00000	0.00000	0.00000

Floor : 4th

Joint	Beam, θ_p positive	Beam, θ_p Negative	Panel Zone, γ_p Positive	Panel Zone, γ_p Negative	Column Below Joint, θ_p Positive	Column Below Joint, θ_p Negative	Column Above Joint, θ_p Positive	Column Above Joint, θ_p Negative
D3	0.02424	-0.01325	0.00908	0.00000	0.00455	-0.00101	0.00080	0.00000
D4	0.02528	-0.01114	0.00630	0.00000	0.00430	-0.00181	0.00000	0.00000
E3	0.02423	-0.01326	0.00909	0.00000	0.00455	-0.00101	0.00080	0.00000
E4	0.02529	-0.01114	0.00630	0.00000	0.00430	-0.00181	0.00000	0.00000
F3	0.01834	-0.01913	0.00000	0.00000	0.00390	-0.00112	0.00047	0.00000
F4	0.02463	-0.01464	0.00000	0.00000	0.00396	-0.00203	0.00107	0.00000
G3	0.02423	-0.01334	0.00913	0.00000	0.00466	-0.00083	0.00086	0.00000
G4	0.02533	-0.01115	0.00629	0.00000	0.00432	-0.00173	0.00000	0.00000

Floor : 5th

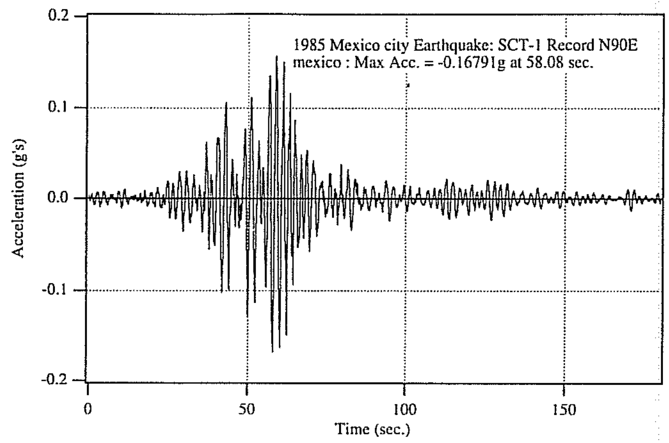
Joint	Beam, θ_p positive	Beam, θ_p Negative	Panel Zone, γ_p Positive	Panel Zone, γ_p Negative	Column Below Joint, θ_p Positive	Column Below Joint, θ_p Negative	Column Above Joint, θ_p Positive	Column Above Joint, θ_p Negative
D3	0.03478	-0.01852	0.01287	0.00000	0.00069	0.00000	0.00000	0.00000
D4	0.03348	-0.01615	0.01189	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.03478	-0.01853	0.01288	0.00000	0.00069	0.00000	0.00000	0.00000
E4	0.03349	-0.01615	0.01189	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.02729	-0.02629	0.00143	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.03366	-0.02352	0.00032	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.03467	-0.01847	0.01281	0.00000	0.00075	0.00000	0.00000	0.00000
G4	0.03349	-0.01603	0.01180	0.00000	0.00000	0.00000	0.00000	0.00000

