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SEISMIC PERFORMANCE STUDY OF A 2-STORY MASONRY WALL-FRAME BUILDING DESIGNED BY TENTATIVE LIMIT STATES DESIGN STANDARD

by

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EKEH

Ewing/Kariotis/Englekirk & Hart

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CHAPTER 1
INTRODUCTION

Task 2 of the TCCMAR Masonry Research Program funded by the National Science Foundation developed analytical methods for modeling the seismic performance of reinforced masonry buildings. The fundamental objective of the Task 2 research was to develop analytical models that were validated using comparison of predicted and measured laboratory experiments performed by other TCCMAR researchers. The final product of the collaborative TCCMAR research is the development of masonry limit states design recommendations.

This report has the following purposes:

(1) To develop analytical models using the Task 2 computer programs of a case study building that was designed using the Limit State Design Standard (LSDS) developed by The Masonry Society, American Concrete Institute and American Society of Civil Engineers and quantify its performance.

(2) To study the sensitivity of the estimate component and system response to variations in material properties, modeling assumptions, and different analytical models.

(3) To make recommendations ,based upon the results of (1) and (2), for improving the Limit State Design Standard.

The performance of a case study must start with a design of the case study building. Therefore, Chapter 2 presents a description of the building in concept similar to the type and form that would be provided by an architect or owner/developer. Chapter 3 then follows with a description of the LSDS design for the case study building with the calculations being presented in Appendix A.

The performance study presented herein can be visualized as an analytical experiment where many different analyses are performed. The reader will, we are sure, draw conclusions from the results presented in this report. However, it is left to the final Task 2 summary report for the authors to state specific conclusions and thus this report has no final conclusion section.

CHAPTER 2
BUILDING DESCRIPTION

This two-story office building of regular configuration is 80 ft. long in the EW direction and 72 ft. wide in the NS direction. The structure rises 28 ft.- 6 in. above grade with a first story height of 13 ft. and second story height of 13 ft.- 6 in. Typical floor plan and elevations are shown in Figures 2-1 to 2-3. The floor and roof are of plywood diaphragms. The 1 1/2 in. lightweight concrete fill over 3/4 in. plywood sheathing is used for the floor while 3/8 in. plywood sheathing without concrete fill is used for the roof.

The vertical load carrying system consists of wood joists supported on the glued-laminated timber beams at the roof and on the steel beams at the 2nd floor. The beams are supported by steel tube columns. The second floor and roof framing plan are shown in Figures 2-4 and 2-5. The lateral load resisting system consists of three concrete masonry shear walls and one masonry wall frame located on the perimeter of the building as shown in Figures 2-2 and 2-3.

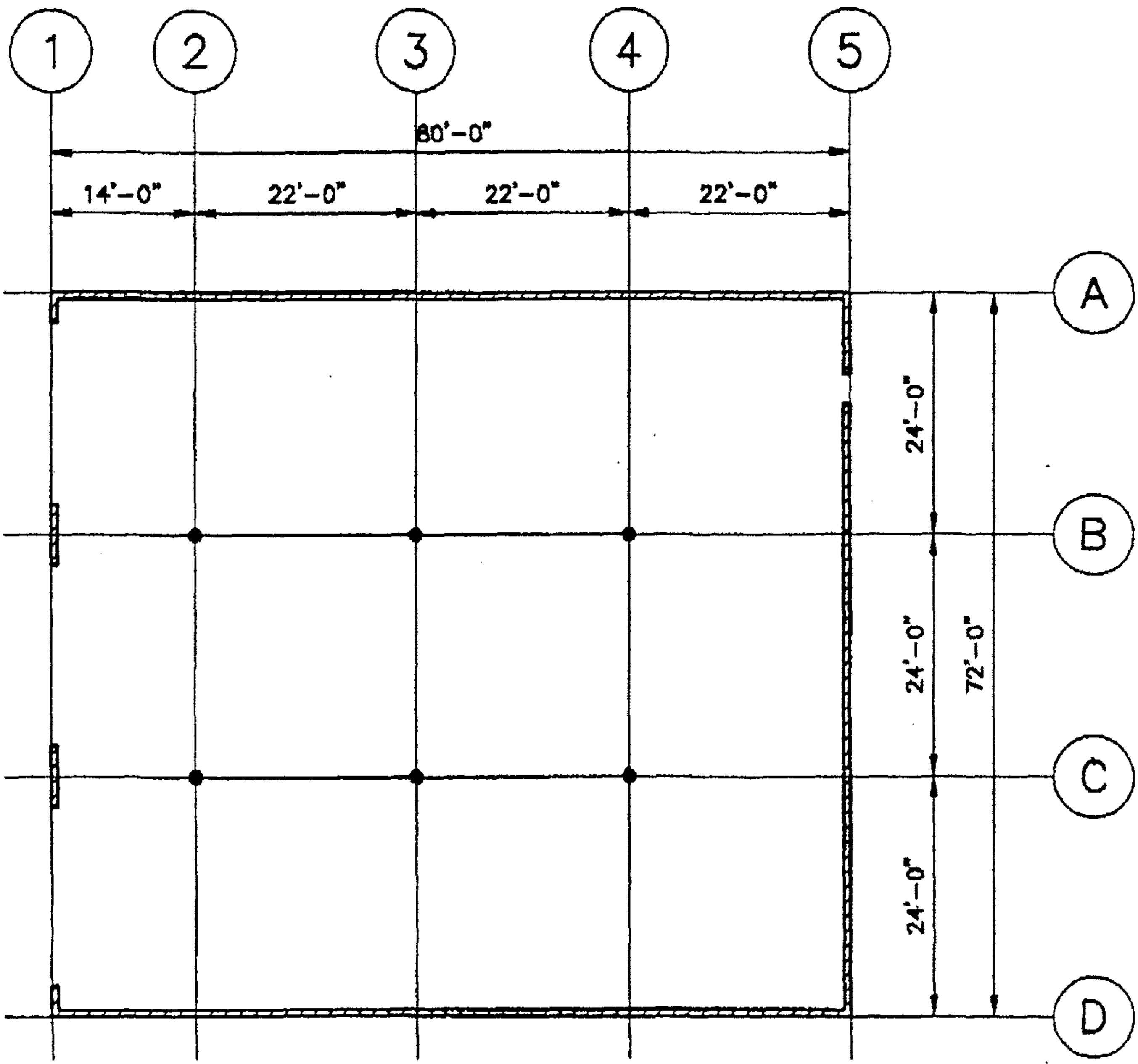
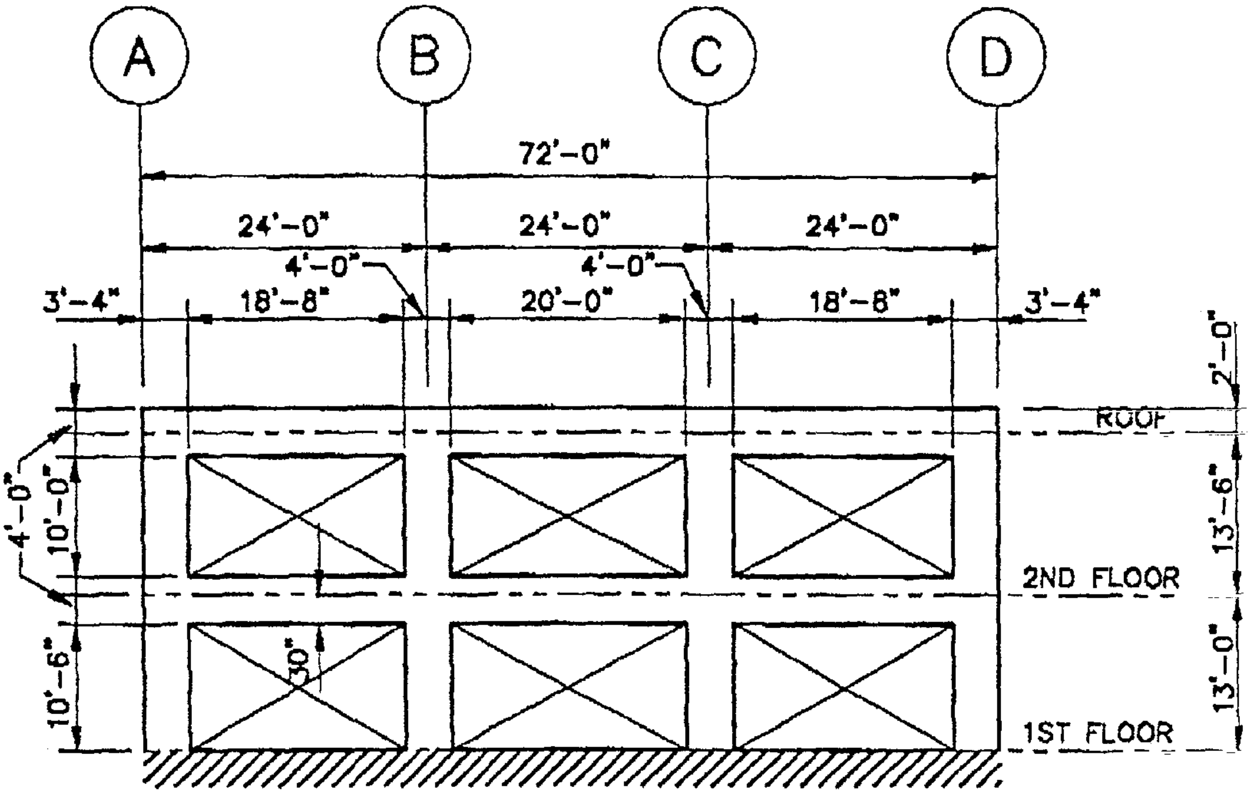


FIGURE 2-1 FLOOR PLAN

SCALE: 1/16"=1'-0"



IGURE 2-2 WEST WALL FRAME ELEVATION

SCALE: 1/16" = 1'-0"

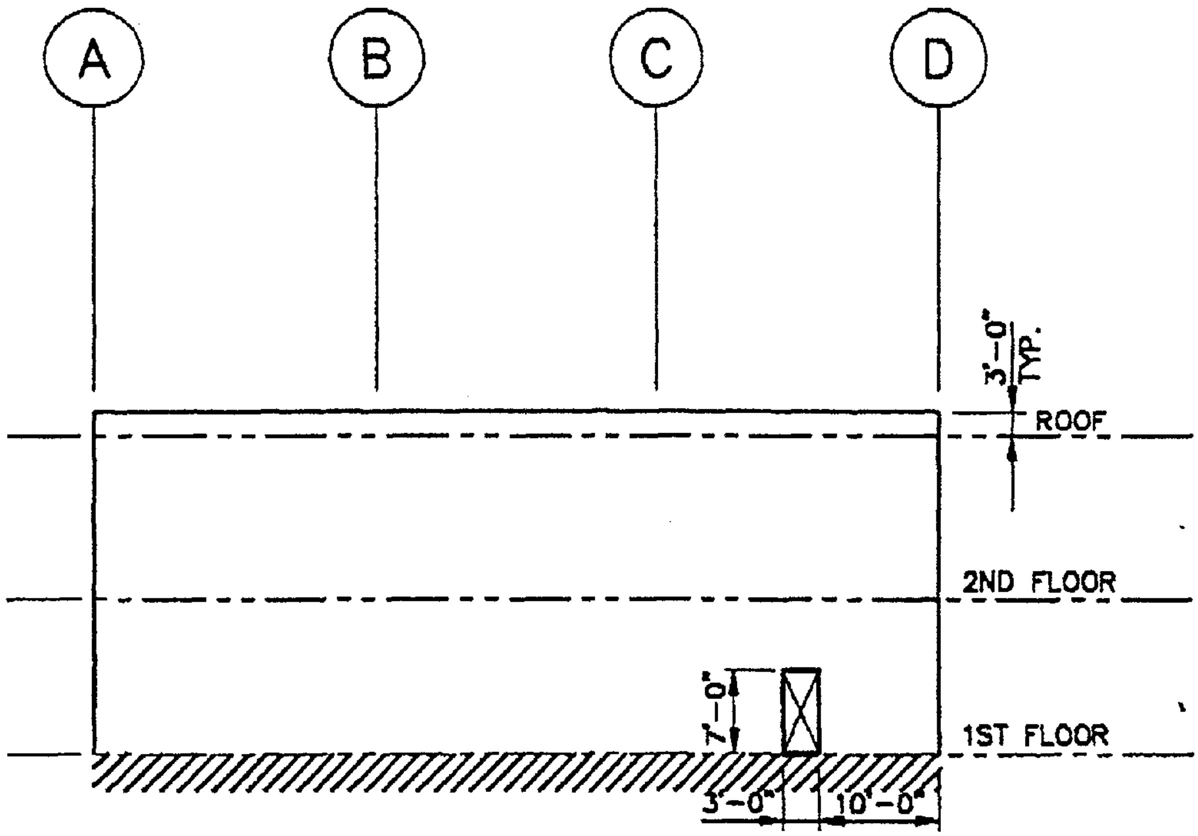


FIGURE 2-3 **EAST WALL ELEVATION**

SCALE: 1/16" = 1'-0"

CHAPTER 3
DESIGN USING TENTATIVE LIMIT STATES DESIGN STANDARD

The building was selected for the trial design to meet the requirements of the tentative Masonry Limit States Design Standards (LSDS) [3-1]. The LSDS adopted the NEHRP Recommended provisions as a basis for general seismic design provisions but with appropriate modifications to take into account, the relationship between the stiffness and strength of the structural system. Further, the LSDS takes the limit states design format which is a departure in concept from the working stress design incorporated in the NEHRP recommended provisions.

For the design development, the masonry wall frame was analyzed using the SAP90 computer program in which the stiffness of the members were based on effective stiffness to calculate the demand on the members. The design of masonry members in the wall frame was further aided by the computer program IMFLEX since it allows the designer to incorporate the desired masonry stress-strain curve in computing the member flexural capacity. The program also facilitates the determination of the required lateral reinforcement to confine vertical reinforcement in the piers.

Detailed design calculations are presented in Appendix A. Figures 3-1 to 3-3 show the required reinforcements and their arrangements for the design.

REFERENCES

3-1 LSDS, Masonry Limit States Design Standard - Draft, The Masonry Society, February 1991.

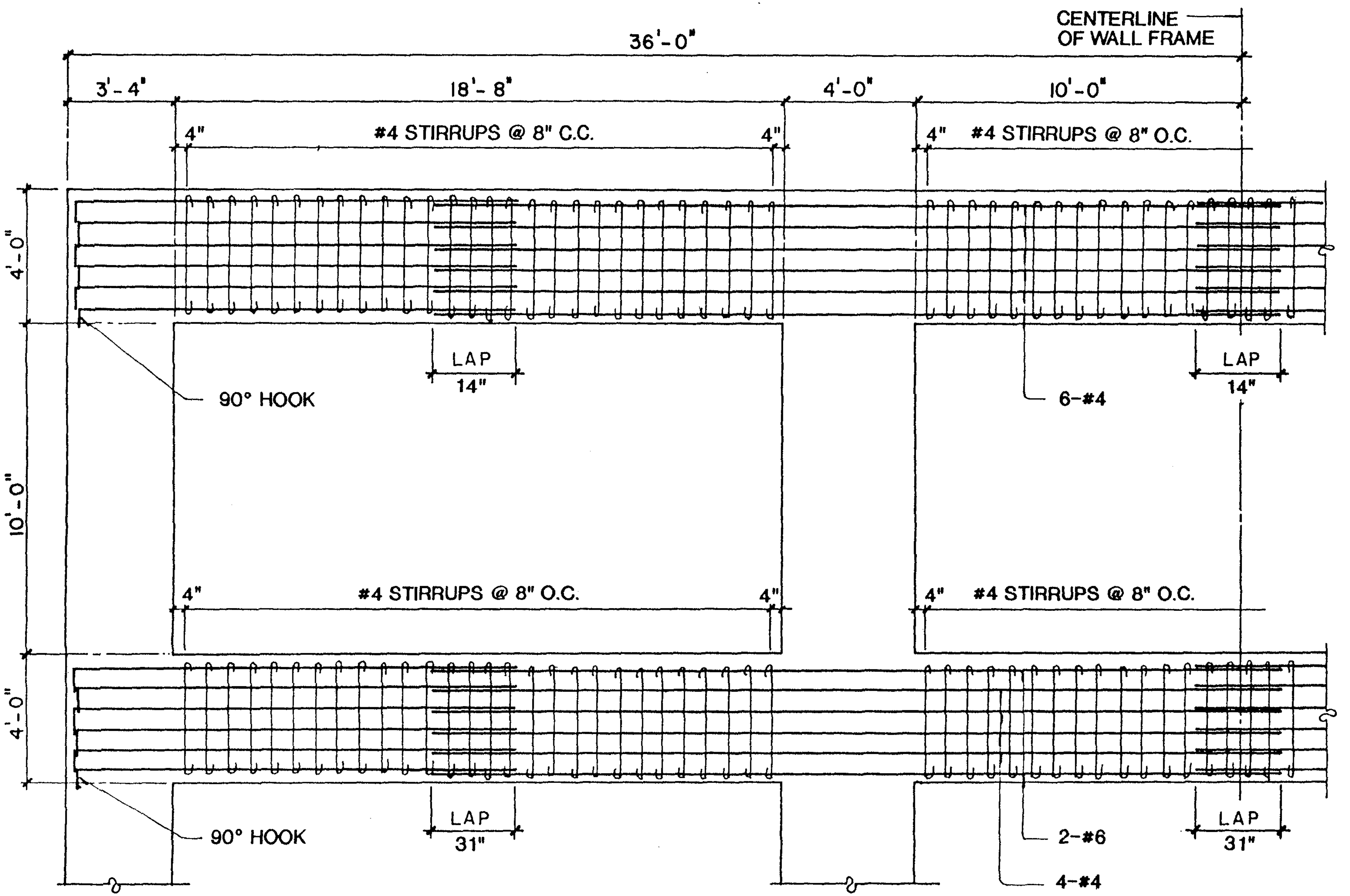
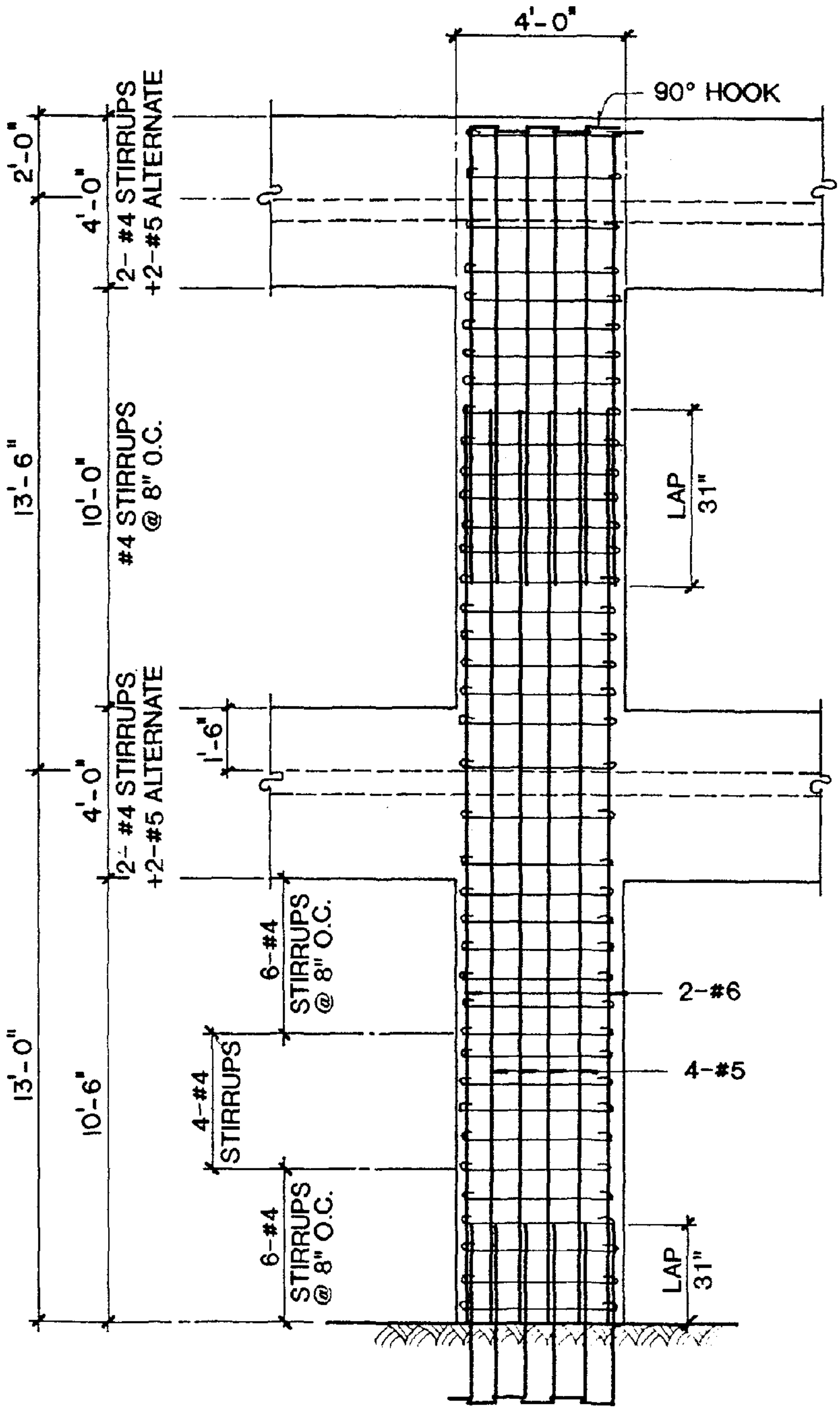


Figure 3-1 Beam Reinforcement (8-inch Wall Frame LSDS Design)

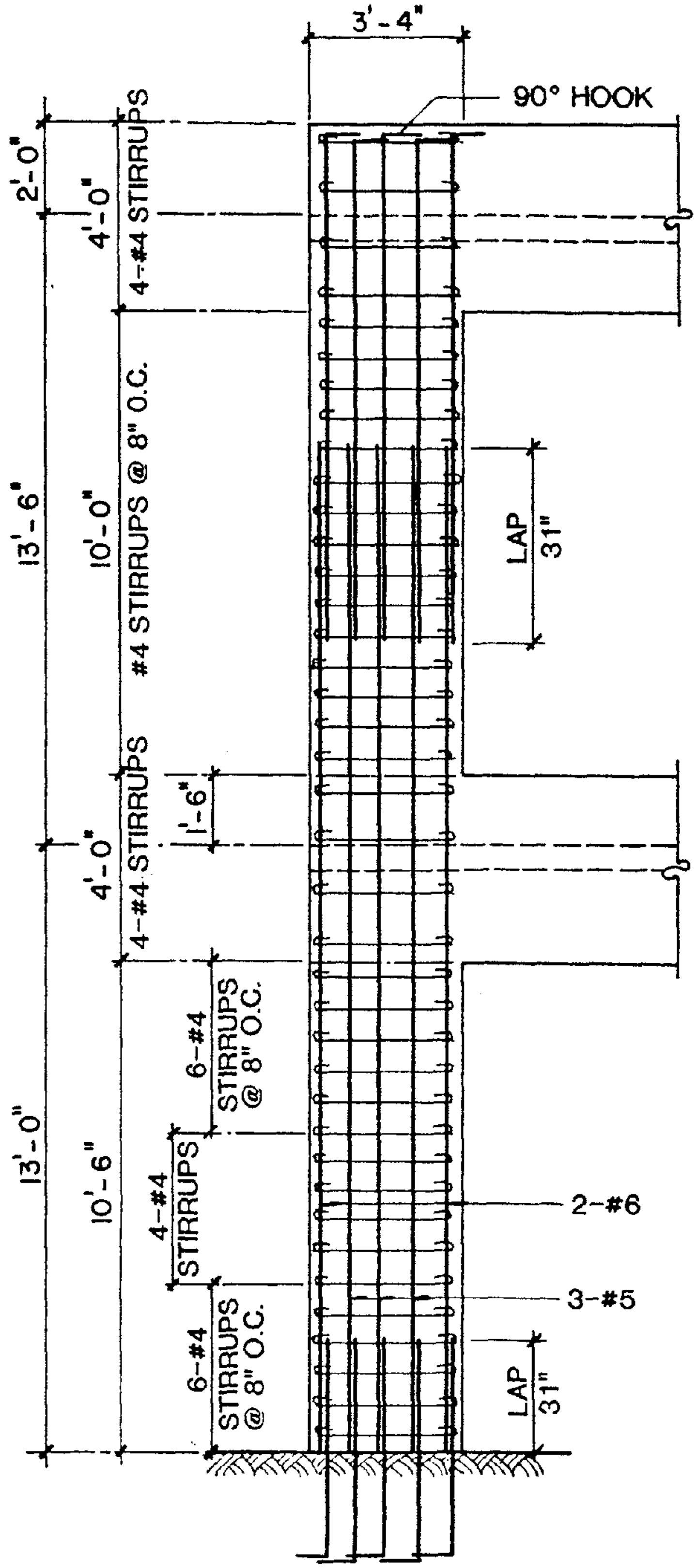
NOTE: LAP DIMENSION NOT TO SCALE



NOTE: LAP DIMENSION NOT TO SCALE.

