米日

U.S. - JAPAN COORDINATED PROGRAM
FOR
MASONRY BUILDING RESEARCH

REPORT NO. 2.1-10

PB93212975

SEISMIC PERFORMANCE STUDY
RCJ HOTEL

by

GARY C. HART
ROBERT E. ENGLEKIRK
JING-WEN JAW
MUKUND SRINIVASAN
SAMPSON C. HUANG
DENNIS J. DRAG

FEBRUARY 1992

supported by:

NATIONAL SCIENCE FOUNDATION

GRANT NO. BCS-8722869

EKEH

Ewing/Kariotis/Englekirk & Hart

This report presents the results of a research project which was part of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States - Japan Coordinated Masonry Research Program conducted under the auspices of the Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

This material is based on work supported by the National Science Foundation under the direction of Program Director, Dr. S.C. Liu.

PB93-212975

SEISMIC PERFORMANCE STUDY
RCJ HOTEL

by

Gary C Hart
Robert E Englekirk
Jing-Wen Jaw
Mukund Srinivasan
Sampson C Huang
Dennis J Drag

EKEH Report Number #2.1-10
February 1992

EKEH, Inc.

Supported by National Science Foundation
Grand No. BCS-8722869

TABLE OF CONTENTS

CHAPTER 1
EXECUTIVE SUMMARY

The Seismic Performance Study of the RCJ Hotel was conducted as part of the TCCMAR program to study and calibrate the recently developed Limit State Design Standards for Masonry Structures (LSDS). The fundamental objective of the Task 2 Research was to develop analytical models for masonry elements and systems that were validated using the comparison of predicted and measured laboratory tests performed by other TCCMAR researchers.

The RCJ hotel is a four story masonry wall building with precast planks for the floors and roof and was designed by the LSDS standard. This study consisted of a threefold analytical approach - SAP90, DRAIN-2DX and SCM modeling. Results from all three approaches are presented in this report. Although a few general conclusions can be drawn from the results, the final Task 2 Summary Report will summarize and make specific recommendations and criticisms.

All techniques and methodologies developed in this report benefitted greatly from the technical input of the other members of the TCCMAR Task 2 team. This close cooperation also served to ensure that appropriate objectives for the research were met.

CHAPTER 2
BUILDING DESCRIPTION

The RCJ hotel is a four story masonry shear wall building. It is rectangular in plan, measuring 120 feet by 67 feet. The first story is 10 feet 10 inches in height and the other three stories are 9 feet eight inches high with an overally masonry wall height of 40 feet 4 inches. The building is shown in Figures 2.1 - 2.3.

The Roof and floors of the hotel are constructed with 8 inch precast hollowcore planks with 2 inch thick normal weight topping. The precast planks are also pretensioned with high-tensile strands. In the longer direction (EW), the main lateral load resistance is provided by a central wall running the entire length of the building. It is interrupted by doors that are 40 inches wide and 84 inches high. The presence of these doors essentially splits the wall into four equivalent walls that are connected at the floor levels by the precast planks and the masonry lintels above the doors i.e. it is a system of shear walls with coupling beams. In the shorter (NS) direction, the main lateral load resistance is provided by a series of five parallel shear walls running along the shorter direction and interrrupted by a central corridor.

The building was studied for response in the longer direction (EW) only due to the presence of the coupling beams by consensus with the other members of the TCCMAR Task 2 team.