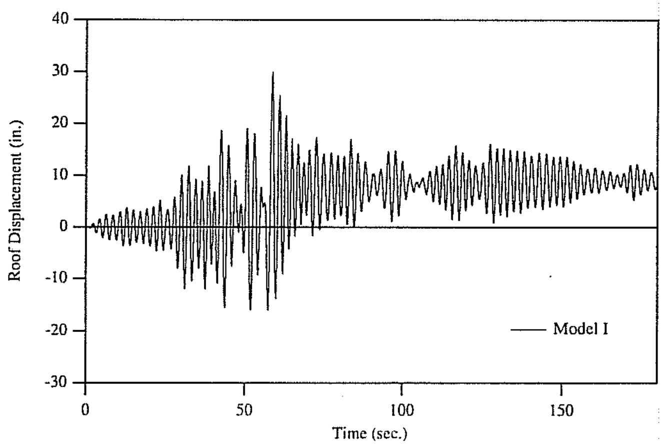
Floor : 6th

Joint	Beam, θ_p positive	Beam, θ_p Negative	Panel Zone, γ_p Positive	Panel Zone, γ_p Negative	Column Below Joint, θ_p Positive	Column Below Joint, θ_p Negative	Column Above Joint, θ_p Positive	Column Above Joint, θ_p Negative
D3	0.03920	-0.01716	0.00285	-0.00072	0.00237	-0.01370	0.00385	0.00000
D4	0.02359	-0.02470	0.01423	0.00000	0.00040	-0.01380	0.00151	0.00000
E3	0.03920	-0.01716	0.00285	-0.00072	0.00237	-0.01370	0.00385	0.00000
E4	0.02337	-0.02493	0.01423	0.00000	0.00040	-0.01380	0.00151	0.00000
F3	0.03876	-0.01721	0.00293	-0.00062	0.00222	-0.01405	0.00423	0.00000
F4	0.02340	-0.02491	0.01421	0.00000	0.00075	-0.01375	0.00118	0.00000
G3	0.04187	-0.01818	0.00414	0.00000	0.00188	-0.01177	0.00696	0.00000
G4	0.02414	-0.02647	0.01517	0.00000	0.00000	-0.01312	0.00466	0.00000

Floor : Roof

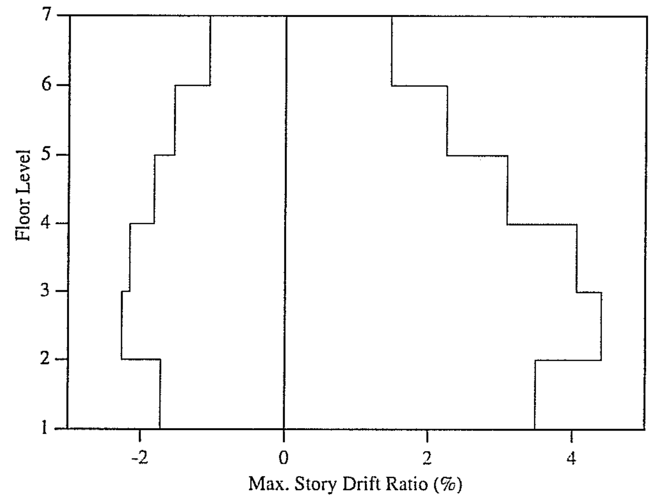
Joint	Beam, θ_p positive	Beam, θ_p Negative	Panel Zone, γ_p Positive	Panel Zone, γ_p Negative	Column Below Joint, θ_p Positive	Column Below Joint, θ_p Negative	Column Above Joint, θ_p Positive	Column Above Joint, θ_p Negative
D3	0.03083	-0.00999	0.00000	0.00000	0.01302	-0.00374	0.00000	0.00000
D4	0.00086	-0.03613	0.00001	0.00000	0.00014	-0.01773	0.00000	0.00000
E3	0.03083	-0.01000	0.00000	0.00000	0.01303	-0.00373	0.00000	0.00000
E4	0.00086	-0.03614	0.00001	0.00000	0.00013	-0.01774	0.00000	0.00000
F3	0.03129	-0.00876	0.00000	0.00000	0.01269	-0.00394	0.00000	0.00000
F4	0.00080	-0.03612	0.00000	0.00000	0.00000	-0.01780	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.04506	-0.01388	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.04015	-0.01881	0.00000	0.00000