Figure 3-2 Interior Column and Joint Reinforcement

(8-inch Wall Frame LSDS Design)



NOTE: LAP DIMENSION NOT TO SCALE.

Figure 3-3 Exterior Column and Joint Reinforcement

(8-inch Wall Frame LSDS Design)

CHAPTER 4
EARTHQUAKE GROUND MOTIONS

In conducting the performance evaluation of the wall frame, an ensemble of nine earthquakes was chosen by the TCCMAR task 2 team. Table 4-1 gives a list of the ground motion records selected for this analysis. A complete description of these motions can be found in Kariotis and Associates Report 9.1-2 [4-1]. Figure 4.1 shows these acceleration time histories.

REFERENCES

1. Kariotis, J.C., and Waqfi, O., "Trial Designs made in accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings", Report 9.1-2, February 1992, Kariotis and Associates, South Pasadena, CA.

Table 4-1 Earthquake Ground Motion

Earthquake	Δt (sec)	Duration (sec)	C_S[1]	C_1[2]	C_2[3]	Designation
El Centro, E-W	0.02	53.0	0.03937	0.9255	1.7875	G1.DAT
El Centro, N-S	0.02	53.0	0.03937	0.6777	1.3145	G2.DAT
Pine Union, 140	0.01	29.0	0.003937	0.8622	1.7067	G3.DAT
Cruickshank Rd., 230	0.01	34.0	0.003937	0.7632	1.4951	G4.DAT
James Road, 140	0.01	29.0	0.003937	0.7126	1.3893	G5.DAT
Kern County, 1969	0.02	54.0	0.03937	1.4080	2.8648	G6.DAT
Cruickshank Rd., 140	0.01	34.0	0.003937	0.6157	1.2024	G9.DAT
Brawley Airport, 315	0.01	37.0	0.003937	1.0644	2.0738	G10.DAT
Keystone Rd., 140	0.01	39.0	1.0	0.9485	1.8501	G11.DAT

[1] C_S = Scaling factor for converting acceleration units to in/s/s.
[2] C_1 = Scaling factor for Seismic Zone 2
[3] C_2 = Scaling factor for Seismic Zone 4

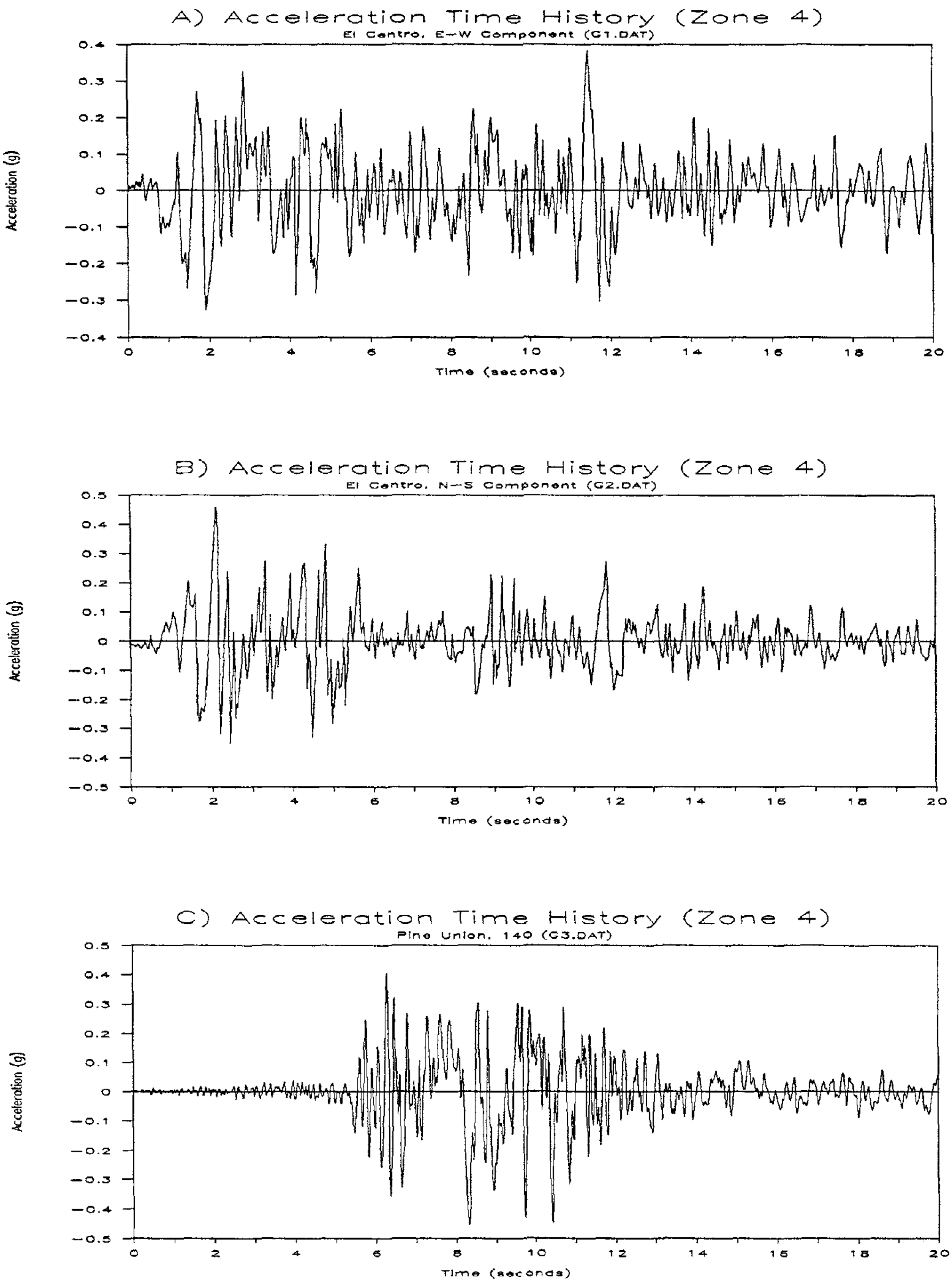
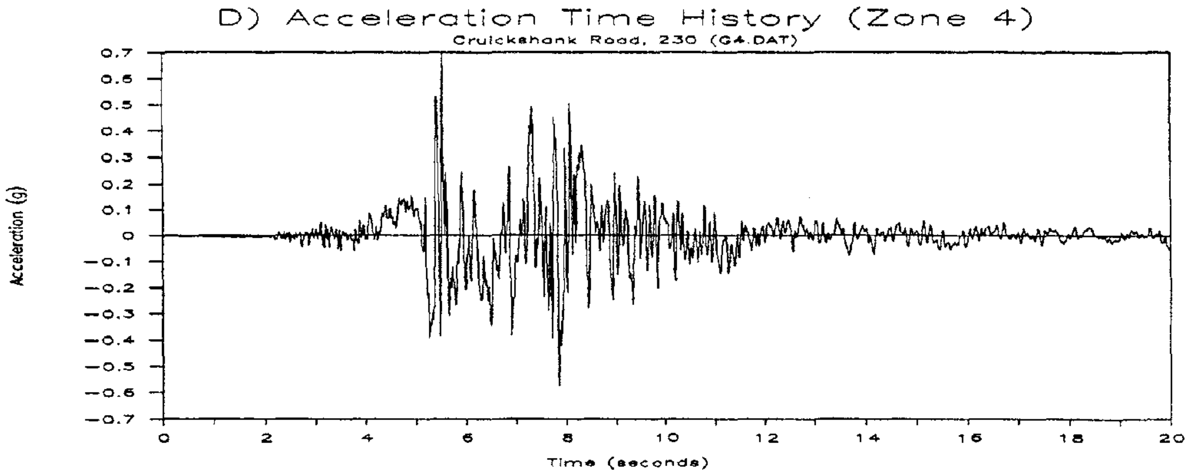
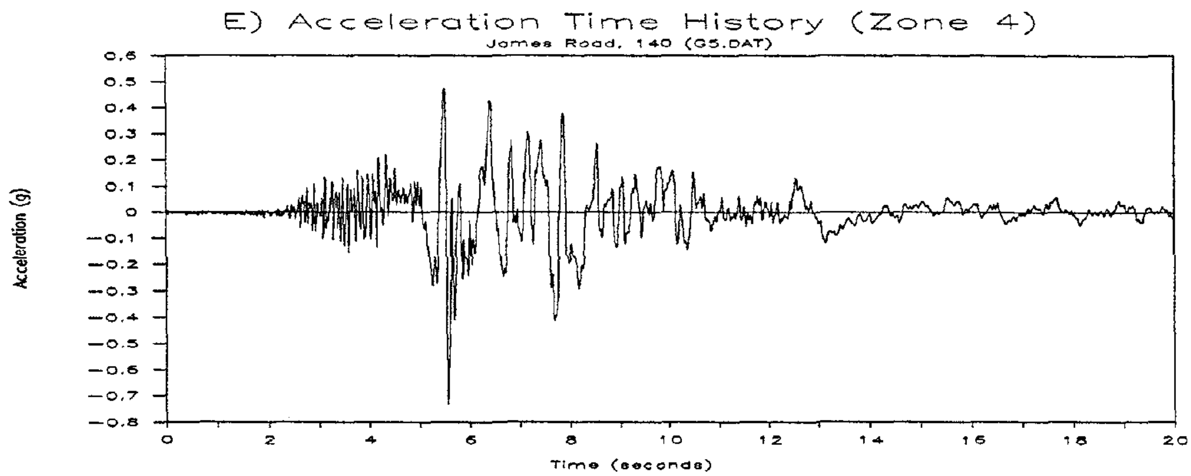


Figure 4-1 Earthquake Ground Motions





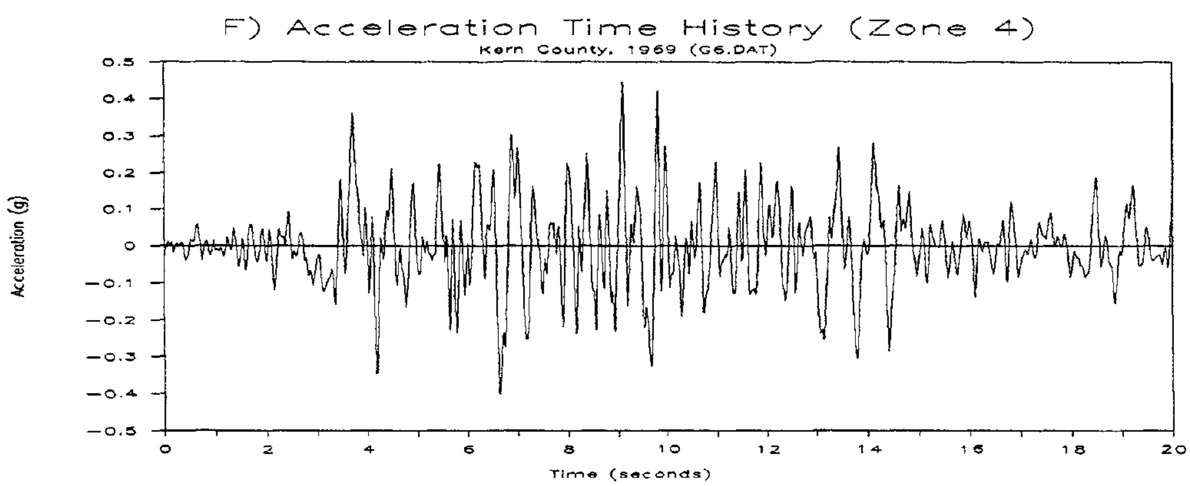
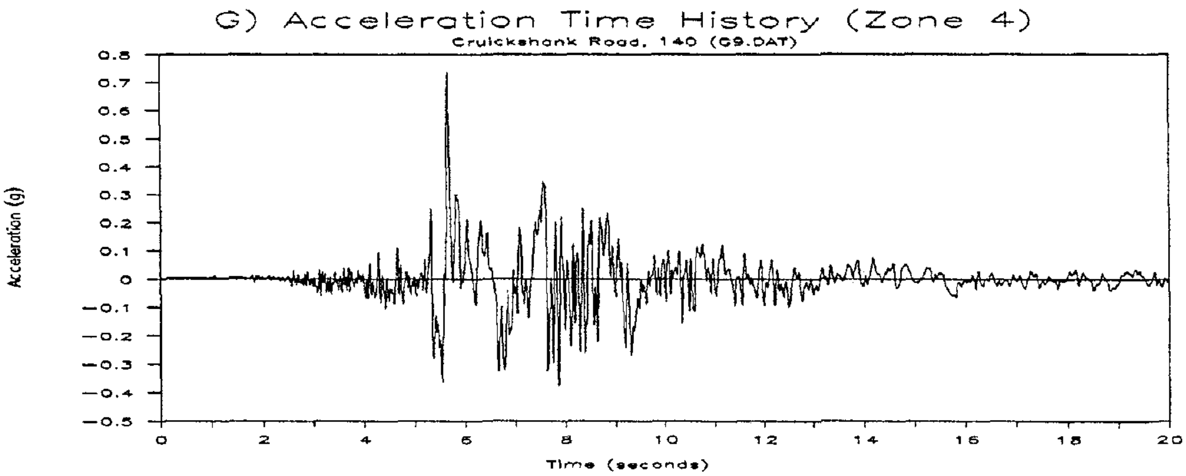
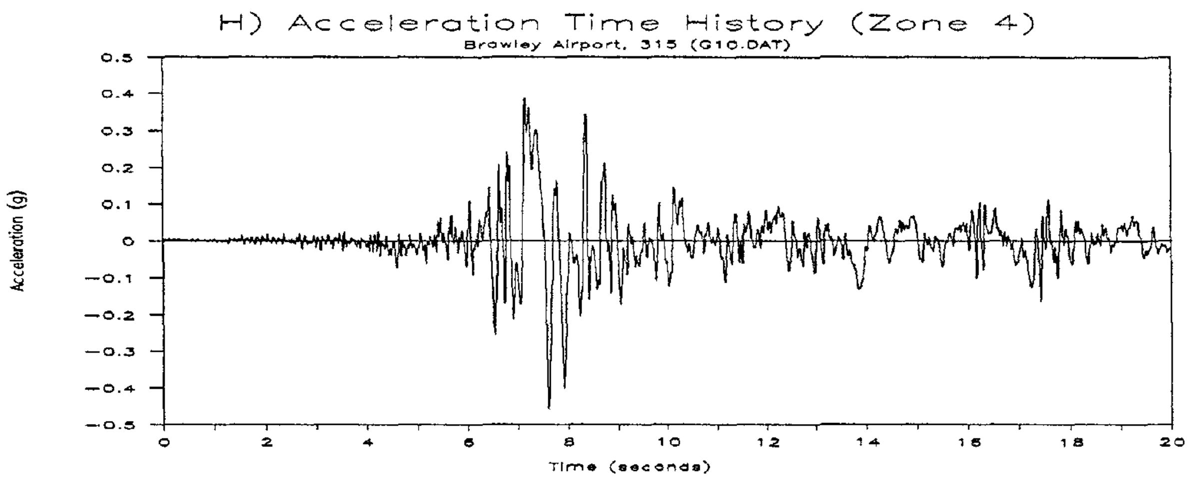


Figure 4-1 (Continued)





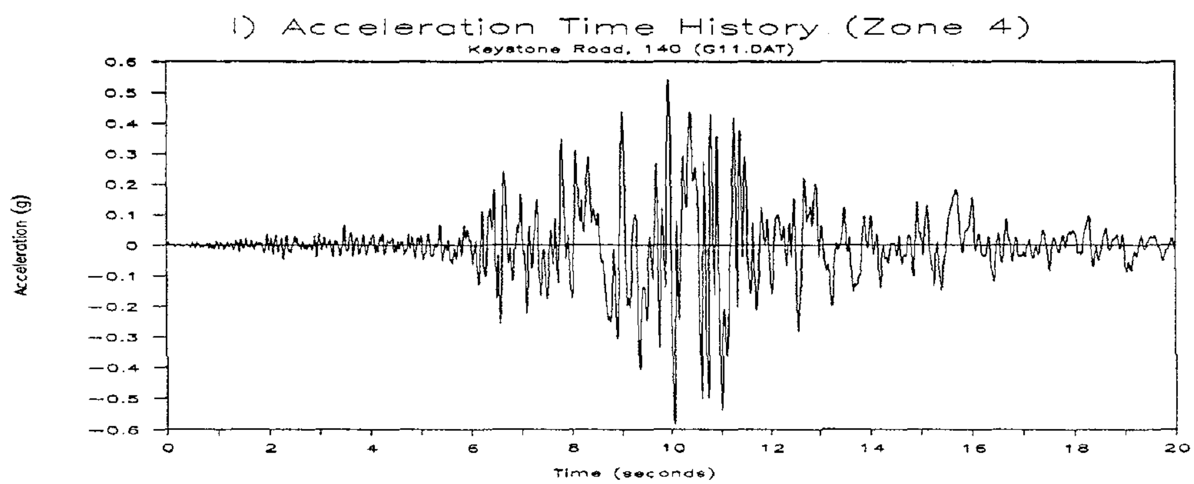


Figure 4-1 (Concluded)

CHAPTER 5
EXPECTED BUILDING PERFORMANCE USING SAP90

5.1 General

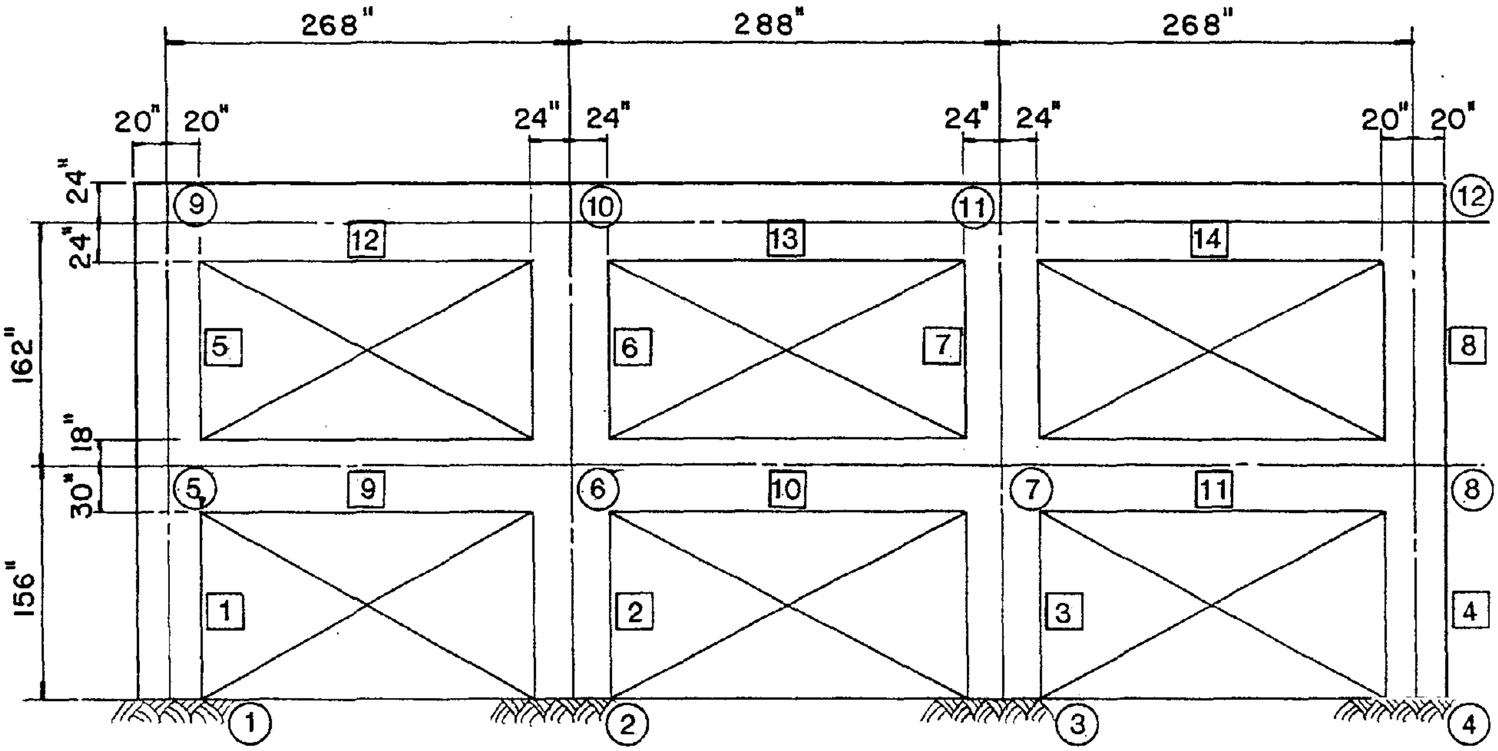
An elastic time history analysis of the wall frame was conducted to study their responses to an ensemble of earthquake ground motions. This analysis studied the global and local response demands so that we can assess the performance of the wall frame system. The analyses were done using the computer program SAP90 [5-1]. The analytical modeling techniques used for the wall frame are also discussed in this chapter.

5.2 Modeling Procedure

Figure 5-1 shows the SAP90 model of the wall frame. In the model, all structural members were represented as linear 2-dimensional beam elements. An analysis based on a centerline to centerline geometry, in general, overestimates deflections. Therefore, this can be alleviated by modelling the joints as rigid zones. Thus, with this model, all member forces are evaluated at the face of the joints. Each nodal point has three degrees of freedom: a horizontal translation, a vertical translation and a rotation. Axial deformations of the beams are neglected. Therefore, only one horizontal DOF is retained for each story level. The model also assumes full fixity at the base of columns.

The nonlinear behavior of a wall frame under earthquake loading may be characterized by performing an equivalent non-linear analysis using a program based on elastic properties such as SAP90. This is done one one hand by using the effective moment of inertia of the frame members to achieve the desired effective stiffness of the system. On the other hand, the equivalent viscous damping associated with the system is estimated to model the energy dissipation characteristics equivalent to the hysteretic damping expected during the inelastic response. It is assumed that a 7% damping ratio along with the effective moment of inertia used for the frame members would closely approximate the nonlinear characteristics of the system.

Historically, member stiffnesses are based on uncracked section properties. However, the use of gross section stiffness has been shown not to give realistic estimates of fundamental periods of vibrations or drift ratios. The member stiffness is dependent on the degree of damage sustained by the member. Various researchers have presented methods for determining the effective moment of inertia in order to characterize the stiffness of the structural member more accurately [5-2 to 5-4]. For example, a formula proposed by Priestley and Hart [5-4], based on TCCMAR theoretical analyses agrees well with experimental results. An iterative process was used to compute the effective member stiffness for the frame. It is described below:



KEY NOTES:

(1) NODE NUMBER

[3] ELEMENT NUMBER

FIGURE 5.1 SAP90 ANALYTICAL MODEL

ITERATIVE PROCESS FOR MEMBER STIFFNESS COMPUTATION

1. Assume initial member sizes and initial stiffness values.

2. Assume or compute the natural period of the frame, compute the base shear, distribute it over the frame height and run the SAP90 analysis of the frame with these lateral loads.