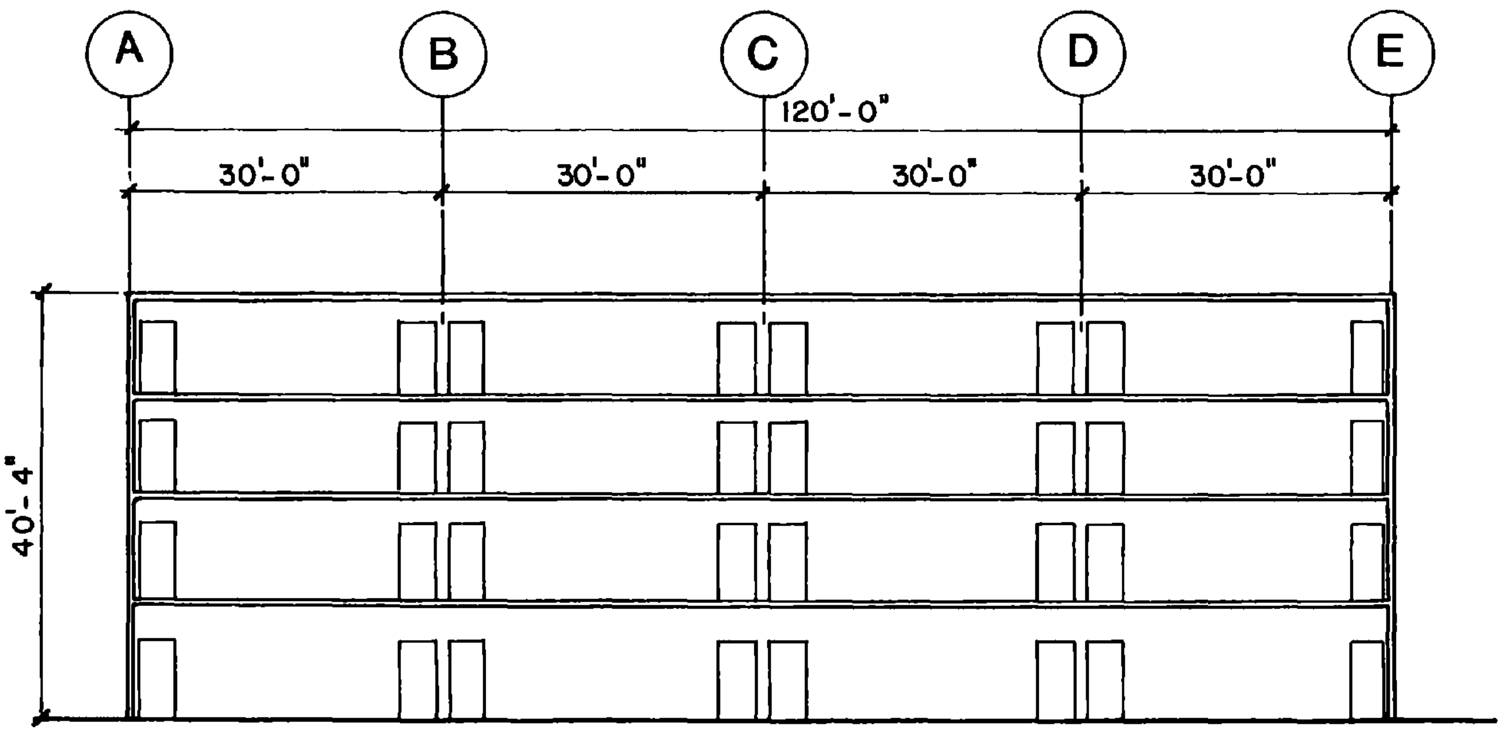