ANALYSIS : 1985 Mexico City Earthquake: SCT-1 Record N90E

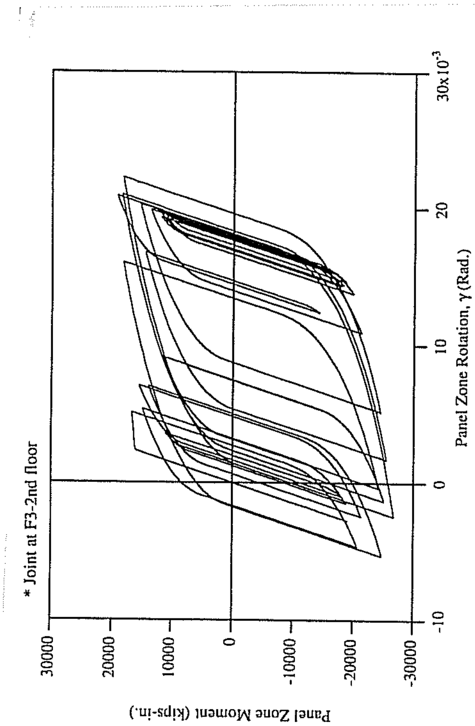
* Floor level 1 is Ground level
Floor level 7 is Roof

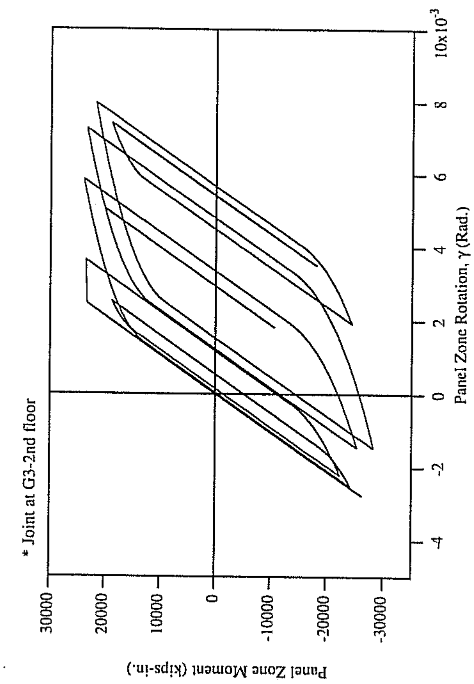


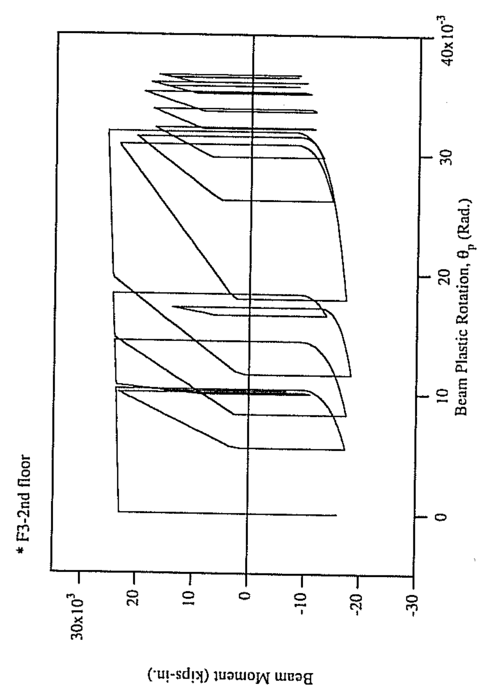
Maximum Story Drift Ratio

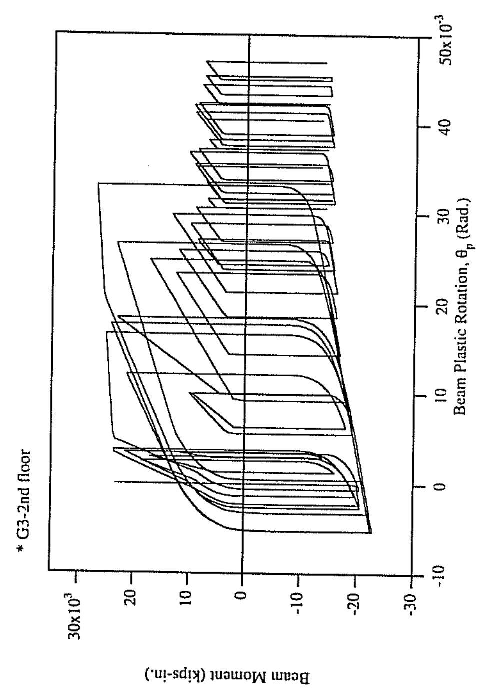
Analysis of Model I : 1985 Mexico city Earthquake: SCT-1 Record N90E