3. Compute the applied moments on the members from the SAP90 run. Use the ACI formula relating the effective moment of inertia to the applied moment on the member. If the assumed moments of inertia match the computed moments of inertia, go to Step 4. If not, choose the next cycle of stiffness values and go to Step 1 till convergence.

4. Use this converged set of stiffness values to set the member sizes and design the members according to the load demands on them.

5.3 Results of Elastic Analysis

An elastic time history analysis was conducted for the wall frame using the ground motions described in Chapter 4. In the modeling, the effects of relative stiffness degradation of the members, based on the study made in the previous section as shown in Figure 5-2, were utilized in the analytical model. Appendix A shows these iterated and final member stiffnesses for the frame. The response of interest in assessing the building system performance are: elastic base shear, relative displacements at all levels, interstory drift ratios, and floor accelerations. Table 5-1 presents the elastic analysis results for the wall frame.

The computed elastic base shear is compared with the base shear strength to determine if the frame system yielded. In Chapter 3, the design base shear was computed to be 18.2% of the building weight. By incorperating a strength reduction factor of 0.85, a minimum nominal base shear strength of approximately 21.4% of the weight is estimated for the code design. Inspection of Table 5-1 reveals that the elastic base shear demand exceeds base shear yielded strength for all of the earthquakes. This suggests that flexural yielding of the frame system is expected. However, this elastic analysis fails to identify the location and sequence of hinge formation. Note that the ratios of roof acceleration to base ground acceleration are close to the base shear demand/strength ratios.

The maximum calculated interstory drift ratios are less than the LSDS allowable drift limit of 1.33% for the wall frame. Figures 5-3 shows the roof displacement of the wall frame for all earthquakes considered.

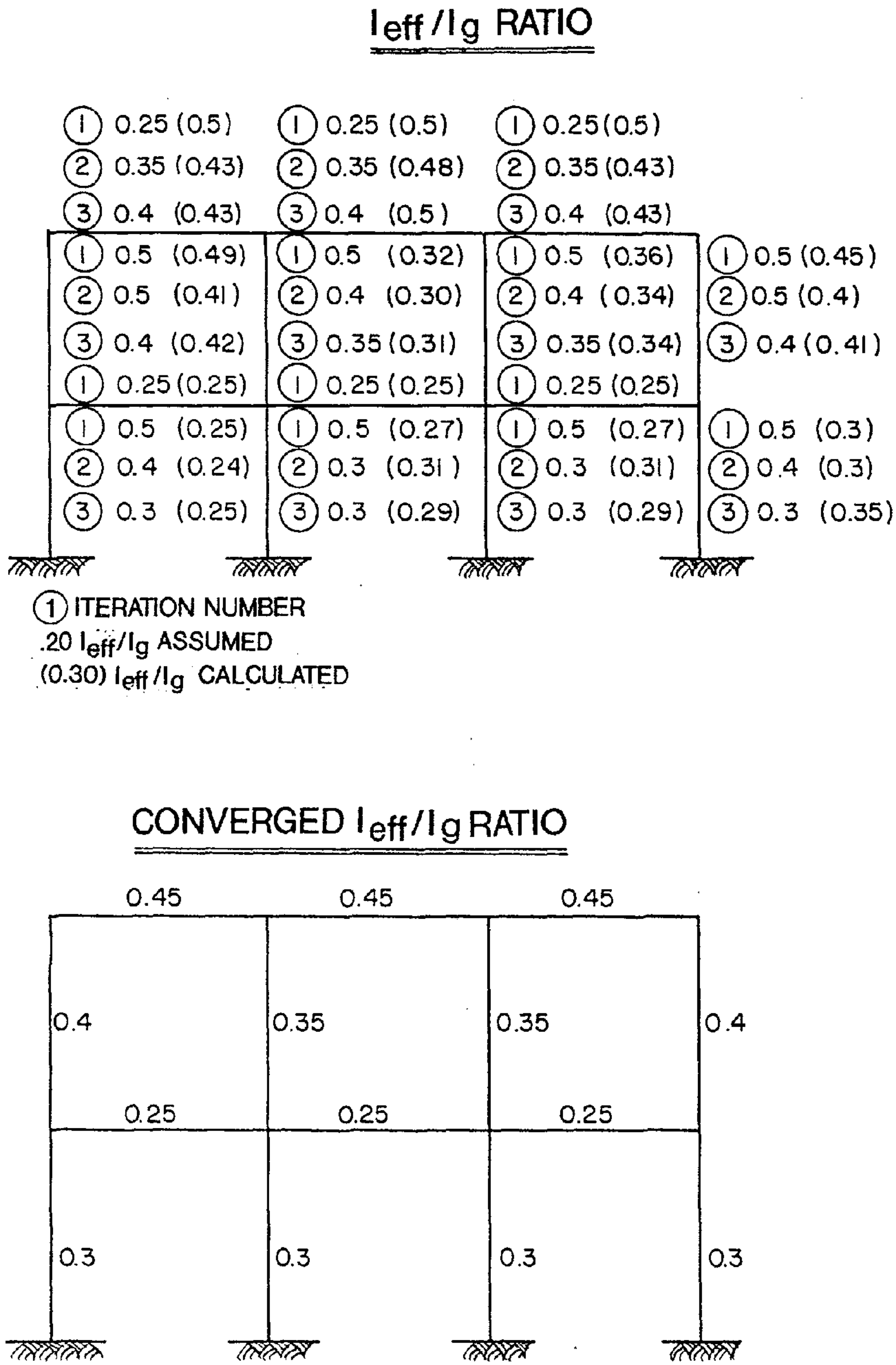


FIGURE 5.2 EFFECTIVE MOMENT OF INERTIA FOR STRUCTURAL MEMBERS

TABLE 5-1 Elastic Analysis Results

Using SAP90 (with Ieff)

	Overall Drift (%)	Roof Drift (%)	Floor Drift (%)	Roof Disp. (inches)	Floor Disp. (inches)	Base Shear V/W (%)	Roof Accel. (g)	Floor Accel. (g)
Response Spectrum Analysis: UBC Zone 4, Soil Type 2								
7% Damping	0.80	0.68	0.94	2.557	1.460	68.8	0.94	0.57
5% Damping	1.09	0.92	1.26	3.456	1.973	92.9	1.27	0.77
Time History Analysis: Earthquake Record								
G1.DAT	1.10	0.91	1.30	3.510	2.032	96.6	1.21	0.83
G2.DAT	1.15	0.95	1.36	3.650	2.117	100.8	1.30	0.87
G3.DAT	0.71	0.63	0.80	2.269	1.247	59.3	0.89	0.52
G4.DAT	0.74	0.62	0.87	2.360	1.350	64.1	0.90	0.62
G5.DAT	0.72	0.60	0.85	2.293	1.329	63.6	0.90	0.59
G6.DAT	0.83	0.71	0.95	2.635	1.477	64.0	1.03	0.63
G9.DAT	0.61	0.50	0.73	1.952	1.146	56.8	0.79	0.70
G10.DAT	0.84	0.70	0.98	2.664	1.532	72.4	0.94	0.65
G11.DAT	1.07	0.92	1.22	3.391	1.899	90.7	1.35	0.92
Minimum	0.61	0.50	0.73	1.952	1.146	56.8	0.79	0.52
Maximum	1.15	0.95	1.36	3.650	2.117	100.8	1.35	0.92
Average	0.86	0.73	1.01	2.747	1.570	74.3	1.03	0.70
C.O.V. (%)	21.2	21.1	21.5	21.2	21.5	21.7	18.4	18.5

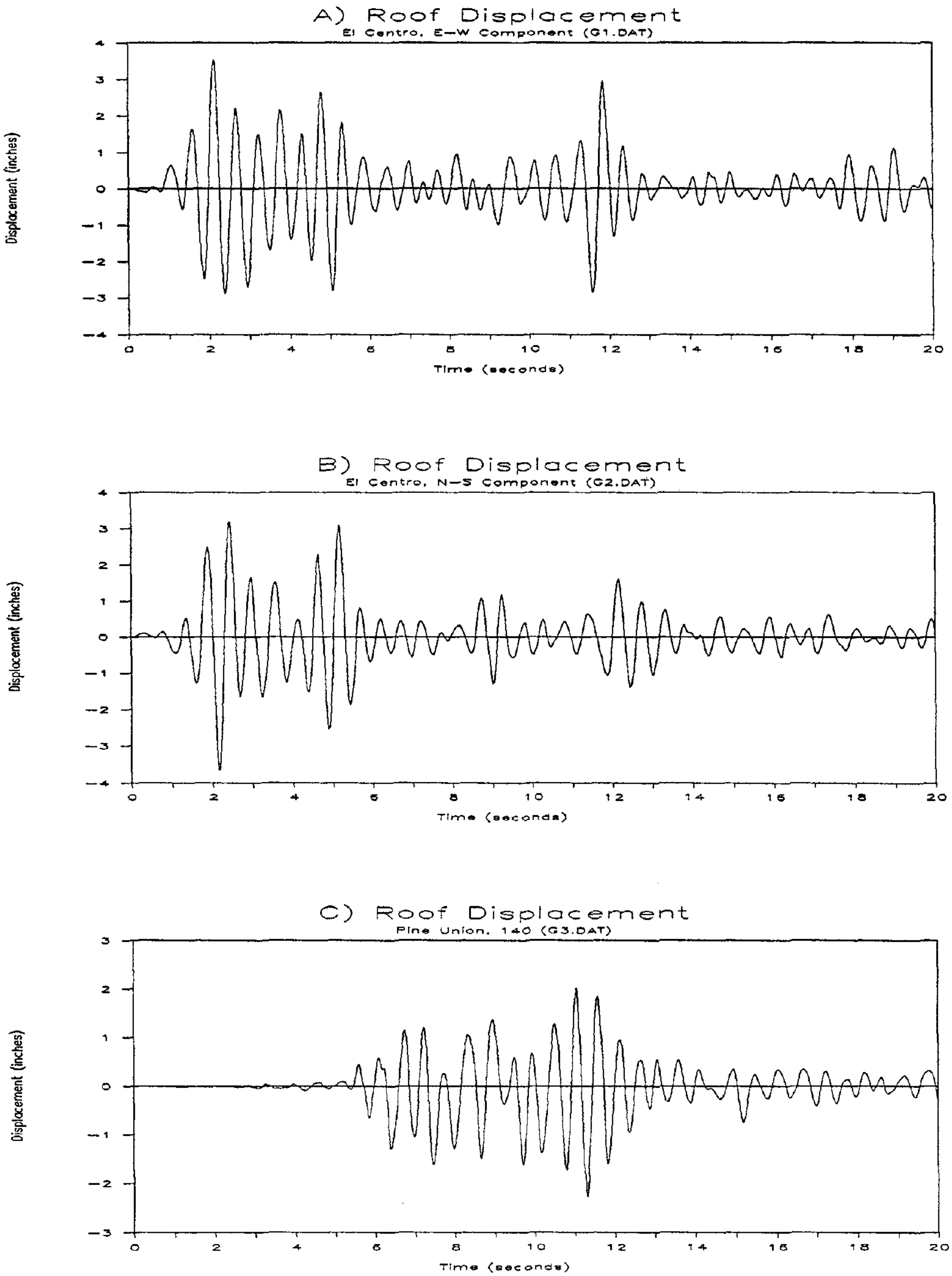
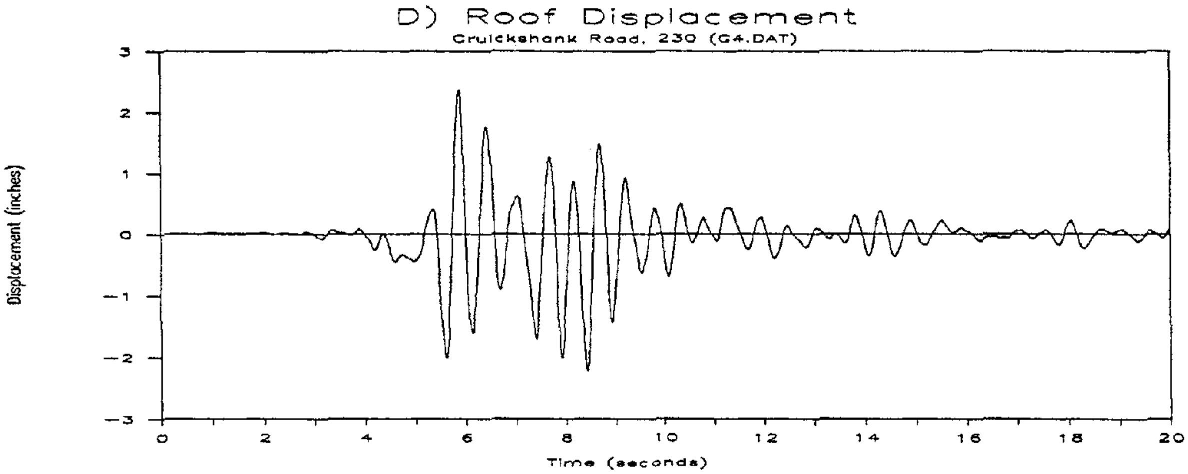
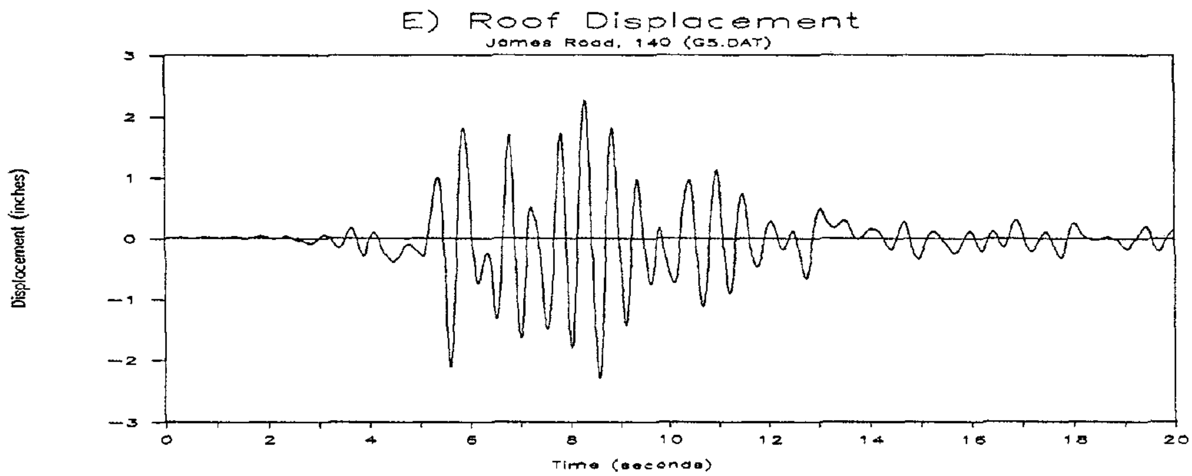


Figure 5-3 Roof Displacement of the Building





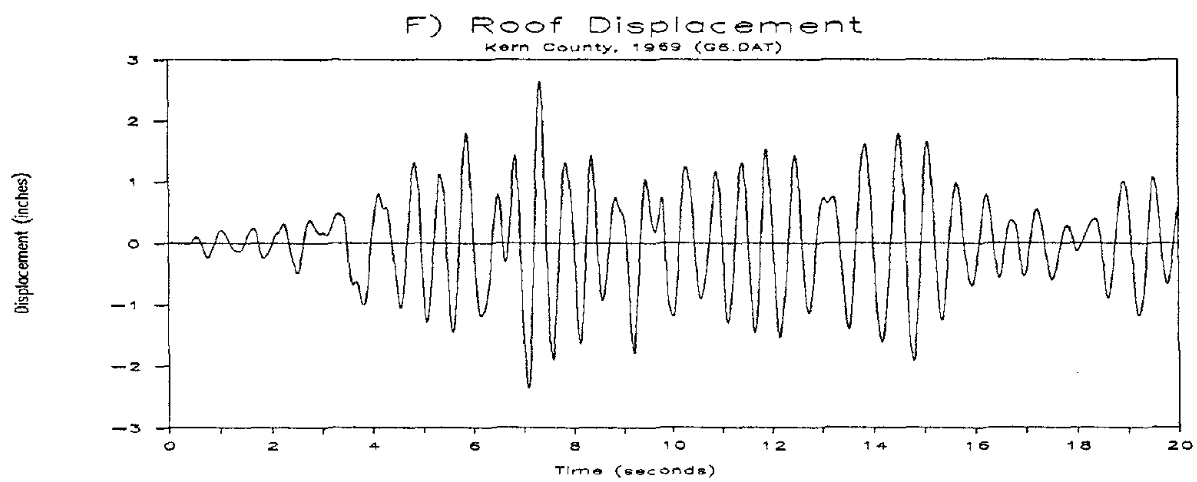
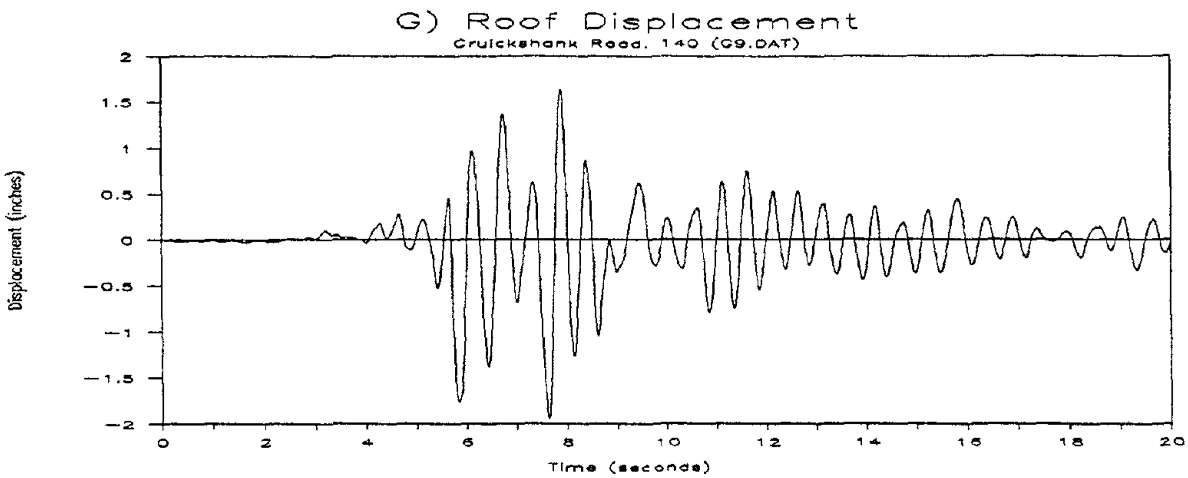
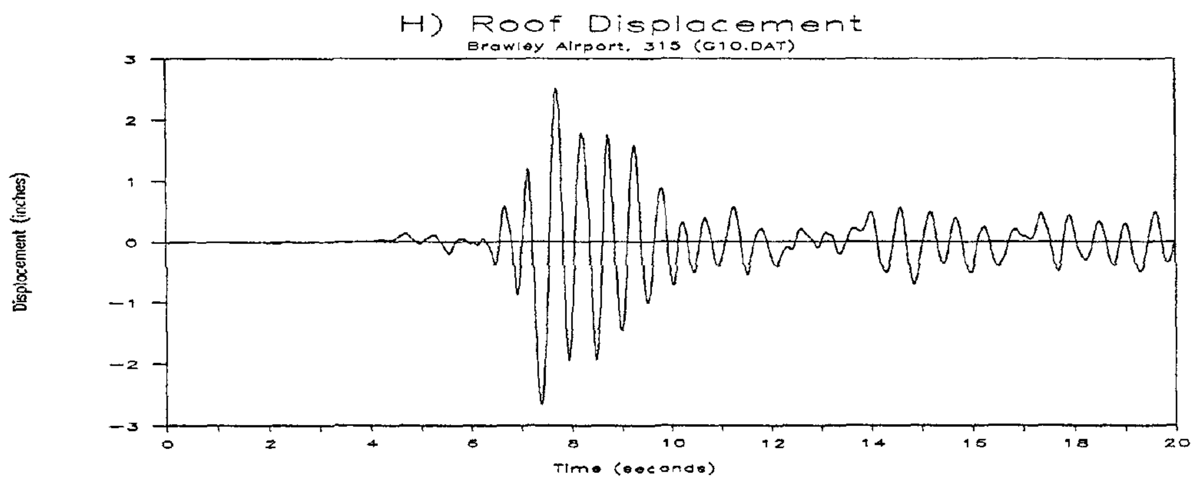


Figure 5-3 (Continued)





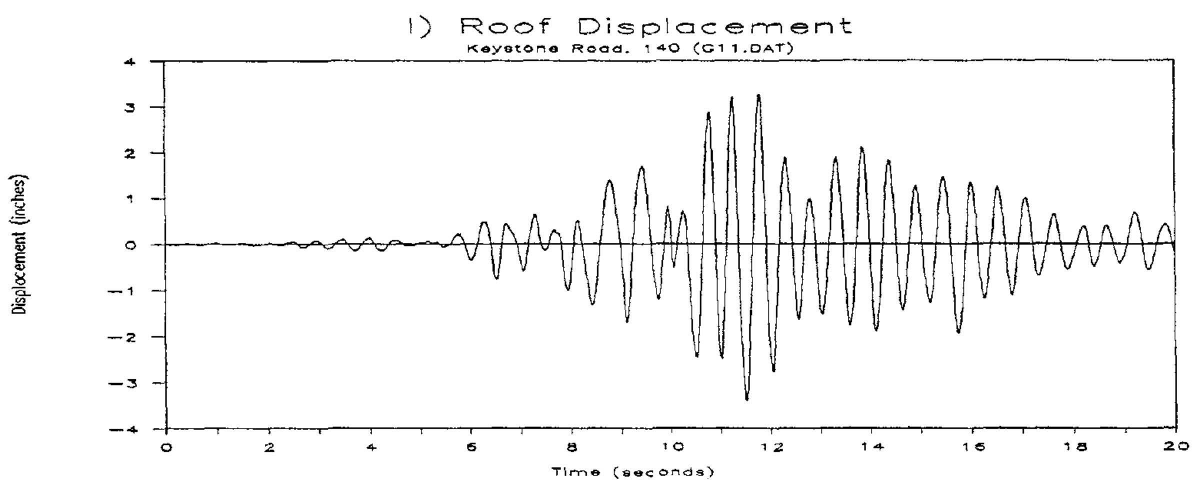


Figure 5-3 (Concluded)

A response spectrum analysis for the structure was performed. The results, as shown in Table 5-1, compared well with the average responses obtained from time history analysis.

5.4 REFERENCES

[5-1] Wilson , E.L. and Habibullah, A., "SAP 90 - A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures, User's Manual," Computers & Structures, Inc., Berkeley, California 1989.

[5-2] Paulay, T., "The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance," Earthquake Spectra, Vol. 2, No. 4, 1986, pp 783-823.

[5-3] Priestley, M.J.N. and Hart, G.C., "Design Recommendations for the period of vibration of masonry wall buildings," SSRP Report No. 89/05, University of California San Diego and University of California Los Angeles, Nov. 1989.

[5-4] Hart, G.C. and Priestley, M.J.N., "Design Recommendations for Masonry Moment-Resisting Space Frames," SSRP Report No. 89/02, University of California Los Angeles and University of California San Diego, July 1989.

[5-5] "ACI 318-89 Building Code and Commentary," American Concrete Institute, Detroit, 1989.

CHAPTER 6
EXPECTED BUILDING PERFORMANCE USING DRAIN-2DX

6.1 General

A nonlinear time history analysis of the wall frame was performed using the two-dimensional computer program DRAIN-2DX [6-1]. DRAIN-2DX is a new generation of PC-based general purpose computer program for static and dynamic analysis.