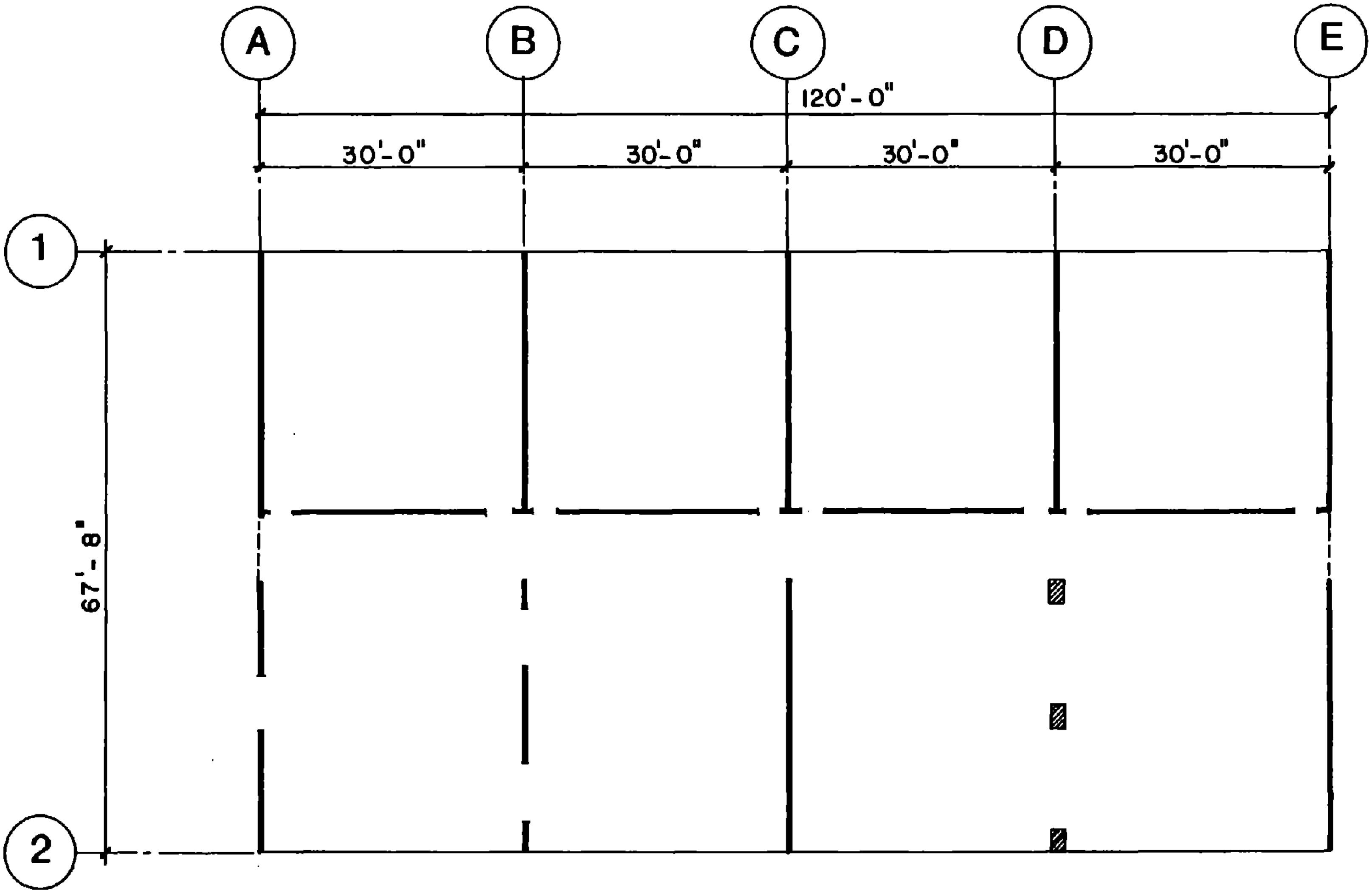