Ground Level-2nd Floor	-0.01726	0.03485
2nd Floor-3rd Floor	-0.02264	0.04395
3rd Floor-4th Floor	-0.02154	0.04050
4th Floor-5th Floor	-0.01823	0.03090
5th Floor-6th Floor	-0.01550	0.02250
6th Floor-Roof	-0.01065	0.01468


* Joint at F3-2nd floor
Panel Zone Moment (kips-in.)
30000
20000
10000
0
-10000
-20000
-30000
-10
0
10
20
30x10^-3
Panel Zone Rotation, γ (Rad.)


* Joint at G3-2nd floor
Panel Zone Moment (kips-in.)
30000
20000
10000
0
-10000
-20000
-30000
-4
-2
0
2
4
6
8
10x10^-3
Panel Zone Rotation, γ (Rad.)


* F3-2nd floor
Beam Moment (kips-in.)
30x10^3
20
10
0
-10
-20
-30
0
10
20
30
40x10^-3
Beam Plastic Rotation, θp (Rad.)


* G3-2nd floor
Beam Moment (kips-in.)
30x10^3
20
10
0
-10
-20
-30
-10
0
10
20
30
40
50x10^-3
Beam Plastic Rotation, θp (Rad.)

Analysis of Model I : 1985 Mexico SCT-1 N90E

Floor : Ground

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01710	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01706	-0.00035
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01733	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01686	-0.00031
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01787	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01609	-0.00025
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01813	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.01647	-0.00020

Floor : 2nd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04559	-0.00523	0.00549	-0.00007	0.00000	0.00000	0.00000	0.00000
D4	0.01683	-0.01966	0.01342	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.04701	-0.00524	0.00553	-0.00004	0.00000	0.00000	0.00000	0.00000
E4	0.01679	-0.01968	0.01341	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.03665	0.00000	0.01961	-0.00175	0.00000	0.00000	0.00000	0.00000
F4	0.02507	0.00000	0.03498	-0.00271	0.00000	0.00000	0.00000	0.00000
G3	0.04695	-0.00529	0.00567	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.01601	-0.01969	0.01340	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 3rd

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04497	-0.00426	0.00449	-0.00041	0.00000	0.00000	0.00000	0.00000
D4	0.01032	-0.02164	0.01062	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.04497	-0.00426	0.00449	-0.00041	0.00000	0.00000	0.00000	0.00000
E4	0.01035	-0.02164	0.01063	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.03666	0.00000	0.01854	-0.00198	0.00000	0.00000	0.00000	0.00000
F4	0.02650	0.00000	0.03401	-0.00014	0.00000	0.00000	0.00000	0.00000
G3	0.04496	-0.00424	0.00446	-0.00043	0.00000	0.00000	0.00000	0.00000
G4	0.01041	-0.02162	0.01066	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 4th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.04221	-0.00301	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00607	-0.01998	0.00683	-0.00025	0.00000	0.00000	0.00000	0.00000
E3	0.04221	-0.00303	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00607	-0.01998	0.00684	-0.00025	0.00000	0.00000	0.00000	0.00000
F3	0.03007	-0.00558	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00763	-0.01957	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.04210	-0.00325	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00611	-0.01999	0.00688	-0.00023	0.00000	0.00000	0.00000	0.00000

Floor : 5th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.01827	-0.00426	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00510	-0.00908	0.00253	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.01829	-0.00427	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00511	-0.00908	0.00254	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.01351	-0.00357	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00934	-0.00689	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.01821	-0.00418	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00503	-0.00906	0.00245	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : 6th

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00692	-0.00026	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00013	-0.00739	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00693	-0.00026	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00014	-0.00753	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00696	-0.00268	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00382	-0.00387	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00935	-0.00280	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00053	-0.00767	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Floor : Roof

	Beam, θ_p		Panel Zone, γ_p		Column Below Joint, θ_p		Column Above Joint, θ_p	
Joint	positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
D3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
D4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
E4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
F4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
G4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000