6.2 Modeling Procedure

In the DRAIN-2DX model, the structural members were represented by elastic line segments connected to nodes by bilinear springs at the ends of the member. The initial stiffness and strain hardening ratio of the bilinear spring were determined from a monotonic loading condition for flexural deformation only. Yield moments and stiffness properties for the bilinear springs at the member ends were obtained from moment curvature relations. These effective moment of inertia, as described in Section 5.2, were used in modeling the stiffness of members. To model the load reversal effect on the member, the beam initial stiffness and strain hardening ratio were determined by averaging the bilinear stiffness curves for positive and negative bending. Different yield moments were used for positive and negative bending.

The DRAIN-2DX model for the wall frame is a 2D model with one horizontal DOF at each floor level. In the model only the translational mass of the structure was considered and assumed to be lumped at the floor levels. The damping used in the model consists of two parts. One part is the viscous damping and the other is the hysteretic damping. The viscous damping is assumed to be a Rayleigh damping and can be expressed as a linear combination of the mass and initial elastic stiffness of the system. The initial stiffness is determined using the procedure discussed in the previous paragraph. Assuming the structure has 5% critical damping in its first two modes, the damping proportionality factors which are required by DRAIN-2DX, can be evaluated using the natural frequencies of vibration of the first two modes of the structure. The other part of the damping is the hysteretic damping and it is dependent on the member force-deformation relation and is implicitly accounted for by the DRAIN-2DX when the structure responds into the inelastic range.

6.3 Static Behavior State Analysis

An inelastic static behavior state analysis was performed to predict the strength of the wall frame. Such a static analysis provides information on the actual strength and the location and sequence of plastic hinge formation.

Two behavior state analyses were conducted. The first analysis used a lateral load with an inverted triangular load pattern and the second analysis used a lateral load pattern consistent with the LSDS seismic load distribution equation. The results of these analyses are shown in Figure 6-1 in terms of base shear versus roof displacement. It is noted that the strength deformation envelope is dependent on the lateral load distribution. Table 6-1 lists the computed base shear strength and the estimated nominal base shear strength from the UBC. The design base shear as computed in Chapter 3 is 18.2% of the building weight. Considering a strength reduction factor of 0.85 required by the LSDS, a nominal base shear strength of 21.4% of the weight is estimated for a code design. The computed base shear strength for the wall frame is about 48% greater than the code strength for the inverted triangular load distribution and 61% higher than the code strength for the LSDS lateral load distribution. Thus, it can be concluded that the frame strength is stronger than the minimum nominal strength resulting from the code requirements.

Figure 6-2 shows the location and the sequence of plastic hinge formation for the two frames. Figure 6-1 identifies the roof displacement corresponding to each hinge formation, i.e. each behavior state. It can be observed from Figure 6-2 that the plastic hinge formed at beam ends and at the base of the columns in a strong-column-weak-beam design fashion exactly as envisioned in the development of the design criteria.

6.4 Results of Inelastic Time History Analysis

The elastic analysis results in Section 5.3 indicated that inelastic responses should occur for both frames for all ground motions considered in this study. To evaluate the global effects of the nonlinear responses and to compare this response with the elastic results, inelastic time history analyses were conducted for the ensemble of earthquakes described in Chapter 4. The structural system responses considered in this study are base shear, floor acceleration and the relative story displacements and drift ratios. The response on the member level include plastic rotation ductility and cumulative rotation ductility. The envelopes, i.e. the maximum value in the response time history for each individual response variable were computed and used in the performance evaluation. Table 6-2 summarizes the computed structural responses of the wall frames. Table 6-3 to 6-4 show the rotation ductility and cumulative rotation ductility demands for all members that experience inelastic deformation. Figure 6-3 shows the roof displacement time history for all earthquakes.

6.5 Comparison of SAP90 and DRAIN-2DX Analyses

The computed responses from the SAP90 and DRAIN-2DX analyses for the wall frame are listed in Tables 5-1 and 6-2. Figures 6-4 to 6-9 plot the ratios of the responses obtained from these two analyses.

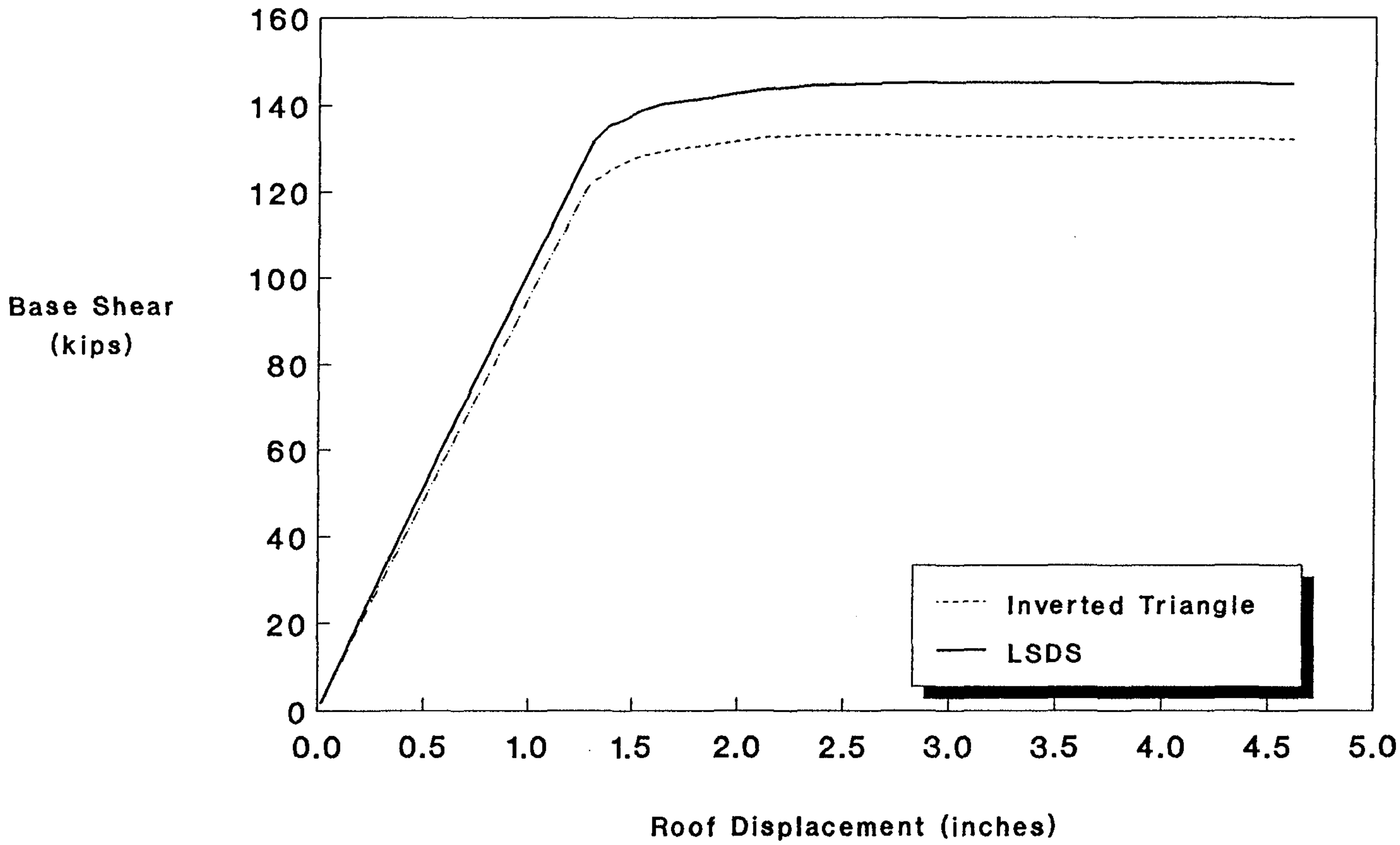
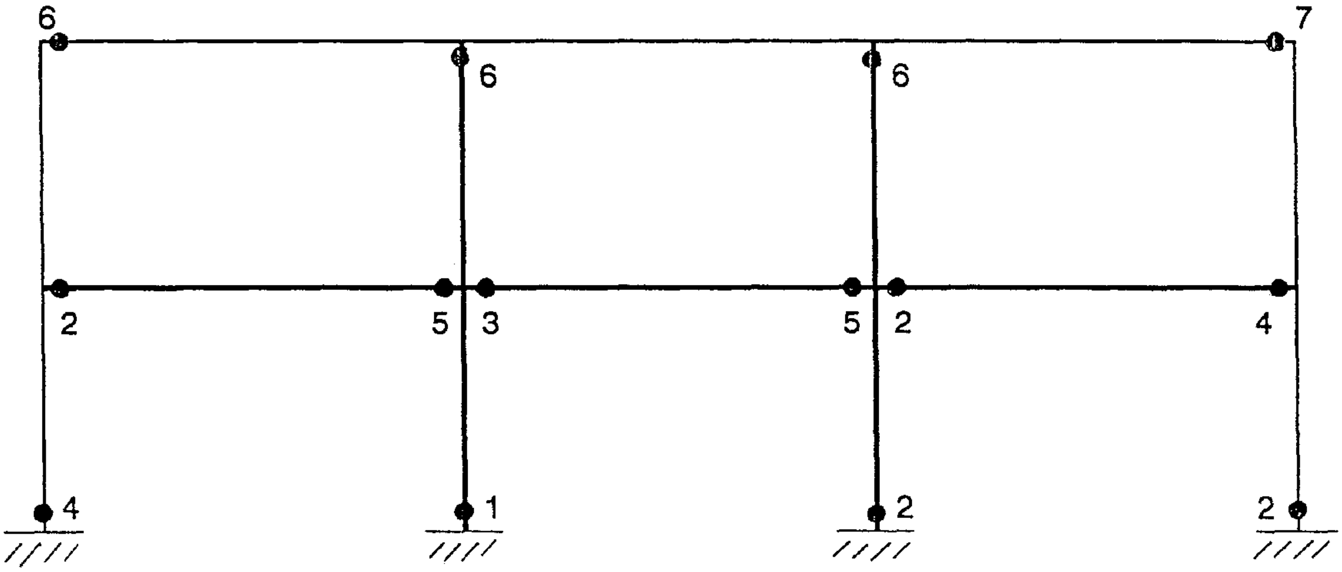


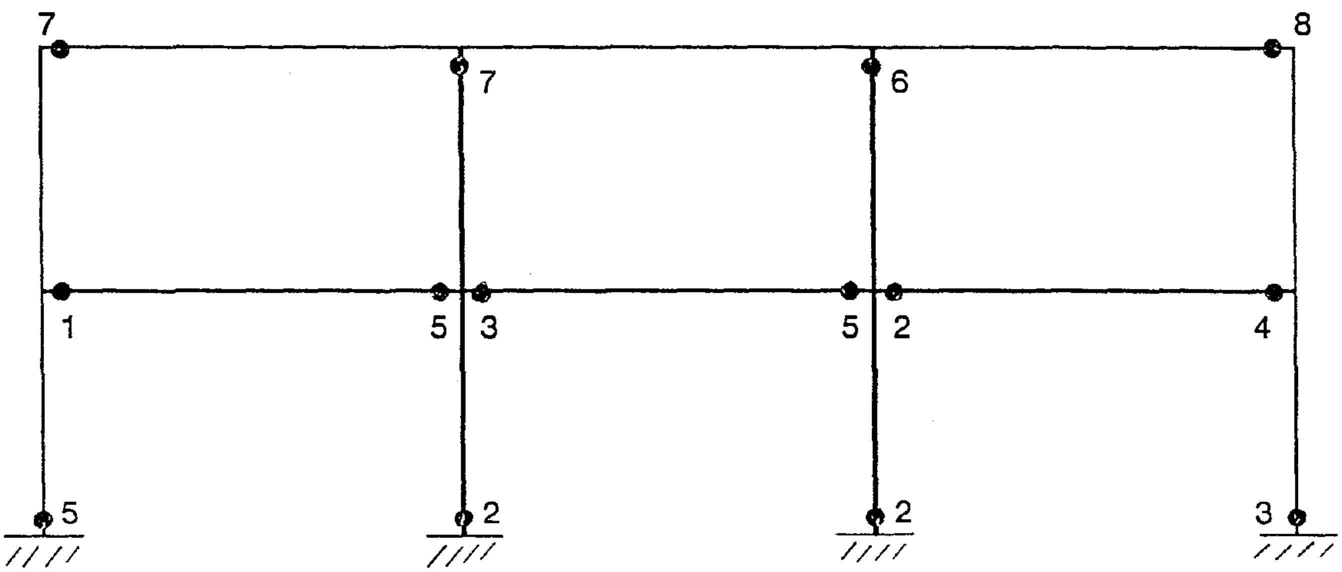
Figure 6-1 Load-Deformation Response of 8-inch Wall Frame

Table 6–1 Base Shear Strength (%W)

Wall Frame Type	8–inch
Code Nominal Strength	21.4
Inverted Triangle	31.7
LSDS Equation	34.5



Inverted Triangular Load Distribution



LSDS Lateral Force Distribution

Figure 6-2 Location and Sequence of Plastic Hinge Formations

TABLE 6-2 Inelastic Analysis Results

	Overall Drift (%)	Roof Drift (%)	Floor Drift (%)	Roof Disp. (inches)	Floor Disp. (inches)	Base Shear V/W (%)	Roof Accel. (g)	Floor Accel. (g)
Earthquakes								
G1.DAT	1.05	0.60	1.53	3.342	2.386	40.7	0.63	0.41
G2.DAT	0.89	0.54	1.27	2.833	1.983	39.8	0.67	0.43
G3.DAT	0.90	0.56	1.25	2.871	1.943	37.9	0.60	0.44
G4.DAT	0.81	0.61	1.09	2.577	1.694	38.3	0.57	0.39
G5.DAT	1.43	1.09	1.78	4.548	2.782	39.0	0.81	0.41
G6.DAT	0.74	0.50	1.02	2.342	1.592	39.0	0.72	0.51
G9.DAT	1.03	0.66	1.41	3.268	2.207	40.6	0.71	0.51
G10.DAT	1.23	0.66	1.81	3.903	2.822	41.1	0.64	0.41
G11.DAT	1.06	0.60	1.56	3.383	2.440	40.8	0.62	0.50
Minimum	0.74	0.50	1.02	2.342	1.592	37.9	0.57	0.39
Maximum	1.43	1.09	1.81	4.548	2.822	41.1	0.81	0.51
Average	1.02	0.65	1.41	3.230	2.205	39.7	0.66	0.45
C.O.V. (%)	19.9	25.4	18.9	19.9	18.9	2.8	10.4	10.2

Table 6-3 Maximum Plastic Hinge Rotation Ductility

	Base Ext. Column	Base Int. Column	Roof Int. Column	Floor Ext. Bm	Floor Int. Bm	Roof Ext. Bm
Earthquakes						
G1.DAT	4.89	5.70	4.89	2.20	1.77	1.04
G2.DAT	3.69	3.69	3.69	1.94	1.61	1.09
G3.DAT	3.48	3.48	3.48	2.03	1.61	1.18
G4.DAT	2.98	2.98	2.98	1.87	1.51	1.22
G5.DAT	4.95	4.95	4.95	3.24	2.93	2.57
G6.DAT	2.95	2.95	2.95	1.63	1.39	1.07
G9.DAT	4.13	4.13	4.13	2.06	1.69	1.51
G10.DAT	5.32	5.32	5.32	2.37	1.93	1.13
G11.DAT	4.38	4.38	4.38	2.10	1.68	1.11

Table 6-4 Maximum Accumulated Rotation Ductility

	Base Ext. Column	Base Int. Column	Roof Int. Column	Floor Ext. Bm	Floor Int. Bm	Roof Ext. Bm
Earthquakes						
G1.DAT	11.79	17.57	1.01	4.22	2.80	1.04
G2.DAT	9.40	14.05	1.09	3.75	2.46	1.09
G3.DAT	7.41	12.58	1.29	2.66	1.61	1.18
G4.DAT	7.40	11.02	1.31	3.28	2.24	1.22
G5.DAT	7.21	11.24	3.49	3.86	2.93	2.80
G6.DAT	9.70	18.14	1.00	2.87	1.45	1.07
G9.DAT	6.70	9.66	1.51	2.98	2.10	1.51
G10.DAT	5.32	6.64	1.30	2.37	1.93	1.13
G11.DAT	10.35	17.52	1.05	3.82	2.22	1.11

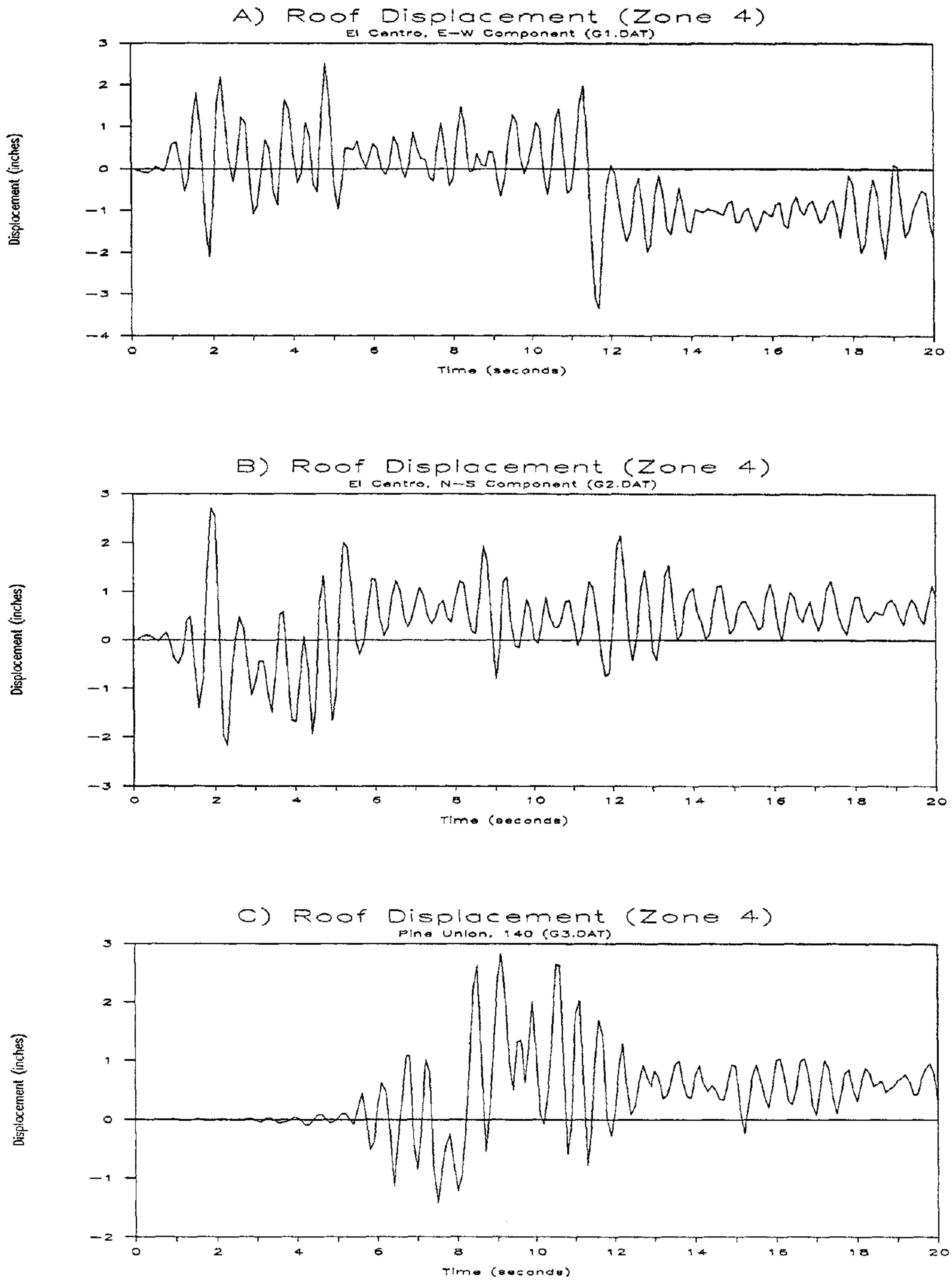
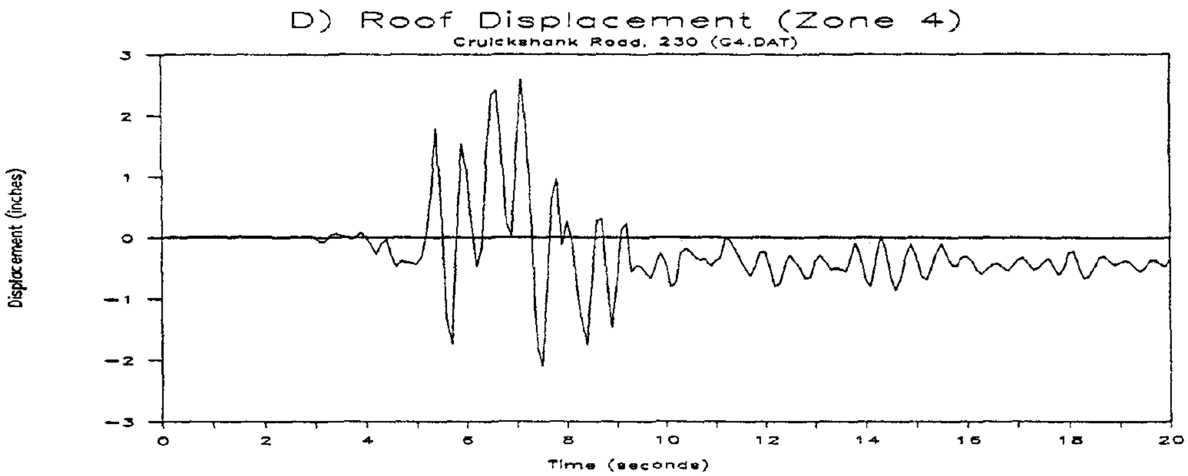
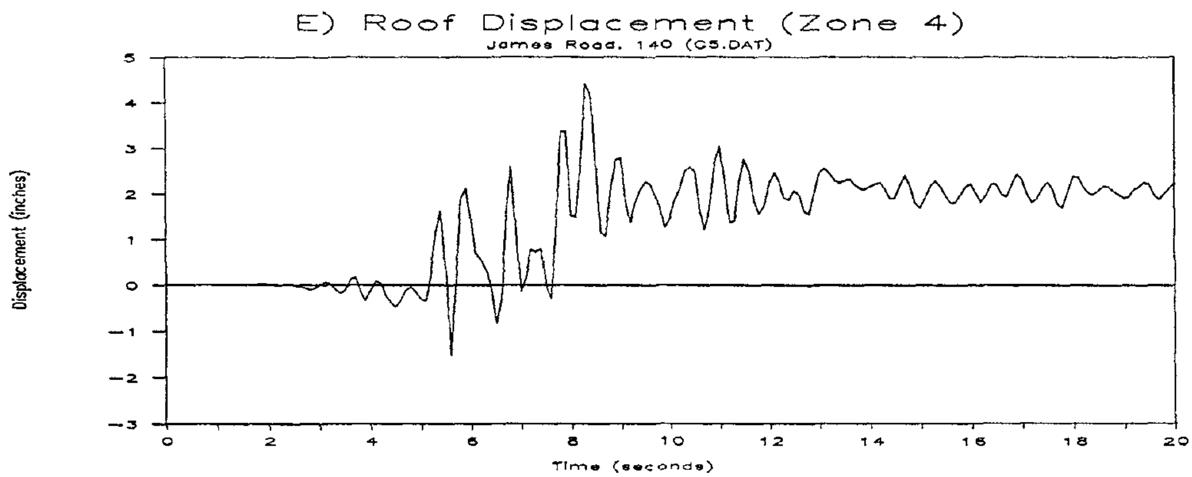