FIGURE 2.1 BUILDING SECTION LOOKING NORTH



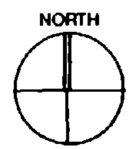
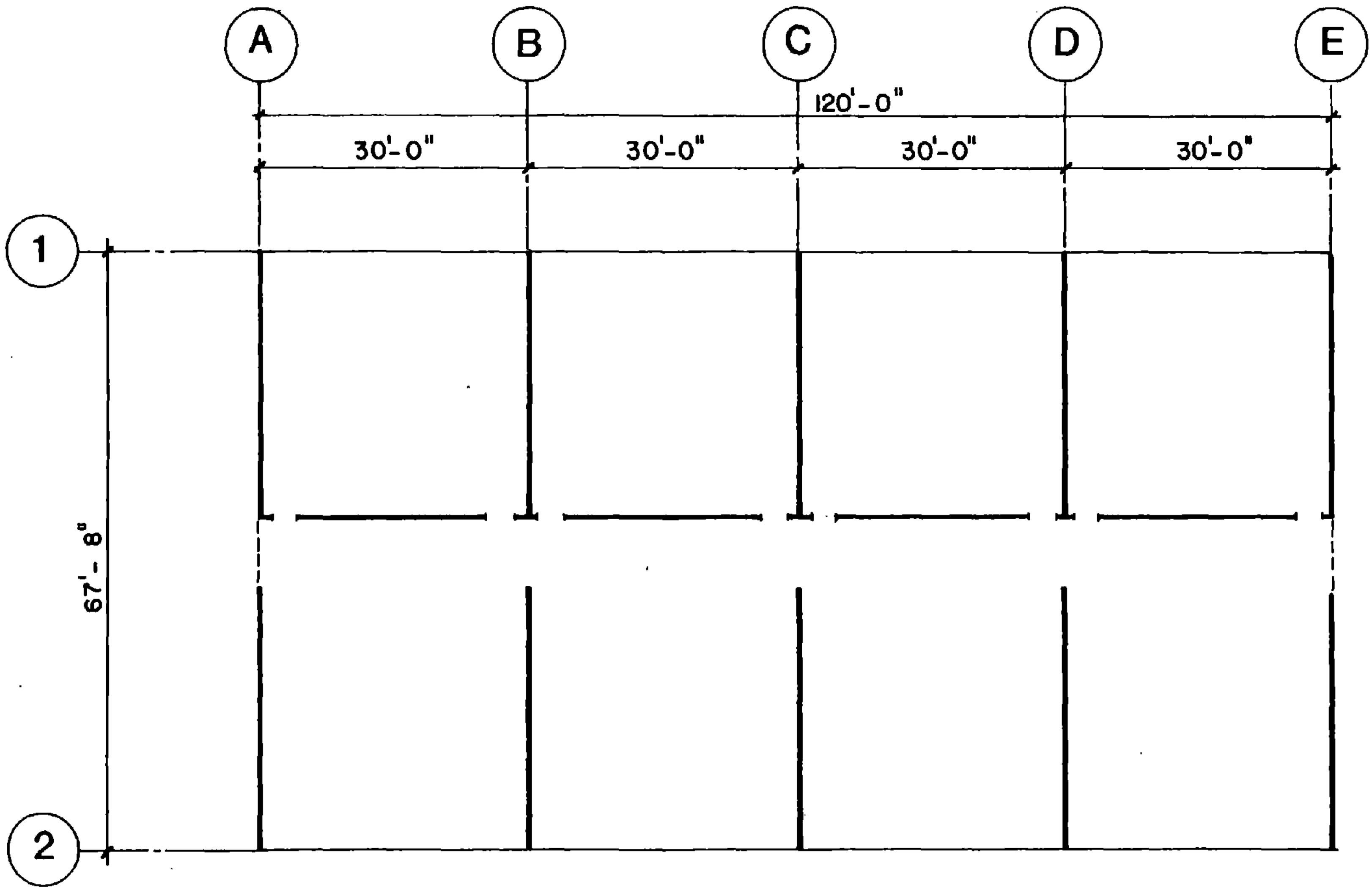


FIGURE 2.2 RCJ HOTEL: 1ST FLOOR PLAN



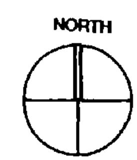


FIGURE 2.3 2ND, 3RD, & 4TH FLOOR PLAN

CHAPTER 3
EXPECTED BUILDING PERFORMANCE USING SAP90

3.1 General

The SAP90 [3] analysis of the RCJ hotel was performed for ground motion in the EW direction only as stated earlier. In the EW direction, the primary lateral force resisting system is the set of four shear walls connected by coupling beams as discussed earlier. Therefore, only these walls were modeled. Therefore, the model was essentially a two dimensional representation of the RCJ hotel. The stiffness characteristics for the coupling beams were computed through inelastic analysis results obtained from Ewing and Associates in the EKEH Report 2.2-3. [4].