Figure 6-3 Roof Displacement of the Building





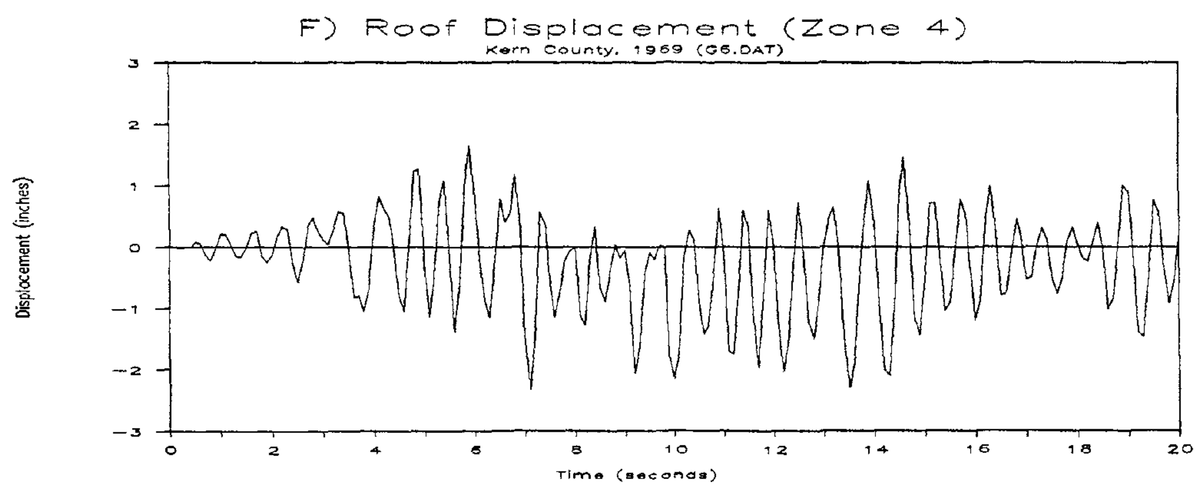


Figure 6-3 (Continued)

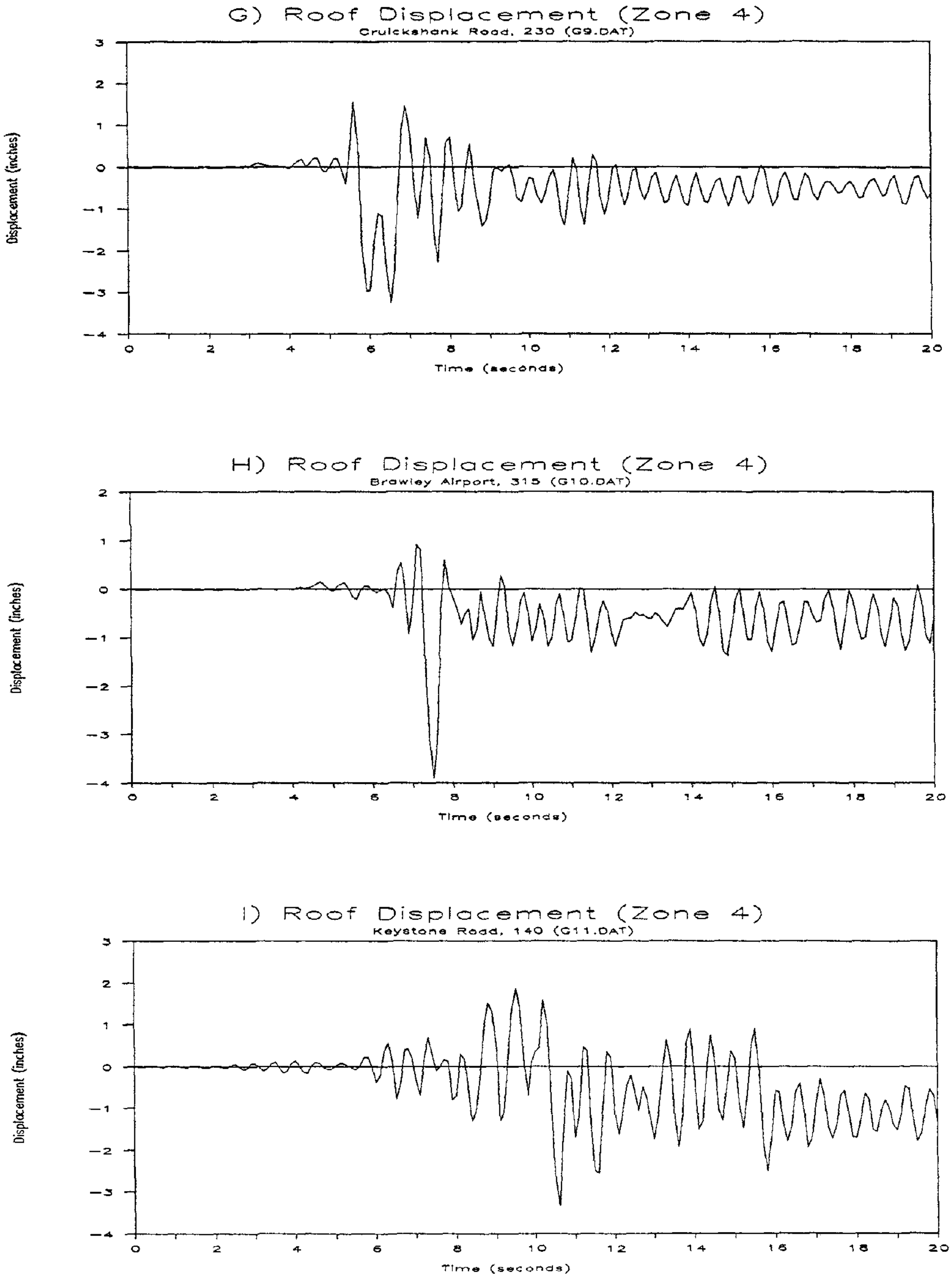


Figure 6-3 (Concluded)