3.2 Modeling Procedure

The lateral force resisting components in the EW direction for the RCJ hotel that were modeled were the shear walls and the coupling beams. The discretized SAP90 model for the RCJ hotel is shown in Figure 3.1. The stiffness characteristics for the model were set as follows:

Shear Walls

The shear walls were idealized as shell elements with the stiffnesses being specified through the element thickness and the modulus of elasticity. The initial modulus of elasticity was set at 2500 ksi. Both bending and membrane effects were considered for the shell elements. The shear walls were allowed to move horizontally as well as vertically and could also rotate. In order to obtain a good estimate of the nonlinear response of the walls, the wall stiffnesses were set at 60 percent of the initial for the first floor only, corresponding to expected yield regions in the wall. The remaining three floors were set to have 100 percent of the initial stiffness.

Coupling Beams

The shear walls are connected at each floor level through the precast concrete planks as well as the masonry lintel above the door openings. For the sake of this analysis, one precast plank (10 inches deep with topping and 40 inches wide) on either side of the wall was assumed to act in conjunction with the lintels above the doorway. It can be seen from the building elevations that each shear wall is separated from the other

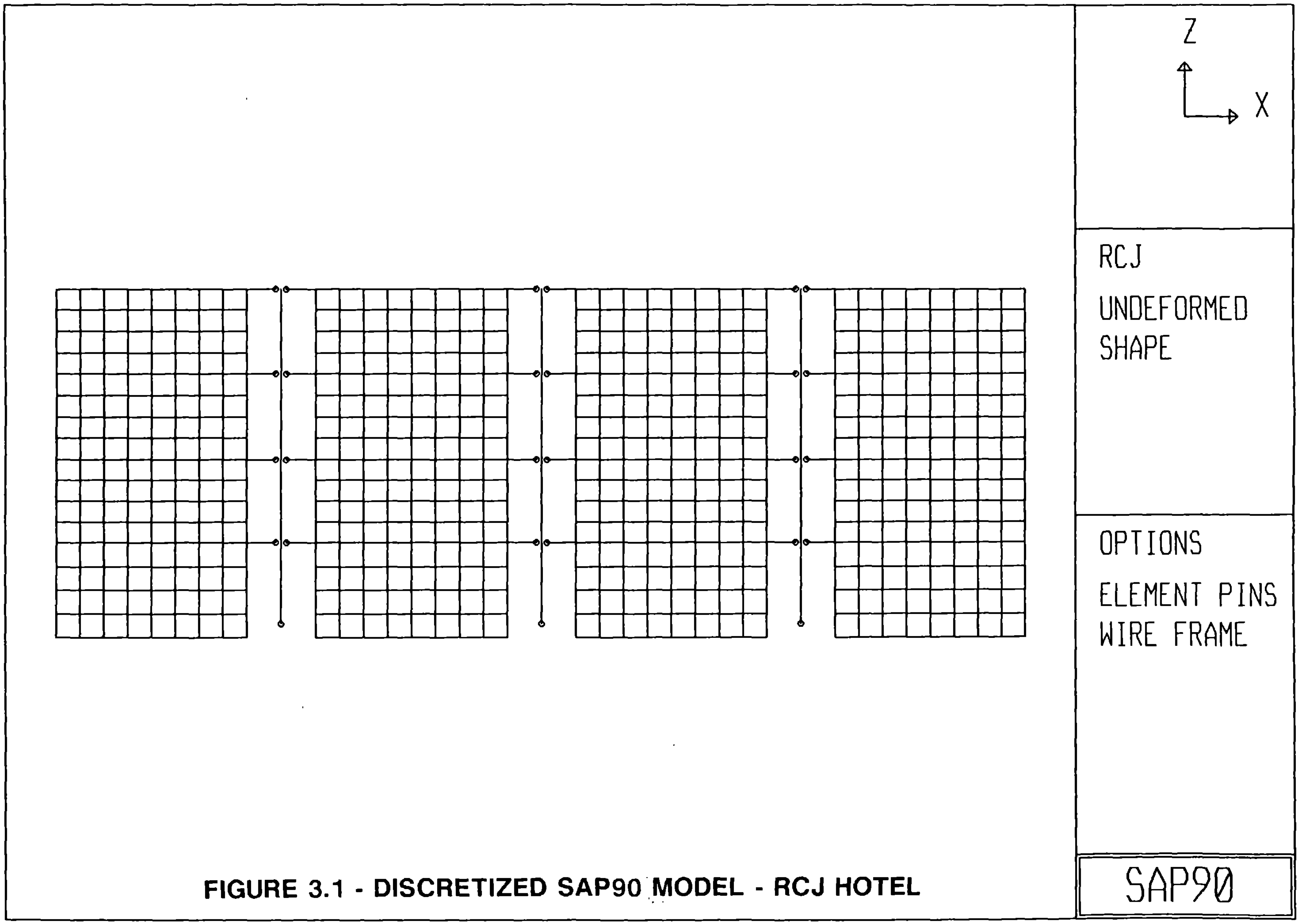


FIGURE 3.1 - DISCRETIZED SAP90 MODEL - RCJ HOTEL

by two doors with a sixteen inch column section in the center. This column line is assumed to act as a point of inflexion for the coupling beam. The beam is assumed to be pinned at the column line. Also, the column is assumed to be pinned at its base.

The precast plank is pretensioned with high strength strands. It also has openings along its length, reducing its net area for an 8 inch by 40 inch section from 320 to 218 square inches. Further the masonry lintel under the plank is physically sawn off from the edge of the shear wall - as per the construction sequence detailed by the TCCMAR Task 2 team. Due to this rather complicated cross section, the stiffness properties were estimated from a nonlinear finite element analysis performed by Ewing and Associates and detailed in the EKEH report 2.2-3 [4]. The location of the strands (1 inch above the bottom of the plank) changes the stiffness characteristics of the coupling beam and makes them dependent on whether the coupling beam bends upwards or downwards. The finite element analysis performed by Ewing and Associates [4] yielded plots between the bending moment at the wall-beam interface versus the rotation at the beam-column interface. These plots (one for each direction of rotation) are shown in Figures 3.2-A and 3.2-B. These plots were used to estimate the stiffness for a fictitious elastic coupling beam of uniform cross section as follows. The relationship between the bending moment at the far end - M , the rotation at the near end θ and the initial stiffness EI for the case of this coupling beam is given by:

$$(M/\theta) = 2\ EI/L$$

where L is the length of the beam. From these curves, an estimate of the initial EI value for the beam was made for upward and downward rotation. The average of these two values was used in the SAP90 model as an approximation to the true beam stiffness. The beam strengths were also noted for determination of appropriate force/deformation levels. The stiffness values were estimated at the "yield" moment for the beam with an assumption that the beams would not sustain any more load/deformation.

3.3 Results of Elastic Time History Analysis

The time history runs were performed on the SAP90 model of the RCJ hotel for five different time histories - records 1,2,4,6 and 9. The earthquake records used in this study are described in Appendix A. For each time history, one run was performed - for east-west motion only. All the time histories were scaled for a seismic zone with a ZPA of 0.4g.

The response quantities that were looked at in this analysis were the floor displacements, the interstory drifts and the base shears. The drifts are important since