Figure 6-4 Base Shear

Comparison between SAP90 and DRAIN-2DX

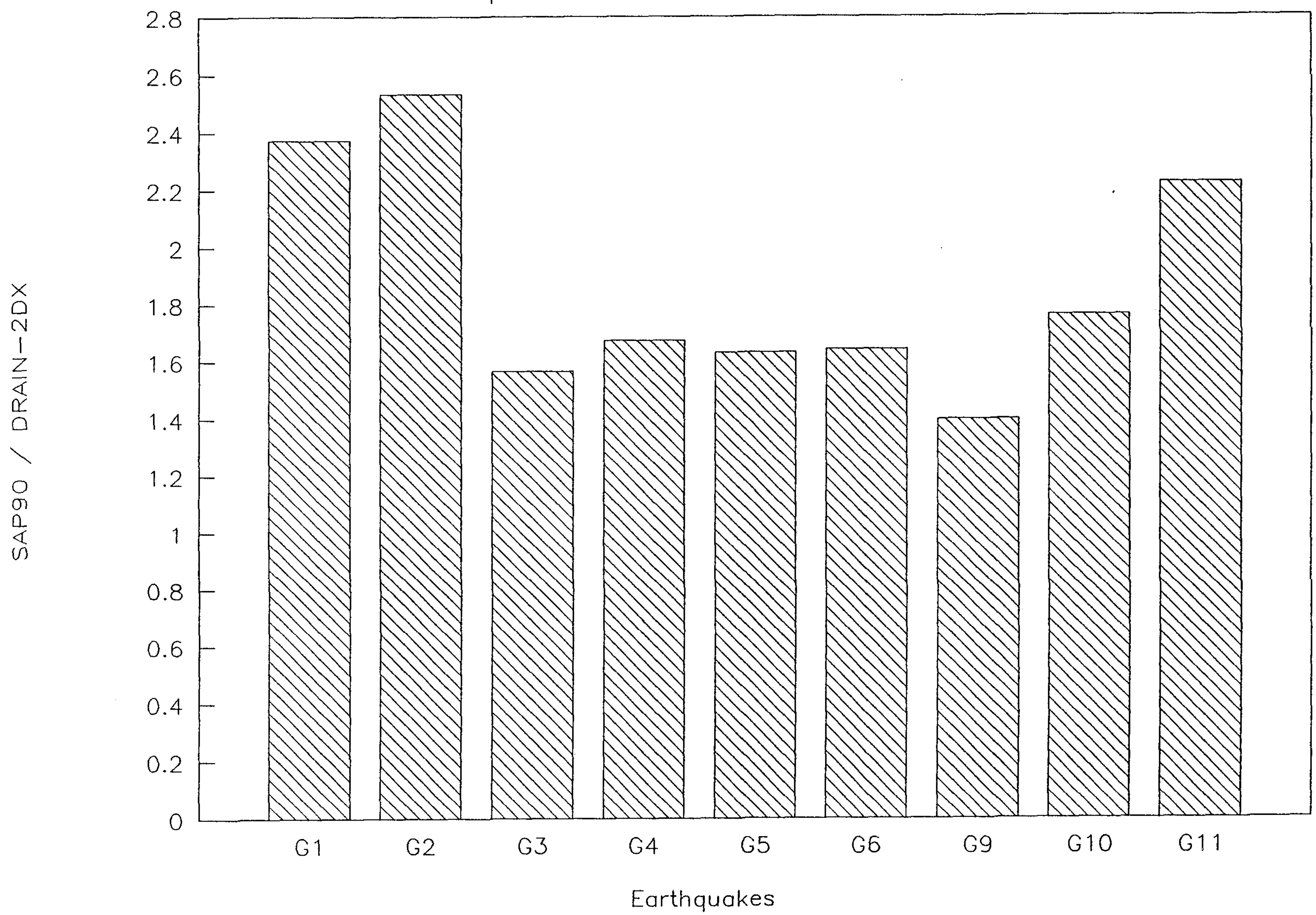


Figure 6-5 Roof Acceleration

Comparison between SAP90 and DRAIN-2DX

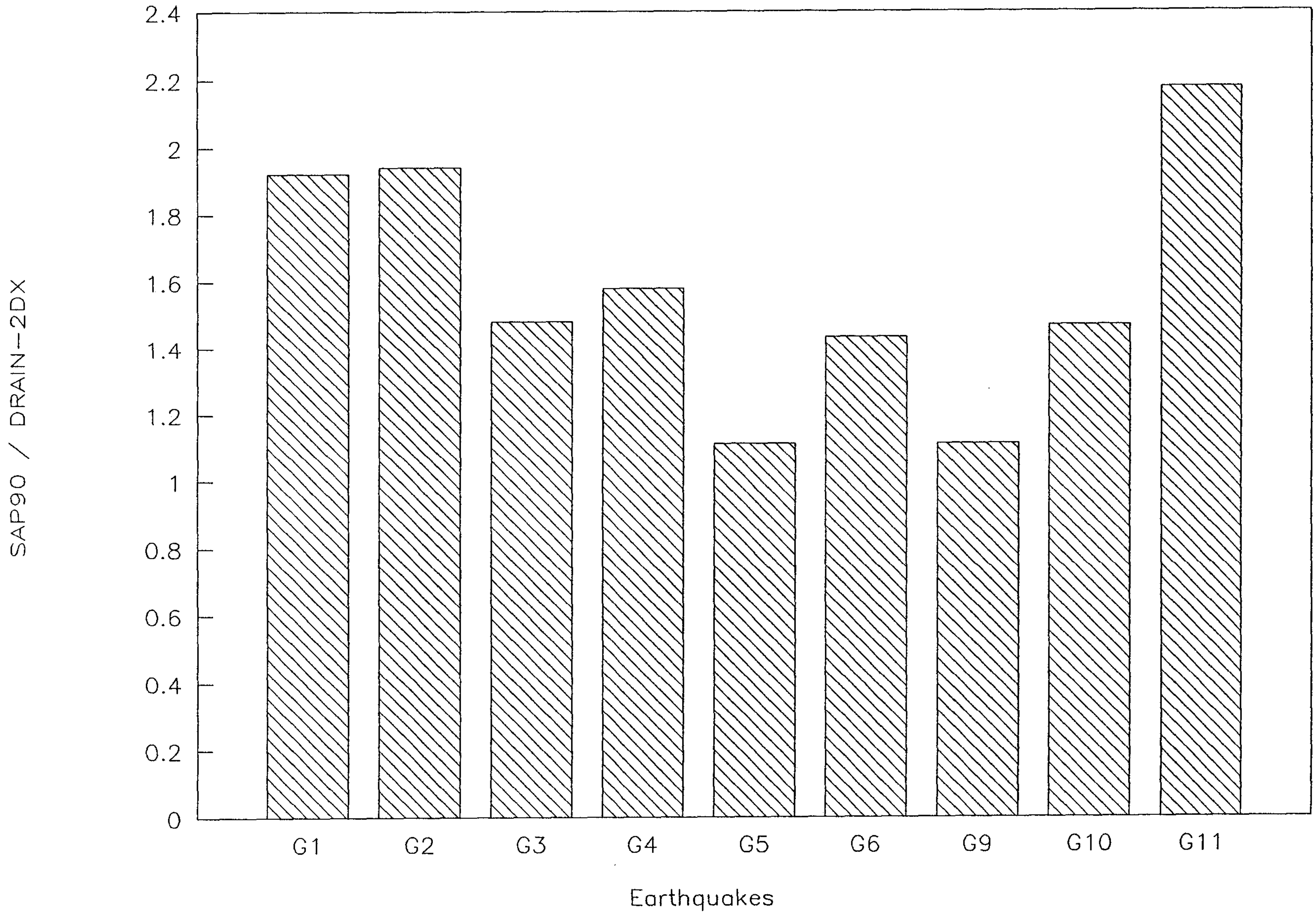


Figure 6-6 Floor Acceleration

Comparison between SAP90 and DRAIN-2DX

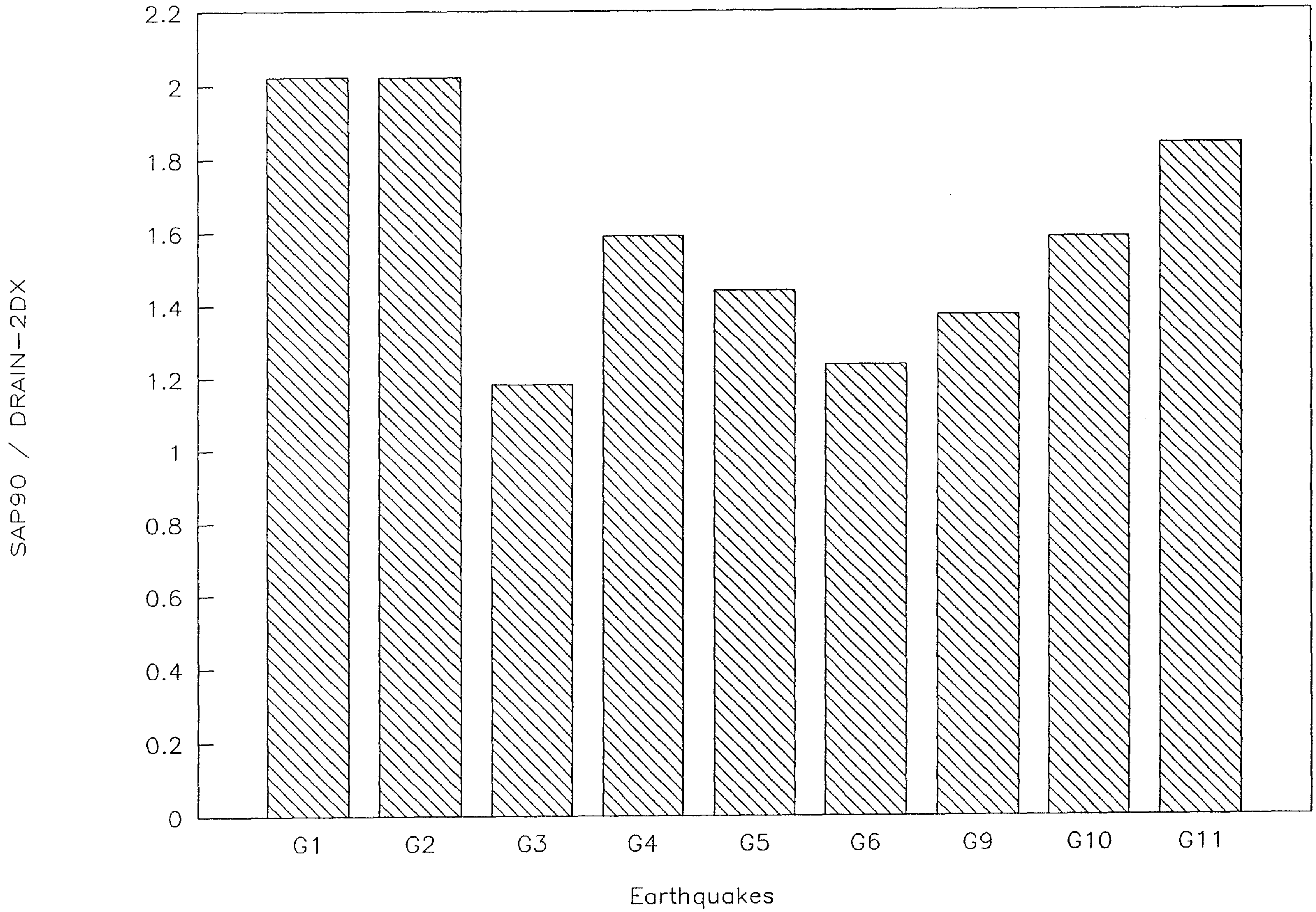


Figure 6-7 Overall Drift

Comparison between SAP90 and DRAIN-2DX

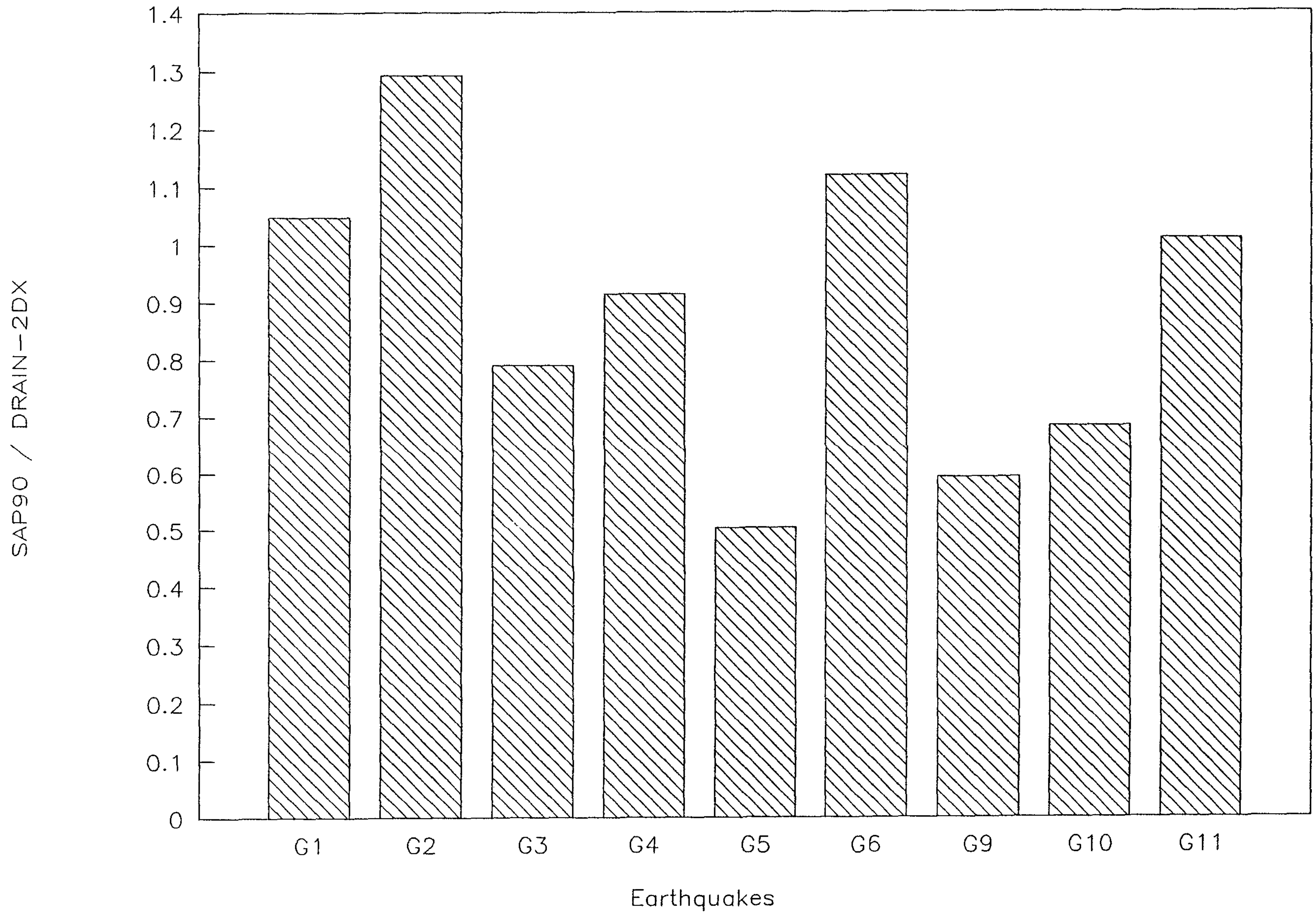


Figure 6-8 Roof Drift

Comparison between SAP90 and DRAIN-2DX

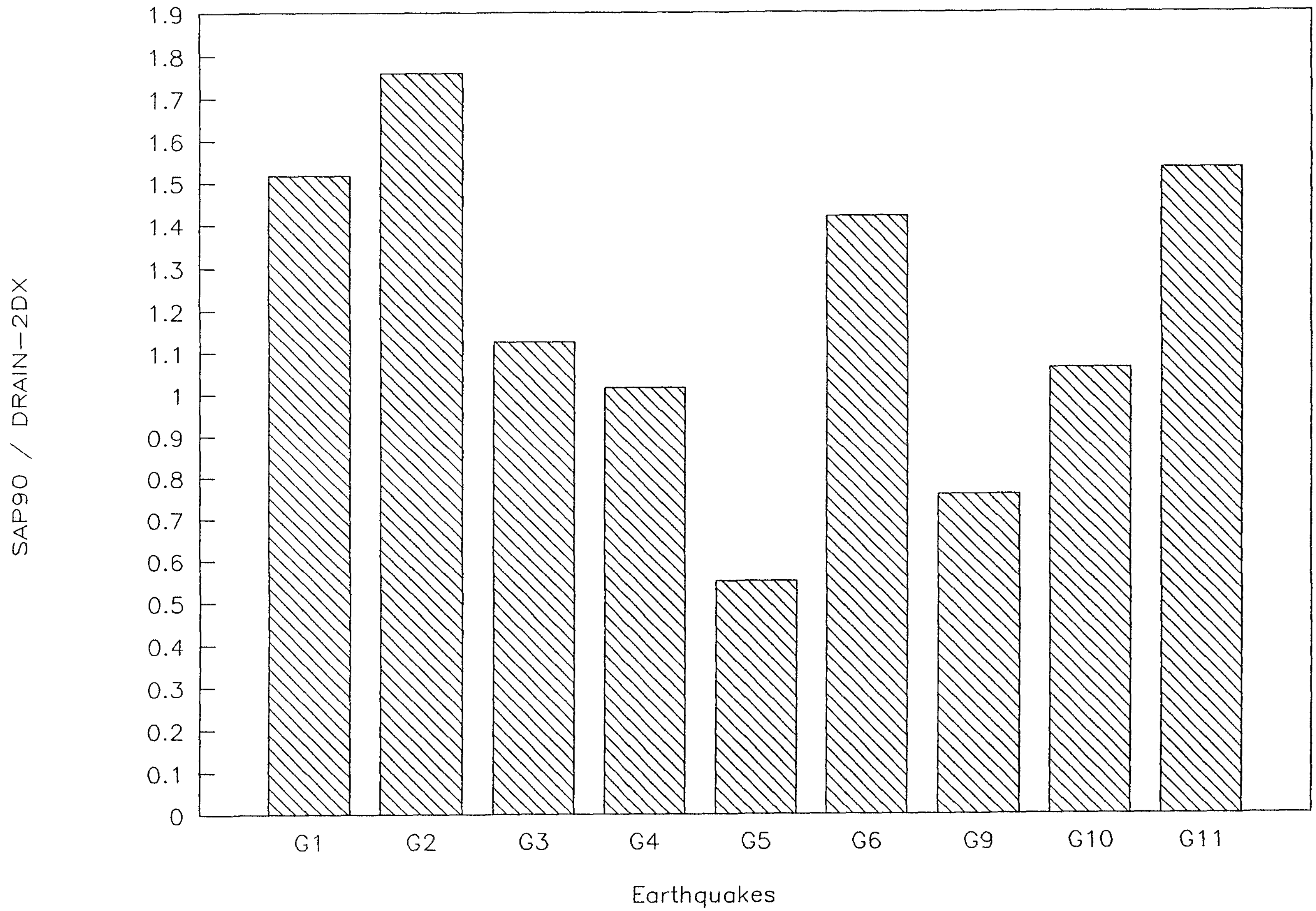
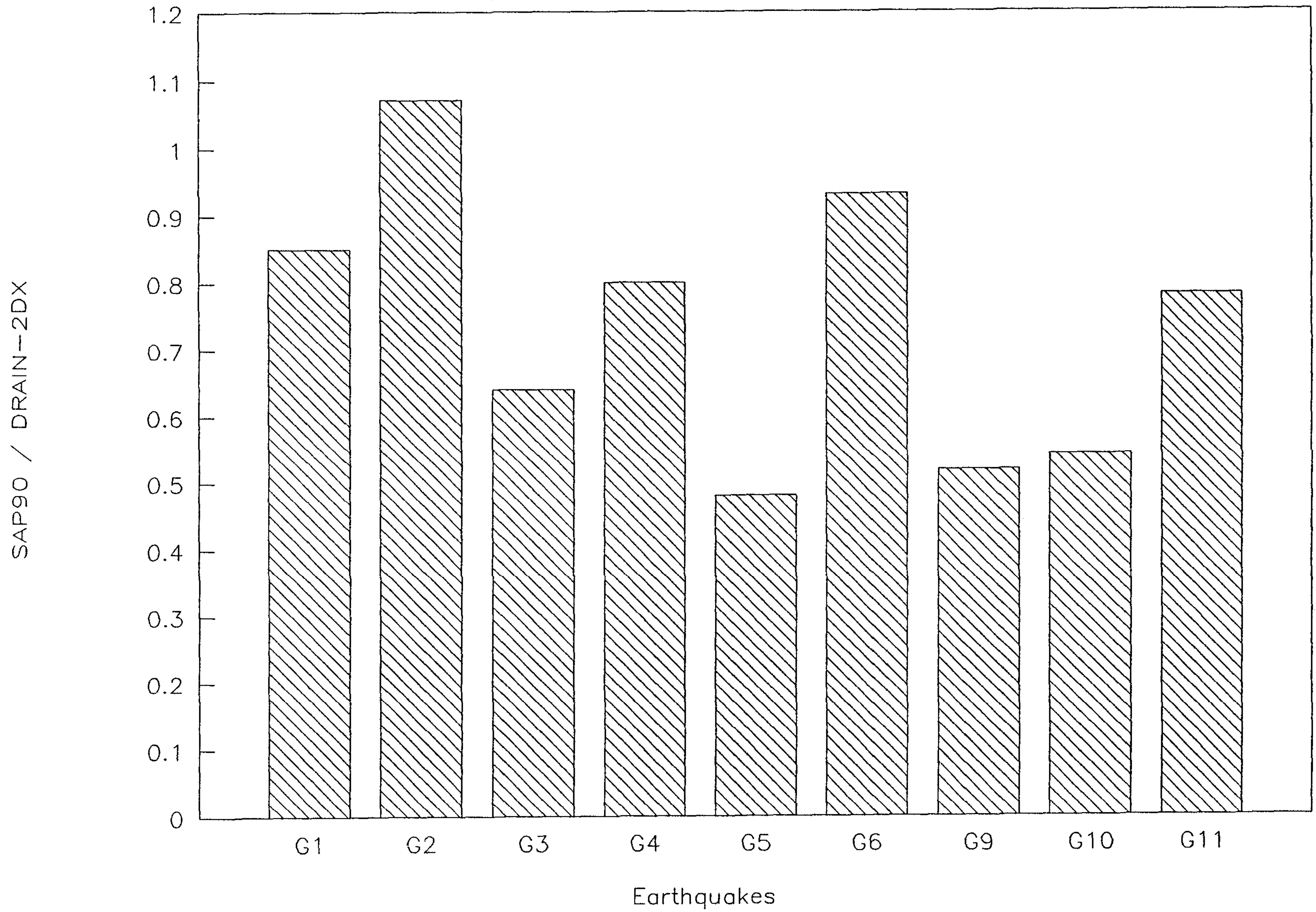


Figure 6-9 Floor Drift

Comparison between SAP90 and DRAIN-2DX



The ratio of maximum base shear from an elastic analysis and that from an inelastic analysis as shown in Figure 6-4 can be used to provide insight into reasonable design values for response modification factors, i.e. R as defined in the NEHRP document. The R factor can be related to the key design parameter R_w stipulated in the 1991 UBC code as R_w = 1.65 R. It has been indicated that the R factor is an empirical variable and can be determined based on both past performance of structural system and collective experience of code committee. It was also recognized that the R factor is dependent on the various parameters such as damping and ductility in the structural system and the earthquake - structure period ratio [6-2]. As can be seen from Figure 6-4, the SAP90/DRAIN-2DX ratio varies with the level of earthquake with an approximate value of 1.5

It can be shown that the ratio of elastic analysis to inelastic analysis for the roof acceleration as shown in Figure 6-5 is approximately equivalent to the base shear ratio. Figure 6-6 shows the ratio of two analyses for the floor acceleration. The ratio varies with increasing earthquake intensity level.

Figures 6-7 to 6-9 compare the relative displacement for elastic and inelastic analysis. It is implicitly implied in the equal displacement design criteria approach that the displacement for the two analyses are identical. However, the ratios of displacements from the two analyses significantly deviate from the unity and can be seen to vary between 0.5 and 1.5.

6.6 References

[6-1] Allahabadi, R. and Powell, G.H., "Drain-2DX - Seismic Response and Damage Assessment for 2D Structures," Ph.D Dissertation, University of California, Berkeley, California 1987.

[6-2] Hwang, H. and Jaw, J.W., "Statistical Evaluation of Response Modification Factors for Reinforced Concrete Structures," Technical Report NCEER-89-0002, National Center for Earthquake Engineering Research, SUNY, Buffalo, New York, December 1988.

CHAPTER 7
EXPECTED BUILDING PERFORMANCE USING SCM

7.1 General

An analytical model, called a Structural Component Model (SCM), has been developed for the nonlinear analysis of masonry structures. This model can be viewed as a substructure analysis approach in which the structure is modeled with a few macro elastic and inelastic elements. In the analytical modeling of a reinforced masonry structure, the finite element approach (FEM) is known to provide a better understanding of inplane response and can reasonably replicate the force-displacement envelope obtained from the experiment work. However, the SCM requires less computational efforts than a FEM method and yet, as will be shown, still is capable of simulating the nonlinear responses of reinforced masonry members.

7.2 Modeling Procedure

Conceptually, the SCM is a modification of the concentrated spring model. Instead of the nonlinear rotation spring concentrated at the end of the member, the inelastic elements are chosen at the region of flexural yielding over a finite plastic hinge length L_p where the nonlinear action is expected . The inelastic element is then connected to elastic elements. The SCM uses two dimensional beam, column, and panel elements to model the inelastic and elastic elements. Only the material nonlinearity is considered and confined to the region of the plastic hinge length L_p, which is characterized by an inelastic element. The basic behavior of the inelastic element is captured from the moment curvature relation reflecting various stages of behavior of the masonry and the reinforcing steel. The change in stiffness between limit states of the inelastic element is taken into account as reflected in the moment curvature relationship. Detailed formation on the SCM is described elsewhere [7-1,7-2].

Figure 7-1 shows the structural idealization of the wall frame using the SCM approach where three types of elements, viz. beam, column and panel elements are utilized to model the structure. It can be seen from this figure that the beams and columns are represented by two identical elements in order to satisfy the compatibility requirement at the corners of the panel element. The SCM model for a nonlinear analysis requires a knowledge of the locations where the inelastic action is expected. We first assumed that plastic hinges may form in all members in the structure. Then, we reviewed the responses to determine whether or not the member has yielded at the locations we selected. After a few cycles of iteration, it was determined that the plastic hinges formed at the beam ends and at the base of columns as indicated by the shaded zones shown in Figure 7-1. The plastic hinge length is assumed to be one-half of the section depth. The inelastic elements were then chosen to model the plastic hinge and the corresponding nonlinear behavior was characterized by the moment curvature relation

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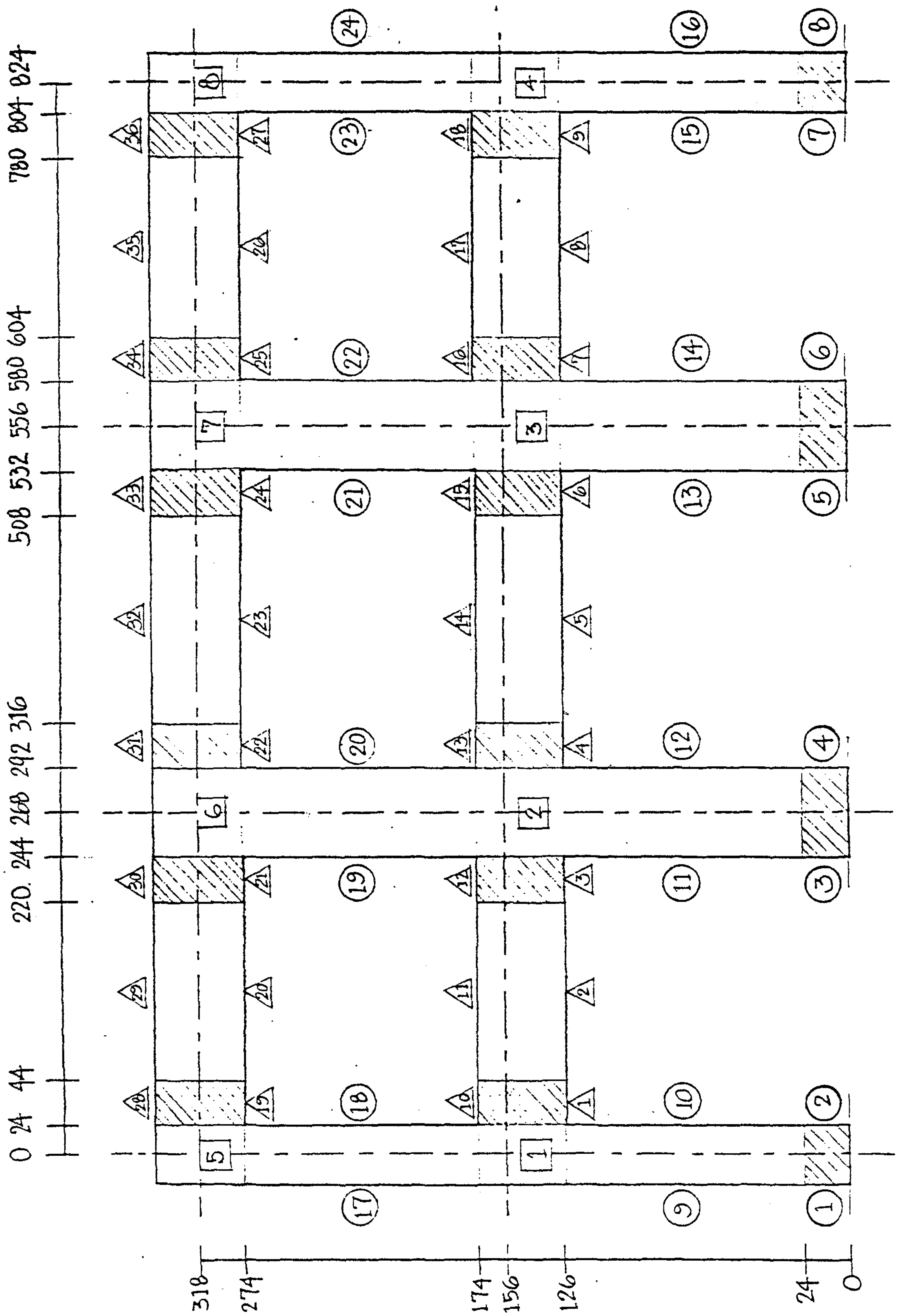


Figure 7–1 SCM Analytical Model

○ : Column
△ : Beam
□ : Panel

expected at the hinge region. The realistic moment curvatures were developed considering the expected axial force level, stress-strain relation of concrete masonry and the reinforcing steel. These curves were in turn represented by the equivalent bilinear curves following the procedure described in Chapter 5 to establish the graduate stiffness change in the structural members.

7.3 Results of SCM Analysis

The response of wall the frame when it was subjected to a monotonically increasing displacement at the roof is characterized by a base shear - roof displacement curve. Figure 7-2 shows the load-deformation curve for the structure under static lateral load having an inverted triangular load pattern. The solid line shown in Figure 7-2 represents the SCM analysis results and the dotted line was obtained from the DRAIN-2DX static behavior state analysis as presented in Section 6.3. Comparison of the two curves indicates that the general trend of load-deformation curve can be captured fairly well by the SCM analysis. Note that the two curves terminated at the roof displacement of 4 inches, corresponding to 1.23% drift ratio, because this is the maximum drift ratio the structure undergone under the ensemble of earthquakes. The correlation between the SCM and DRAIN-2DX analysis is established through the comparison of base shear at the maximum roof displacement. Considering a typical earthquake, say 1940 El Centro earthquake (G2.DAT), the maximum roof displacement obtained from an inelastic time history analysis is about 2.8 inches and the corresponding base shear is 165 kips. For the same magnitude of roof displacement the SCM analysis would give the base shear of 150 kips. It is noted that one significant reason for the disparity of the results from the two analyses is the uncertainty in the determination of static lateral load used as equivalent earthquake loading.

7.4 References

[7-1] Hart, G.C., Low, Y.K., Jaw, J.W., and Englekirk, R.E., "SCM Model for University of Colorado Flexural Walls," Technical Report, Englekirk & Hart Consulting Engineers, Inc., Dec. 1989.

[7-2] Hart, G.C. and Jaw, J.W., "A SCM Model for Masonry Shear Walls," Proceedings, Sixth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, Seattle, Washington, August 1990.

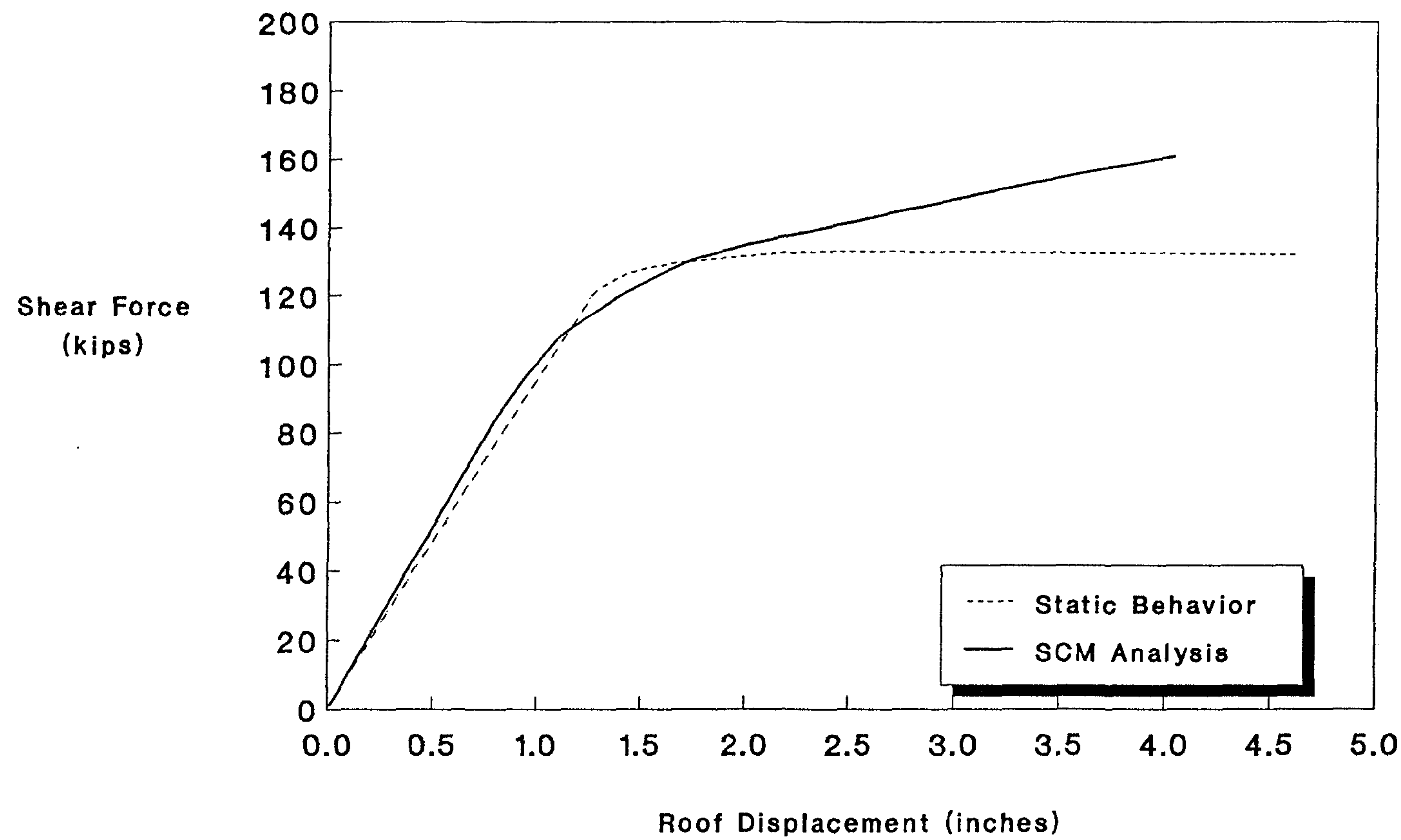


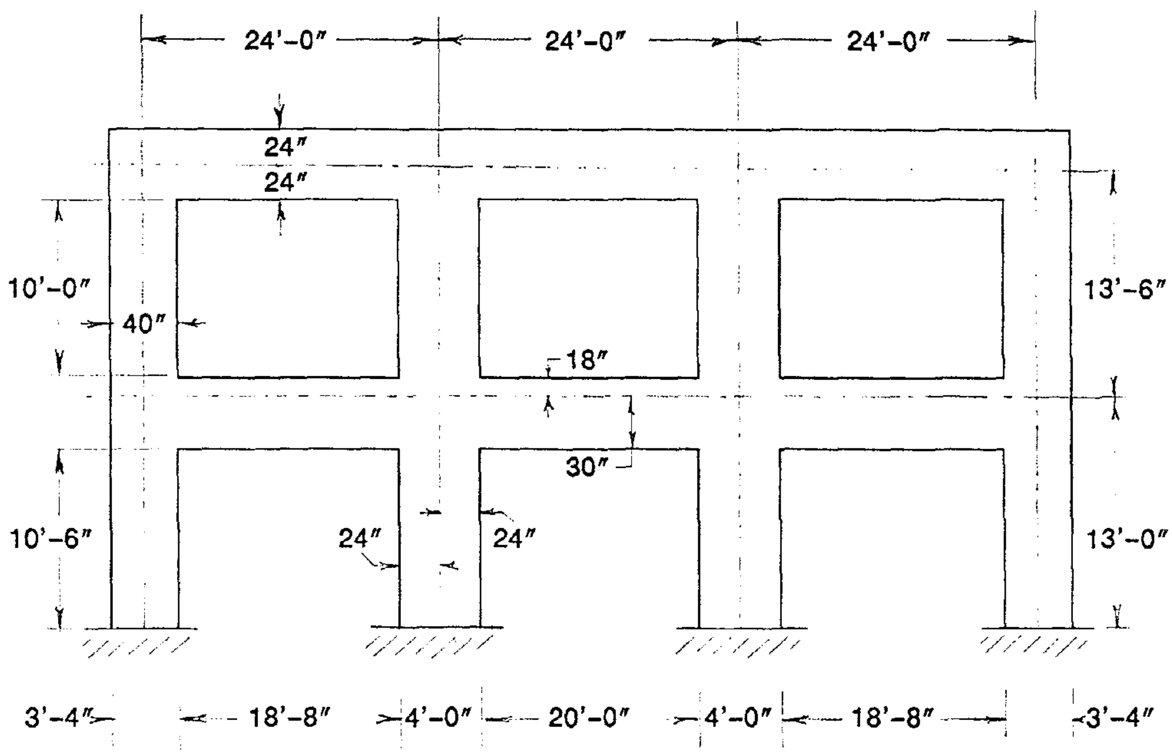
Figure 7-2 Load-Deformation Response of 8-inch Wall Frame

APPENDIX A
MOMENT RESISTING WALL FRAME DESIGN CALCULATIONS

Please refer to the attached calculations sheets for a complete description of the design process.

Moment Resisting Wall Frame Design

8″ Wall Frame



<u>Loading Criteria</u>

Roof:	DL:	Roofing	6.0 psf
		1/2" Plywood	1.5
		Insulation	2.0
		Framing (@24" o.c.)	3.0
		Ceiling	2.5
		Mech.	2.5
		Misc.	1.0
			18.5 psf
		Beams	2.0
			20.5 psf
	LL:	20/16/12 psf	

Floor:	DL:	Flooring	1.0 psf
		1-1/2" Lightweight Concrete Fill	14.0
		3/4" Plywood	2.3
		Framing (@16" o.c.)	4.0
		Ceiling	2.5
		Mech.	2.0
		Misc.	1.2
			27.0 psf
		Beams	3.0
			30.0
	LL:	80 psf (Reducible)	

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LATERAL LOAD DESIGN

ROOF: DL = 20.5
10.0 PARTITIONS
30.5 PSF

WALL: N, S & E WALLS = 84 PSF × 9.75' = 819 #/, (8" CMU)

W WALLS = (14.67 × 5/72 + 4)(84) = 422 #/,

FLOOR: DL = 30.0
20.0 PARTITIONS
50.0 PSF

WALL: N, S & E WALLS = 84 × 13.25 = 1113 #/,

W WALLS = (14.67 × 12.25/72 + 4)(84) = 511 #/,

ROOF WT = 30.5(5760) + 819(80+80) + (422)(72) = 337 k
FLOOR WT = 50.0(5760) + 1113(80+80) + (511)(72) = 503 k
BLDG WT W = 840 k

SEISMIC LOAD (1991 NEHRP RECOMMENDED PROVISION):

BASE SHEAR COEFFICIENT $C_s = V/W = \dfrac{S_{a(1.0)} S}{R\, T_{es}^{n}} \le \dfrac{S_{a(0.3)}}{R}$

FOR SEISMIC ZONE 4: $S_{a(1.0)} = 0.58$, $S_{a(0.3)} = 1.0$

FOR S_2 SOIL: $S = 1.0$

WALL FRAME: $R = 5\frac{1}{2}$, $C_d = 5\frac{1}{2}$

ASSUME 25% OF UNCRACKED MOMENT OF INERTIA FOR BEAMS & 50% FOR COLUMNS, WE HAVE

$T_{es} = 0.45$ SEC; $n = 1$

$C_s = \dfrac{(0.58)(1.0)}{(5.5)(0.45)} = 0.234 > \dfrac{1.0}{5.5} = 0.182$ USE $C_s = 0.182$

$V = 0.182(840) = 153^k$

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VERTICAL DISTRIBUTION OF SEISMIC FORCES:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i}$$ FOR $T_{es} \le 0.5\,s$, $k=1$; $T_{es} \ge 2.5\,s$, $k=2$

$0.5 \le T_{es} \le 2.5$ BY LINEAR INTERPOLATION

	w_x	h_x	$w_x h_x$	$w_x h_x/\Sigma w_i h_i$
ROOF	337	26.5	8931	$0.577 \times 153/2 = 44.2^k$
FLOOR	503	13.0	6539	$0.423 \times 153/2 = 32.4^k$
	840		15470	

STATIC ANALYSIS OF WALL FRAME WITH ASSUMED EFFECTIVE MOMENT OF INERTIA FOR MEMBERS SUBJECTED TO DESIGN LATERAL FORCE F_x, DEAD AND LIVE LOADS USING COMPUTER PROGRAM SAP90.

ELEMENT PROPERTIES:

$f_{me} = 1950$ psi, $E_m = 550 f_{me} = 1073$ ksi, $f_{ye} = 66$ ksi, $E_{se} = 29000$ ksi

BEAMS: ROOF $A = 366\ \text{IN}^2$, $A_v = 305\ \text{IN}^2$, $I_{eff} = 17568\ \text{IN}^4$

FLOOR $A = 366\ \text{IN}^2$, $A_v = 305\ \text{IN}^2$, $I_{eff} = 17568\ \text{IN}^4$

PIERS: EXT $A = 305\ \text{IN}^2$, $A_v = 254.2\ \text{IN}^2$, $I_{eff} = 20333\ \text{IN}^4$

INT $A = 366\ \text{IN}^2$, $A_v = 305\ \text{IN}^2$, $I_{eff} = 35136\ \text{IN}^4$

LOADS:

FLOOR: $P_{DL} = 5.0^k$, $P_{LL} = 16.8^k$, $P_E = 32.4^k$

ROOF: $P_{DL} = 3.5^k$, $P_{LL} = 2.7^k$, $P_E = 44.2^k$

RESULTS OF ANALYSIS ARE USED TO DESIGN MEMBERS.

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CHECK LIMITS ON MEMBER SIZES

BEAMS: WIDTH $t_b = 8"$
DEPTH $h_b = 48"$
CLEAR SPAN $L_{CLR} = 18.67'$ OR $20'$

8.5.4.5.3 : $h_b \le L_{CLR}/4 = 56"$ ok

8.5.4.5.4 : $h_b \geq 4$ UNITS ok

$h_b \geq 32"$ ok

$\frac{h_b}{t_b} \le 4 \Rightarrow \frac{48}{8} = 6 \not\le 4$ NG

8.5.4.5.5 $t_b \geq 8"$ ok

$t_b \geq \frac{1}{26} L_{CLR} \Rightarrow 8 \not\geq 9.23$ NG

PIERS: WIDTH $t_p = 8"$
DEPTH $h_p = 40"$ OR $48"$
CLEAR HEIGHT $h_{CLR} = 10'$ OR $10.5'$

8.5.4.6.2 $h_p \geq 2$ UNITS ok

$h_p \geq 24"$ ok

$t_p \geq 8"$ ok

$t_p \geq 1/14\, L_{CLR} = 8.57"$ NG

$P_u \le 0.3 A_n f'_m$
$P_u \le 0.6 P_b$ SEE COLUMN CALCULATION

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BEAM DESIGN

$M_u^+ = 1066''\text{-}k = 88.8'\text{-}k$

FLOOR BEAMS: EXT: $M_E = 1118''\text{-}K$, $M_{DL} = 131''\text{-}K$, $M_{LL} = 269''\text{-}K$, $M_u^- = 1557''\text{-}K = 130'\text{-}K$

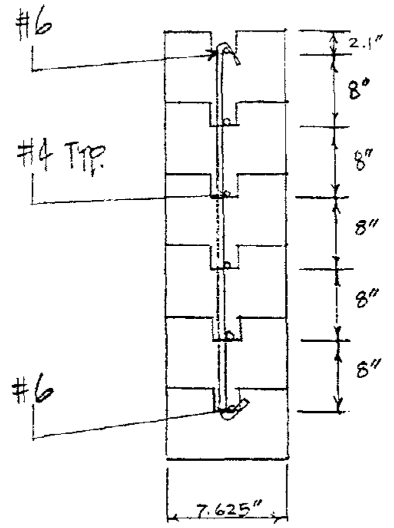
INT: $M_E = 1049''\text{-}K$, $M_{DL} = 140''\text{-}K$, $M_{LL} = 280''\text{-}K$, $M_u^- = 1511''\text{-}K = 126'\text{-}K$

$M_u^+ = 950''\text{-}k = 79.2'\text{-}k$

USE PROGRAM IMFLEX:

UNCONFINED STRESS STRAIN CURVE

(2-#6 + 4-#4) b=7.625", h=48.0"



$f_{me} = 1.3 \times 1500 = 1950$, $f_{ye} = 66\text{ksi}$, $\varepsilon_p = \varepsilon_{mu} = 0.0026$

TENSION @ TOP (BM9T.*), $M_{n1}^- = 203'\text{-}K = 2436''\text{-}k$

TENSION @ BOT (BM9B.*), $M_{n2}^+ = 182'\text{-}K = 2184''\text{-}k$

$\phi M_n^- = (0.85)(203) = 173 > 130$ ok; $\phi M_n^+ = 155 > 88.8$ ok

$\rho = [2(0.44) + 4(0.20)]/(7.625)(48)$

$= 0.0046$

$\rho = 0.0046 > \rho_{min} = 130/f_{ye} = 130/66 = 0.002$ OK

$\rho = 0.0046 \leq \rho_{max} = 0.35\rho_b = 0.0119$ OK

$c_b = \frac{0.0026}{0.0026 + \frac{66}{29000}}(42.1) = 22.45''$

$\rho_b = \frac{(1950)(22.5)}{66 \times 10^3 (42 - 22.5)} = 0.0339$

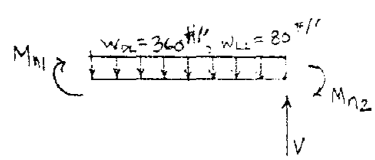
$0.35\rho_b = 0.0119$

$\therefore$ USE 2-#6 + 4-#4

TRY 2-#5 & 4-#4

TENSION @ TOP $M_n^- = 173'\text{-}k$

TENSION @ BOT $M_n^+ = 153'\text{-}k$



$\phi M_n^- = (0.85)(153) = 147'\text{-}k \approx 131'\text{-}k$; $\phi M_n^+ = 130 > 83.3$ ok

$\rho = 0.00388 > \rho_{min} = 0.002$

$\rho < \rho_{max} = 0.019$ OK

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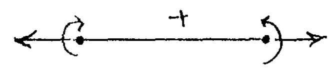
FLOOR BEAM

EQN
(A) 1.2D + 1.6L
(B) 1.3D + 1.0L + 1.0E
(C) 0.7D + 1.0E
(D) 0.7D − 1.0E



BEAM 9

EQN.	JOINT 5	MIDSPAN	JOINT 6
A	−431.86	298.04$^+$	−587.55
B	811.52		−1557.30
C	1065.9		−1209.71
D	−1210.17		1026.57

BEAM 10

EQN.	JOINT 6	MIDSPAN	JOINT 7
A	−615.94	309.12	−615.94
B	586.91	628.28*	−1510.57
C	950.91		−1146.57
D	−1146.58		950.91

BEAM 11

EQN	JOINT 7	MIDSPAN	JOINT 8
A	−587.55	298.04	−431.86
B	678.98	686.73$^+$	−1464.64
C	1026.57		−1210.17
D	−1209.71		1065.99

BEAM 9

JOINT 5:

$M_u^+ = 1065.9$ K-in

$M_{CAP}^+ = 18[illegible]6.4$ K-in $> M_u^+$ ok

$M_u^- = 1210.17$ K-in

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

JOINT 6

$M_u^+ = 1026.57$ K-in

$M_{CAP}^+ = 1856.4$ K-in $> M_u^+$ ok

$M_u^- = 1557.30$ K-in

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

BEAM 10

JOINT 6:

$M_u^+ = 950.91$ K-in

$M_{CAP}^+ = 1856.4$ K-in $> M_u^+$ ok

$M_u^- = 1146.58$ K-in

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

JOINT 7:

$M_u^+ = 950.91$ K-in

$M_{CAP}^+ = 1856.4$ K-in $> M_u^+$ ok

$M_u^- = 1510.57$ K-in

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

BEAM 11

JOINT 7:

$M_u^+ = 1026.57$ K-in

$M_{CAP}^+ = 1856.4$ K-in $> M_u^+$ ok

$M_u^- = 1209.71$

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

JOINT 8

$M_u^+ = 1065.99$ K-in

$M_{CAP}^+ = 1856.4$ K-in $> M_u^+$ ok

$M_u^- = 1464.64$

$M_{CAP}^- = 2070.6$ K-in $> M_u^-$ ok

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SHEAR DESIGN:

$$V_u = 1.3\,V_{DL} + 1.0\,V_{LL} + 1.0\,V_E$$

$$V_u = \frac{203 + 182}{18.67} + \frac{(1.3 \times 0.36 + 0.08)(18.67)}{2} = 25.7^K \text{ (EXTERIOR BAY)}$$

$$\rho_n = \frac{25.7}{(48)(7.625)(66)} = 0.00106 < \rho_{min} = 0.0015$$

USE $\rho = 0.0015$, MAX SPACING $= \frac{1}{4}(48) = 12''$, USE $s = 8''$

$$A_{sv} = 0.0015(8)(7.625) = 0.09\,in^2 \leq 0.11\,in^2\ (\#3)$$

TRY #3 @ 8" FOR SHEAR REINFORCEMENT

SHEAR STRENGTH V_n (AFTER FATTAL, 1991 NBS PUBLICATION)

$$v_n = \left[\left(\frac{0.76}{r_4 + 0.76} + 0.012\right)(4.04\,\rho_{ve})\sqrt{f_{me}} + 0.01575\left(\rho_h f_{yh} f_{me}\right)^{1/2} \delta + 0.175\,\sigma_o\right] d/L$$

$$r_4 = \frac{h}{d} = \frac{224}{48 - 2.1\ (\text{OR } 48 - 5.9)} = 4.88 \text{ or } 5.32 \quad ; \quad d = 45.9'' \text{ or } 42.1''$$

ht of wall = Beam Length

$L = 48''$

$\rho_{ve} = 0$ (NO END CORE)

$f_{me} = 1.95\,ksi = 1.95(6.89476) = 13.445\,Mpa$

$$\rho_h = \frac{A_h}{s_h t} = \frac{0.11}{(8)(7.625)} = 0.00180 = 0.18\% \text{ (USE PERCENTAGE)}$$

$f_{yh} = 66\,ksi = 66(6.89476) = 455.05\,Mpa$

$\delta = 1.0$ (FOR INFLECTION PT. @ MID HEIGHT)

$\sigma_o = 0$ (NO AXIAL FORCE)

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$\rho_v = \dfrac{0.44}{(7.625)(48)} = 0.12\%$

THEN,

$v_n = \left[0 + (0.01575)(1.0)(0.18 \times 455.05 \times 13.445)^{1/2} + 0\right]\left(\dfrac{45.9}{48}\right)$

$= 0.50 \text{ Mpa} = 72.5 \text{ psi}$

$V_n = (72.5)(48)(7.625) = 26.5^K > V_u = 25.7^K$ (3% GREATER)

TRY #4 @ 8"

$\rho_n = \dfrac{0.2}{8(7.625)} = 0.00328 = 0.328\%$

$v_n = 0.01575(1.0)(0.328 \times 455.05 \times 13.445)^{1/2}\left(\dfrac{45.9}{48}\right) = 0.675 \text{ Mpa} = 97.8 \text{ psi}$

$\phi V_n = (0.9)(97.8)(48)(7.625) = 32.2^k > V_u = 25.7^K$ — ok

USE #4 @ 8" O.C.

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ROOF BEAMS:

$M_u^+ = 604''^{-k} = 50.3'^{-k}$

EXT: $M_E = 646''^{-k}$, $M_{DL} = 60''^{-k}$, $M_{LL} = 61''^{-k}$, $M_u^- = 785''^{-k} = 65.4'^{-k}$

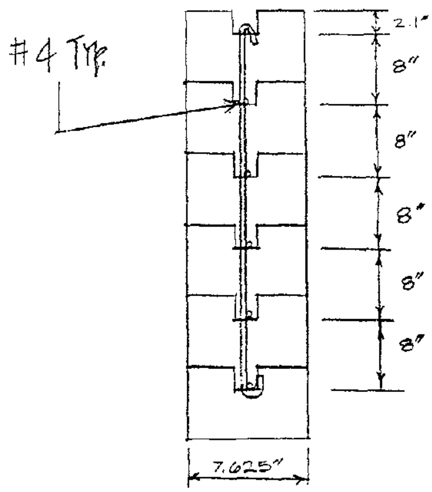
INT: $M_E = 610''^{-k}$, $M_{DL} = 84''^{-k}$, $M_{LL} = 55''^{-k}$, $M_u^- = 773''^{-k} = 64.4'^{-k}$

$M_u^+ = 551''^{-k} = 45.9'^{-k}$

USE PROGRAM IMFLEX:
UNCONFINED STRESS-STRAIN CURVE
(6-#4) $b = 7.625''$, $h = 48''$

$f_{me} = 1950\,psi$, $f_{ye} = 66\,ksi$

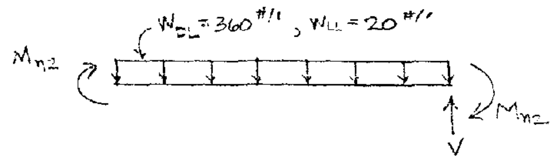
TENSION @ TOP (BM12T.*), $M_{n1}^- = 148'^{-k}$
TENSION @ BOT (BM12B.*), $M_{n2}^+ = 130'^{-k}$



$\phi M_n^- = 126 > 65.4$ OK; $\phi M_n^+ = 111 > 50.3$ OK

$\rho = 0.0033 \geq 0.002$ OK
$\rho = 0.0033 \leq 0.35\rho_b = 0.0119$ OK

$\therefore$ USE 6-#4



SHEAR DESIGN:

$$V_u = \frac{148 + 130}{18.67} + \frac{(1.3 \times 0.36 + 0.02)(18.67)}{2} = 19.4^k$$

USE #4 @ 8" O.C. $\phi V_n = (0.9)(35.8) = 32.2^k > V_u = 19.4^k$ OK

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ROOF BEAM

BEAM 12

EQN	JOINT 9	MIDSPAN	JOINT 10
A	-169.42	85.87	-158.60
B	[illegible]		-775.77
C	603.84		-688.29
D	-687.96		579.53

BEAM 13

EQN	JOINT 10	MIDSPAN	JOINT 11
A	-187.78	98.20	-187.78
B	446.42		-733.09
C	551.25		-668.27
D	-668.27		551.24

BEAM 14

EQN	JOINT 11	MIDSPAN	JOINT 12
A	-158.60	85.87	-169.42
B	492.06		-784.83
C	579.53		-687.96
D	-688.29		603.84

BEAM 12

JOINT 9:

M_u^+ = [illegible] K-in

$M_{CAP}^+ = \phi M_n^+ = (0.85)(130)(12)$
$= 1326$ K-in $> M_u^+$ ok

M_u^- = 687.96 K-in

$M_{CAP}^- = \phi M_n^- = (0.85)(148)(12)$
$= 1509$ K-in $> M_u^-$ ok

JOINT 10:

M_u^+ = 579.6 K-in

M_{CAP}^+ = 1326 K-in $> M_u^+$ ok

M_u^- = 775.77 K-in

M_{CAP}^- = 1509 K-in $> M_u^-$ ok

BEAM 13

JOINT 10:

M_u^+ = 551.25 K-in

M_{CAP}^+ = 1326 K-in $> M_u^+$ ok

M_u^- = 668.27 K-in

M_{CAP}^- = 1509 K-in $> M_u^-$ ok

JOINT 11:

M_u^+ = 551.25 K-in

M_{CAP}^+ = 1326 K-in $> M_u^+$ ok

M_u^- = 733.09

M_{CAP}^- = 1509 K-in $> M_u^-$ ok

BEAM 14

JOINT 11:

M_u^+ = 579.53 K-in

M_{CAP}^+ = 1326 K-in $> M_u^+$ ok

M_u^- = 688.29 K-in

M_{CAP}^- = 1509 K-in $> M_u^-$ ok

JOINT 12:

M_u^+ = 603.84 K-in

M_{CAP}^+ = 1326 K-in $> M_u^+$ ok

M_u^- = 784.83

M_{CAP}^- = 1509 K-in $> M_u^-$ ok

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INTERIOR COLUMN DESIGN

$$M_n = (203+182)/2 = 192.5^{'-K}$$

$$P_n = 0.7P_{DL} - 1.0P_E$$

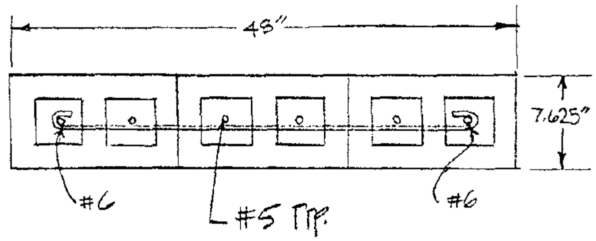
$$= 0.7(32.6) - \frac{148+130}{18.67} + \frac{148+130}{20} - \frac{203+182}{18.67} + \frac{203+182}{20}$$

$$= 20.5^K$$

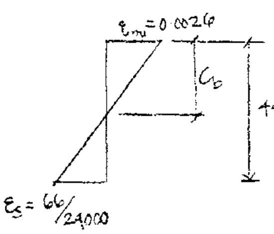
USING IMFLEX: (COL2.*)

$b = 7.625$, $h = 48''$, $f_{me} = 1950$ psi, $P_n = 20.5^K$

$(2\text{-}\#6 + 4\#5) \rightarrow M_n = 259.4^{'-K} > 192.5^{'-K}$ — ok



AT BALANCE CONDITION:



$$\frac{C_b}{0.0026} = \frac{44 - C_b}{66/29000} \Rightarrow C_b = \frac{44(0.0026)}{\left(0.0026 + \frac{66}{29000}\right)} = 23.46''$$

$$\rho_b = \frac{\left[0.5 f_m t C_b - (P_D + P_{NE})\right]}{0.5 f_{ye}\, t\,(d - C_b)}$$

$$= \frac{\left[(0.5)(1.95)(7.625)(23.46) - 20.5\right]}{(0.5)(66)(7.625)(44 - 23.46)} = 0.0298$$

$$0.35\rho_b = 0.35(0.0298) = 0.0104$$

$$\rho_n = \left[2(0.44) + 4(0.31)\right]/(7.625)(48) = 0.00579$$

$$\rho_n = 0.00579 > \rho_{min} = {}^{130}/_{f_{yc}} = 0.002 \quad \text{— ok}$$

$$\rho_n = 0.00579 < \rho_{max} = 0.35\rho_b = 0.0104 \quad \text{— ok}$$

USE 2-#6 + 4-#5

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AXIAL FORCE CHECK:

$P_u = 1.3P_{DL} + 1.0P_{LL} + 1.0P_E = 1.3(32.6) + 18.9 + 2.3 = 63.6^k$

$0.3A_n f_{me} = 0.3(7.625)(48)(1.95) = 214^k > P_u$ ok

$0.6P_b = 0.5 f_{me} t C_b - 0.6\rho_b(0.5 f_{ye}) t (d - C_b) = (0.5)(1.95)(7.625)(23.46) - (0.6)(0.0293)$
$(0.5 \times 66)(7.625)(44 - 23.46) = 82^k > P_u$ ok

SHEAR DESIGN

$M_{PIER} = (203 + 182)/2 = 192.5'^{-k}$

$h_c/2 = 10.5/2 = 5.25'$

$V_u = 192.5/5.25 = 36.7^K$

$\rho_n = \frac{36.7}{(48)(7.625)(66)} = 0.001527 \quad \rho_{min} = 0.0016$

MAX SPACING = $h/4$, USE $S = 8''$

$A_{SV} = 0.00152(8)(7.625) = 0.0923$

TRY #4 @ 8"

SHEAR CAPACITY FOR COLUMN

$V_u = 36.7^K, \quad Q_n = 63.6^K, \quad d = 44'', \quad L = 48'', \quad h = 10.5' = 126''$

$r_d = \frac{h}{d} = \frac{126}{44} = 2.86$

$\rho_{ve} = 0$ (NO END CORE) OR $\rho_{ve} = \frac{0.44}{7.625(48)} = 0.12\%$ (1-#6 AS CORE REBAR)

$f_{me} = 1.95(6.895) = 13.445\,Mpa, \quad f_{yh} = 66\,ksi = 455.05\,Mpa$

$\rho_h = \frac{0.2}{8(7.625)} = 0.328\%$

$\delta = 1.0$

$\sigma_0 = Q/A = \frac{63.6^k}{48(7.625)} = 0.173\,ksi = 1.198\,Mpa$

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$$v_n = [0 + 0.01575(1.0)(0.328 \times 455.05 \times 13.445)^{1/2} + 0.175(1.198)]\left(\frac{44}{48}\right)$$

$$= [0 + 0.706 + 0.210]\left(\frac{44}{48}\right) = 0.839\,Mpa = 121.7\,psi$$

$$\phi V_n = 0.9(121.7)(48)(7.625) = 40.1^k > V_u = 36.7^k$$ OK

USE #4 @ 8"

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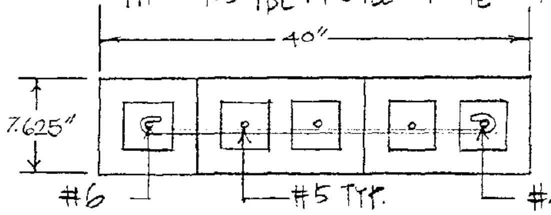
EXTERIOR COLUMN DESIGN

$M_n = 203/2 = 101.5^k$

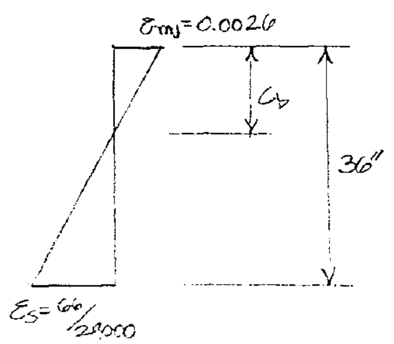
$P_n = 0.7P_{DL} - 1.0P_E = 0.7(13.8) - (148+130+203+182)/18.67 = -25.9$ (TENSION)

OR $P_n = 1.3P_{DL} + 1.0P_{LL} + 1.0P_E = 1.3(13.8) + 0.6 + 35.5 = 54.1^k$ (COMPRESSION)



$b = 7.625"$, $h = 40"$, $f'_{me} = 1950\,psi$, $P_n = -25.9^k$

AT BALANCE CONDITION :



$$c_b = \frac{36(0.0026)}{(0.0026 + 66/29000)} = 19.2"$$

$$\rho_b = \frac{[0.5 f'_m t c_b - (P_c + P_{wc})]}{0.5 f_{ye} t (d - c_b)}$$

$$= \frac{[(0.5)(1.95)(7.625)(19.2) - 54.1]}{(0.5)(66)(7.625)(36-19.2)} = 0.0210$$

$0.35\rho_b = 0.00734$

USING IMFLEX,

$(2\text{-}\#6 + 3\#5) \rightarrow M_n = 133^{1\text{-}K} > 101.5^{1\text{-}K}$ — ok

$\rho_n = [2(0.44) + 3(0.31)]/(7.625)(40) = 0.00593$

$\rho_n = 0.00593 > \rho_{min} = 0.002$ — ok

$\rho_n = 0.00593 < \rho_{max} = 0.35\rho_b = 0.00734$ — ok

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AXIAL FORCE CHECK:

$P_u = 1.3P_{DL} + 1.0P_{LL} + 1.0P_E = 1.3(13.8) + 0.6 + 35.5 = 54.1^K$

$0.3A_n f'_{me} = 0.3(7.625)(40)(1.95) = 178^k > P_u$ OK

$0.6P_b = 0.5(1.95)(7.625)(19.2) - 0.6(0.021)(0.5)(66)(7.625)(36-19.2) = 89.5^k > P_u$ OK

USE 2-#6 + 3-#5

SHEAR DESIGN;

$M_{PIER} = 203/2 = 101.5^{k\text{-}'}$

$V_n = (101.5)(2)/10.5 = 19.3^K$

$\rho_n = \dfrac{19.3}{(40)(7.625)(66)} = 0.00096 < \rho_{min}$

USE $\rho = 0.0015$

$A_{sv} = 0.0015(8)(7.625) = 0.092$

TRY #4 @ 8

$V_u = 19.3^K$, $Q_u = 54.1^K$, $d = 36''$, $L = 40''$, $h = 10.5' = 126''$

$r_d = h/d = 126/36 = 3.5$

$\rho_{ve} = 0$; $f_{me} = 13.445\,\text{Mpa}$, $f_{ye} = 445.05\,\text{Mpa}$, $\rho_h = 0.328\%$

$\delta = 1.0$; $\sigma_o = \dfrac{54.1}{(40)(7.625)} = 0.177\,\text{Ksi} = 1.223\,\text{Mpa}$

$v_n = \left[0 + 0.01575(0.328 \times 455.05 \times 13.445)^{1/2} + 0.1775(1.223)\right]\left(\dfrac{36}{40}\right)$

$= 0.828\,\text{Mpa} = 120\,\text{psi}$

$\phi V_n = 0.9(120)(40)(7.625) = 32.9^k > V_u = 19.3^K$ — OK

USE #4 @ 8"

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CHECK ADEQUACY OF COLUMN DESIGN

COLUMN 1

EQN	AXIAL LOAD (KIPS)	MOMENT @ JOINT 1 (K-IN)	M @ JOINT 5 (K-IN)
A	-21.52	162.41	-277.30
B	-0.97	-1100.52	315.05
C	+11.62	-1197.31	480.31
D	-19.96	1241.35	-555.51

COLUMN 5

EQN	AXIAL LOAD (KIPS)	M @ JOINT 5 (K-IN)	M @ JOINT 9 (K-IN)
A	-5.28	280.94	-175.91
B	+1.18	-61.0	437.70
C	+4.15	-225.25	536.89
D	-7.28	331.14	-637.33

EXTERIOR COLUMNS

COLUMN 2

EQN	AXIAL LOAD (KIPS)	MOMENT @ JOINT 2 (K-IN)	MOMENT @ JOINT 6 (K-IN)
A	-47.74	3.82	-6.74
B	-39.11	-2111.39	967.57
C	-11.26	-2113.48	971.26
D	-7.33	2116.24	-976.14

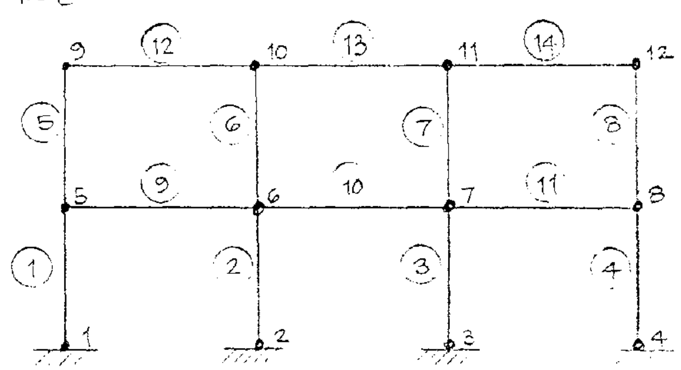
COLUMN 6

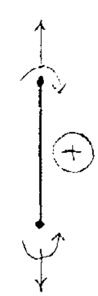
EQN.	AXIAL LOAD (KIPS)	MOMENT AT JOINT 6 (K-IN)	MOMENT @ JOINT 10 (K-IN)
A	-11.08	19.90	-28.32
B	-10.29	-625.99	1124.8
C	-4.12	-637.64	1141.71
D	-2.85	644.94	-1149.10

INTERIOR COLUMNS

EQN.

(A) 1.2D + 1.6L
(B) 1.3D + 1.0L + 1.0E
(C) 0.9D + 1.0E
(D) 0.7D - 1.0E





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COLUMN 3

EQN	AXIAL LOAD (KIPS)	MOMENT AT JOINT 3 (KIP-IN)	MOMENT @ JOINT 7 (KIP-IN)
A	−47.74	−3.82	−6.74
B	−35.18	−2114.86	937.70
C	−7.34	−2116.24	976.14
D	−11.26	2113.48	−971.26

COLUMN 7

EQN	AXIAL LOAD (KIPS)	M @ JOINT 7 (KIP-IN)	M @ JOINT 11 (KIP-IN)
A	−11.09	−19.90	28.32
B	−9.03	−656.59	1166.01
C	−2.86	−644.94	1149.11
D	−4.12	637.64	−1141.71

INTERIOR COLUMNS

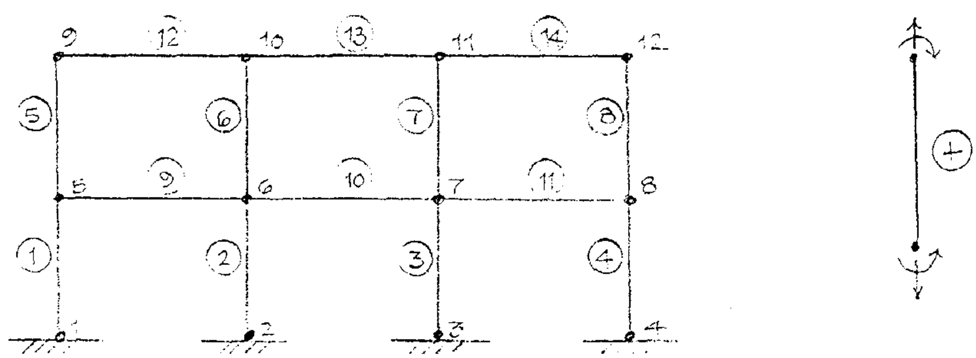
COLUMN 4

EQN	AXIAL LOAD (KIPS)	MOMENT @ JOINT 4 (KIP-IN)	MOMENT @ JOINT 8 (KIP-IN)
A	−21.52	−162.41	277.30
B	−32.54	−1338.16	720.76
C	−19.96	−1241.35	555.51
D	+11.61	1197.31	−480.30

COLUMN 8

EQN	AXIAL LOAD (KIPS)	MOMENT @ JOINT 8 (KIP-IN)	MOMENT @ JOINT 12 (KIP-IN)
A	−5.68	−280.94	175.91
B	−10.25	−459.39	736.52
C	−7.27	−331.15	687.33
D	+4.15	225.25	−536.89

EXTERIOR COLUMNS



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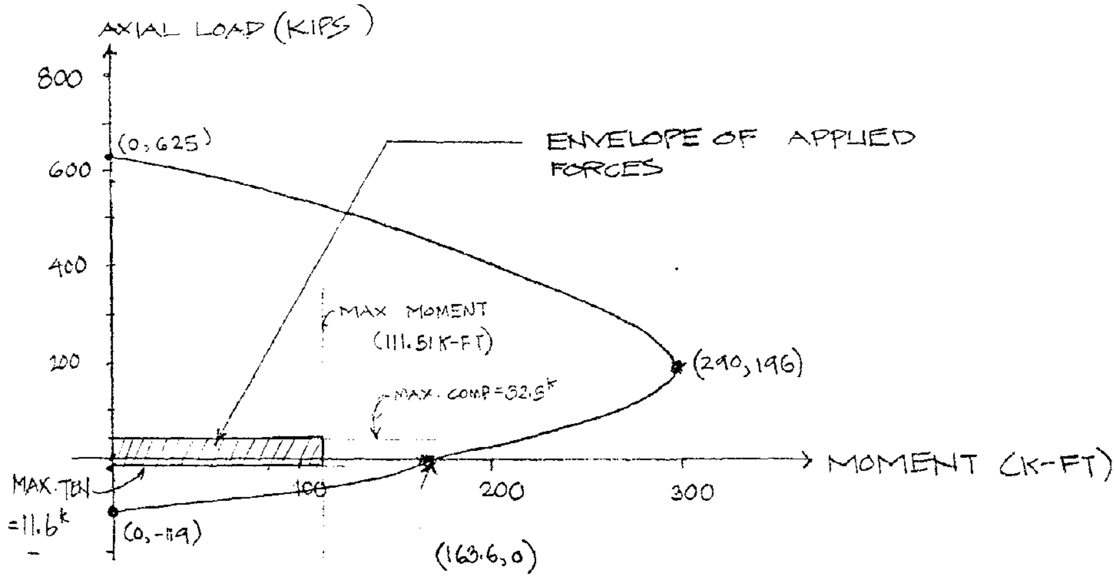
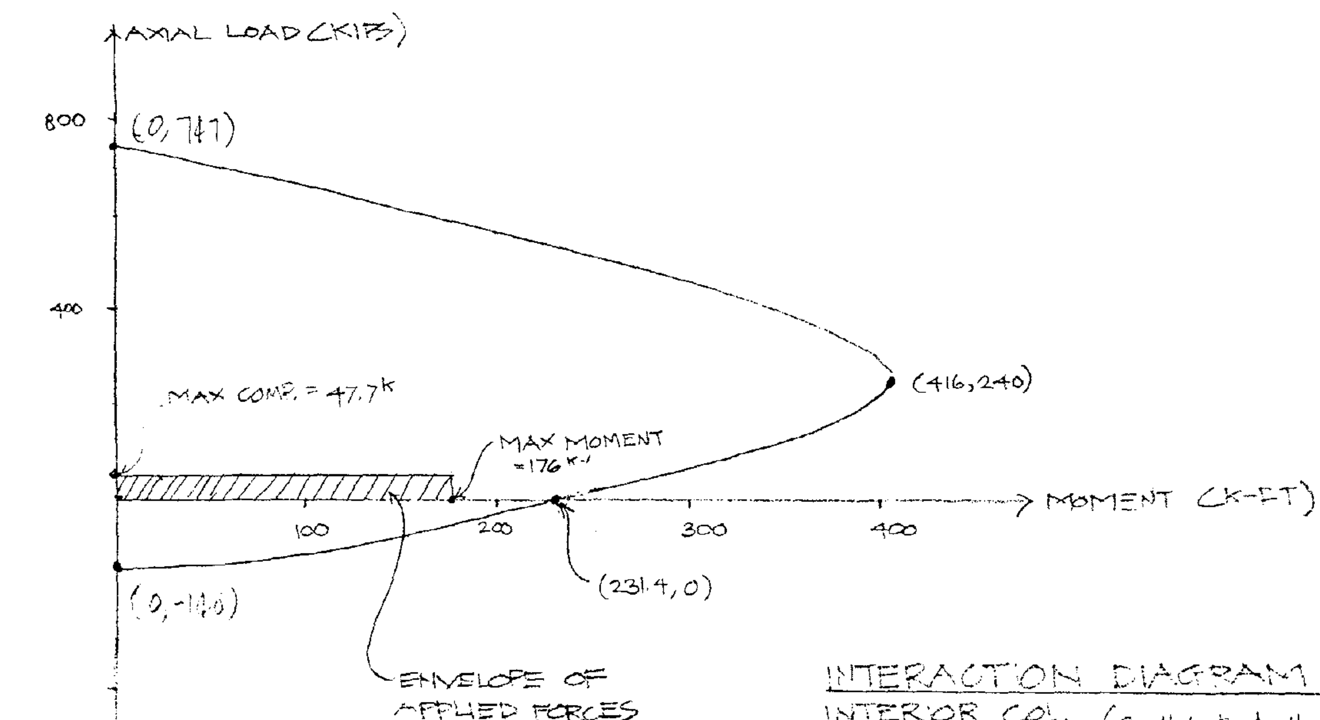


INTERACTION DIAGRAM FOR EXTERIOR COL. (2-#6 & 3-#5)



INTERACTION DIAGRAM FOR INTERIOR COL. (2-#6 & 4-#5)

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JOINT SHEAR (INTERIOR)

$h_c \geq 60 d_b$: $48'' > 60(\frac{6}{8}) = 45''$ OK

$h_b \geq 40 d_c$; $48'' > 40(\frac{6}{8}) = 30''$ OK

BEAM EXPECTED FLEXURAL MOMENT

TENSION @ TOP ; $M_{n1} = 203^{'-K}$

TENSION @ BOT ; $M_{n2} = 182^{'-K}$

JOINT SHEAR:

$V_{jh} = \frac{M_T + M_B - V_B h_c'}{h_b'}$; $V_{jv} = \frac{M_T + M_B - H h_b'}{h_c'}$

$M_T + M_B = 203 + 182 = 385^{'-K}$; $h_c' = 0.8 h_c = 38.4''$, $h_b' = 0.8 h_b = 38.4''$

$V_B = 385(12)/(112+120) = 20^k$; $H = \frac{385(12)}{60+63} = 37.6^K$

$V_{jh} = \frac{385(12) - (20)(38.4)}{38.4} = 100^k$; $V_{jv} = \frac{385(12) - 37.6(38.4)}{38.4} = 82.7^K$

$v_{jh} = 100/[48(7.625)] = 274$ psi ; $v_{jv} = 82.7/(48)(7.625) = 226$ psi

< 350 psi - ok < 350 psi - ok

$A_{jh} = \frac{0.5(100)}{0.8(66)} = 0.95 \text{ in}^2$; $A_{jv} = \frac{0.5(82.7)}{0.8(66)} = 0.78 \text{ in}^2$

$\leq 1.24 \text{ in}^2$ (4-#5 PIER INTERIOR REBAR)

PROVIDE 2-#5 + 2-#4 $A_s = 1.02 \text{ IN}^2$

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JOINT SHEAR (EXTERIOR)

$$V_{jh} = \frac{M_T + M_B - V_B h_c'/2}{h_b'} \quad ; \quad V_{jv} = \frac{M_T + M_B - 2H\, h_b'}{h_c'}$$

$h_c' \simeq 0.8h_c = 0.8(40) = 32''$, $h_b' = 0.8h_b = 0.8(48) = 38.4''$

TENSION @ TOP , $M_{n1} = 203^{1-k}$

TENSION @ BOT. , $M_{n2} = 182^{1-k}$

$M_T + M_B = 203^{1-k}$ or 182^{1-k}

$V_B = (203)(12)/112 = 21.8^k$; $H = \frac{203(12)}{60+63} = 19.8^k$

$V_{jh} = \frac{203(12) - (21.8)(32)/2}{38.4} = 54.4^k$; $V_{jv} = \frac{(203)(12) - 2(19.8)(38.4)}{32} = 28.6^k$

$v_{jh} = \frac{54.4}{(40)(7.625)} = 178\text{psi} < 350\text{psi}$ – OK ; $v_{jv} = \frac{28.6}{(48)(7.625)} = 78\text{psi} < 350\text{psi}$ – OK

$A_{jh} = \frac{0.5(54.4)}{0.8(66)} = 0.52\,\text{in}^2$; $A_{jv} = \frac{0.5(28.6)}{0.8(66)} = 0.27\text{in}^2 < 0.93\text{in}^2$

(3-#5 PIER INTERM REBAR)

PROVIDE 4-#4 , $A_s = 0.8\,\text{in}^2$

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CHECK STIFFNESS ASSUMPTION

1) EXTERIOR FLOOR BEAM

$M_{DL} = 131^{k''}$, $M_{LL} = 269^{k''}$, $M_E = 1118^{k''}$;

$M_a = 1.3(131) + 269 + 1118 = 1557^{k''} = 130^{k'}$

$M_{cr} = (f_r + P/A)S = (199 + 0)(2923) = 48.5^{k'}$

$f_r = 4.5\sqrt{f_{me}} = 4.5\sqrt{1950} = 199$ psi , $I_g = \frac{1}{12}(7.625)(48)^3 = 70272$ IN^4

$I_{cr}^t = 16757$ IN^4; $I_{cr}^b = 12438$ IN^4 ; $I_{cr}^t/I_g = 0.24$; $I_{cr}^b/I_g = 0.18$; $(I_{cr}/I_g)_{AVG} = 0.21$

$\alpha = \left(\frac{M_{cr}}{M_a}\right)^3 = \left(\frac{48.5}{130}\right)^3 = 0.052$; $1 - \alpha = 0.95$

$I_{eff}/I_g = \left(\frac{M_{cr}}{M_a}\right)^3 + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right](I_{cr}/I_g) = \alpha + (1-\alpha)(I_{cr}/I_g)_{AVG}$

$I_{eff}/I_g = (0.052) + (0.95)(0.21) = 0.25 \approx 0.25$ ASSUMED OK

2) INTERIOR FLOOR BEAM

$M_{DL} = 140^{k''}$, $M_{LL} = 280^{k''}$, $M_E = 1049^{k''}$, $M_a = 1511^{k''} = 126^{k'}$

$\alpha = \left(\frac{48.5}{126}\right)^3 = 0.057$, $1 - \alpha = 0.94$

$I_{eff}/I_g = (0.057) + (0.94)(0.21) = 0.25 \approx 0.25$ ASSUMED OK

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3) EXTERIOR ROOF BEAM

$M_{DL} = 60^{k''}$, $M_{LL} = 61^{k''}$, $M_E = 646^{k''}$, $M_a = 785^{k''} = 65.4^{k'}$, $M_{cr} = 48.5^{k'}$

$I_g = 70272$ IN^4; $I_{cr}^t = 11357$ IN^4; $I_{cr}^b = 8257$ IN^4

$(I_{cr}^t / I_g) = 0.16$, $(I_{cr}^b / I_g) = 0.12$; $(I_{cr} / I_g)_{Avg} = 0.14$

$\alpha = \left(\frac{48.5}{65.4}\right)^3 = 0.41$, $1-\alpha = 0.59$

$I_{eff} / I_g = 0.41 + (0.59)(0.14) = 0.49 \gg 0.25$ NG TRY 0.35

4) INTERIOR ROOF BEAM

$M_{DL} = 84^{k''}$, $M_{LL} = 55^{k''}$, $M_E = 610^{k''}$, $M_a = 773^{k''} = 64.4^{k'}$, $M_{cr} = 48.5^{k'}$

$\alpha = \left(\frac{48.5}{64.4}\right)^3 = 0.43$, $1-\alpha = 0.57$

$I_{eff} / I_g = 0.43 + (0.57)(0.14) = 0.51 \gg 0.25$ NG TRY 0.35

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5) EXTERIOR COLUMN AT 1ST FLOOR

a) $P_a = 32.5^k,\ M_a = 1338^{k''} = 112^{k'}$; $P_a = 20^k,\ M_a = 103^{k'}$

$M_{cr} = (f_r + P/A)\,S = \left[199 + \frac{32.5\times10^3}{(40)(7.625)}\right](2033) = (199+107)(2033) = 51.8^{k'}$

$I_g = \frac{1}{12}(7.625)(40)^3 = 40667\ IN^4,\ I_{cr} = 9235\ IN^4$; $I_{cr}/I_g = 0.23$

$\alpha = \left(\frac{51.8}{112}\right)^3 = 0.099$; $1-\alpha = 0.90$

$I_{eff}/I_g = 0.099 + 0.9\,(0.23) = 0.30 \ll 0.5$ NG TRY 0.4

b) $P = -11.6^k,\ M_a = 1197^{k''} = 100^{k'}$

$M_{cr} = \left[199 - \frac{11.6\times10^3}{(40)(7.625)}\right](2033) = 27.3^{k'}$, $\alpha = \left(\frac{27.3}{100}\right)^3 = 0.020$, $1-\alpha = 0.98$

$I_{eff}/I_g = 0.02 + (0.98)(0.23) = 0.25 \ll 0.5$ NG TRY 0.4

6) EXTERIOR COLUMN AT 2ND FLOOR

a) $P_a = 10.3^k,\ M_a = 736^{k''} = 61.3^{k'}$.

$M_{cr} = \left(199 + \frac{10.3\times10^3}{(40)(7.625)}\right)(2033) = 39.4^{k'}$; $\alpha = \left(\frac{39.4}{61.3}\right)^3 = 0.27$; $1-\alpha = 0.73$

$I_{cr} = 9157\ IN^4$; $I_{cr}/I_g = 9157/40667 = 0.23$

$I_{eff}/I_g = 0.27 + (0.73)(0.23) = 0.45 \approx 0.5$ OK

b) $P_a = -4.2^k,\ M_a = 537^{k''} = 44.8^{k'}$

$M_{cr} = \left(199 - \frac{4.2\times10^3}{(40)(7.625)}\right)(2033) = 31.4^{k'}$; $\alpha = 0.34$; $1-\alpha = 0.66$

$I_{eff}/I_g = 0.34 + 0.66\,(0.23) = 0.49 \approx 0.5$ OK

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7) INTERIOR COLUMN AT 1ST FLOOR

a) $P_a = 35.2^k$, $M_a = 2118^{k''} = 177^{k'}$

$M_{cr} = (199 + 35.2\times10^3/48/7.625)(2928) = 72^{k'}$; $\alpha = \left(\frac{72}{177}\right)^3 = 0.067$; $1-\alpha = 0.93$

$I_g = \frac{1}{12}(7.625)(48)^3 = 70272$, $I_{cr} = 15584$, $I_{cr}/I_g = 0.22$

$I_{eff}/I_g = 0.067 + (0.93)(0.22) = 0.27 \ll 0.5$ NG TRY 0.3

8) INTERIOR COLUMN AT 2nd FLOOR

a) $P_a = 9.0^k$, $M_a = 1166^{k''} = 97.2^{k'}$

$M_{cr} = (199 + 9\times10^3/48/7.625)(2928) = 54.6^{k'}$; $\alpha = \left(\frac{54.6}{97.2}\right)^3 = 0.18$; $1-\alpha = 0.82$

$I_{cr} = 15566$, $I_{cr}/I_g = 0.22$

$I_{eff}/I_g = 0.18 + (0.82)(0.22) = 0.36 < 0.5$ NG TRY 0.4

b) $P_a = -0.63^k$, $M_a = 1145^{k''} = 95.4^{k'}$

$M_{cr} = (199 - 0.63/48/7.625)(2928) = 48.6^{k'}$; $\alpha = \left(\frac{48.6}{95.4}\right)^3 = 0.13$; $1-\alpha = 0.87$

$I_{eff}/I_g = 0.13 + (0.87)(0.22) = 0.32 < 0.5$ NG TRY 0.4

THE STIFFNESS CHECKING CONTINUED UNTIL EFFECTIVE MOMENT OF INERTIA AS ASSUMED IS CLOSE TO THE COMPUTED VALUE FOR ALL MEMBERS IN THE WALL FRAME. THE RATIOS OF EFFECTIVE MOMENT INERTIA FOR EACH ITERATION CYCLE AND THE FINAL RATIOS WHEN CONVERGENCE IS REACHED ARE SHOWN AS FOLLOWS.

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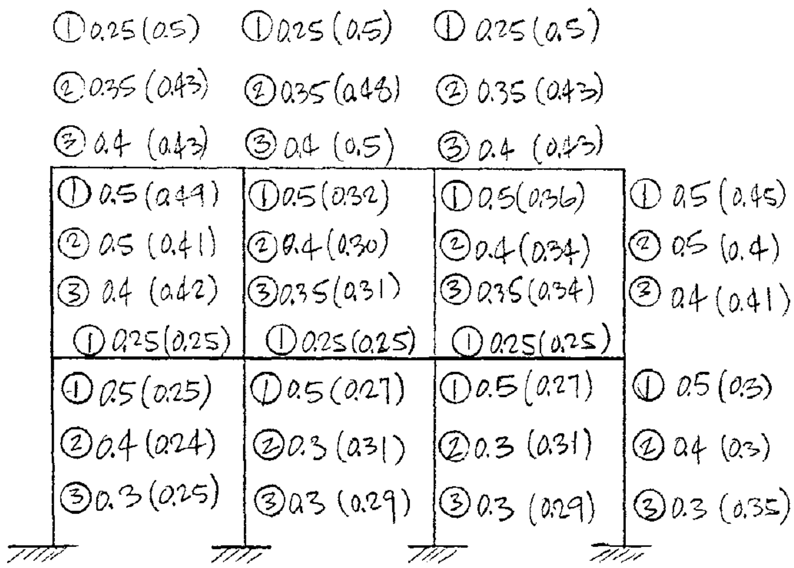
I_{eff}/I_G RATIO



CONVERGED I_{eff}/I_G RATIO