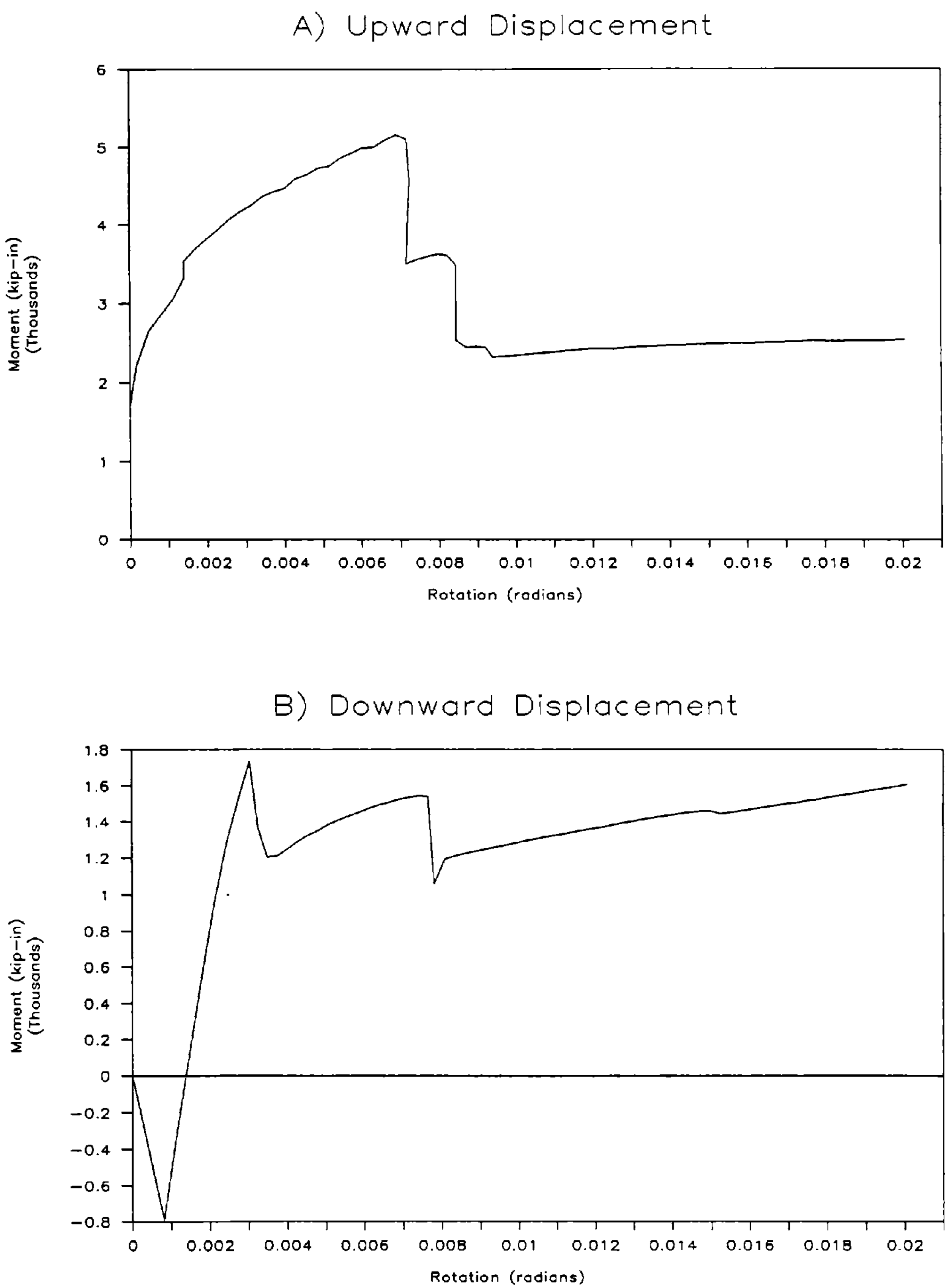


Figure 3.2 Moment-Rotation Curve, Coupling Beam #1

they convey an overall sense of the damage level for the building. The base shears can be compared to those actually used in the building design. An eigenvalue analysis of the hotel was performed as a preliminary to the time history analysis. Table 3.1 gives the first five periods of the building. Figures 3.3 - 3.4 show the first two modes of the hotel.

Table 3.2 shows the results of the elastic time history analysis of the RCJ hotel. The base shear coefficient is in the region of seventy percent - a fairly high value. The roof drift is in the region of 0.1 percent indicating low damage. It was noted early on in the analysis that all of the coupling beams yielded early on in the loading. The maximum rotation of the beam ends is also indicated in Table 3.2 - these values are two times the rotation corresponding to the beam yield. Figures 3.5 - 3.9 show the time history of the roof displacement for the five earthquake records.

The roof accelerations are very high, most being over 1g. In actuality, the structure will soften and the actual roof acceleration should be less as obtained from a nonlinear analysis. A preliminary examination of the results indicates that the building will suffer damage primarily due to the coupling beam action, these beams failing first. The analysis is not consistent since once the coupling beams fail, the structure itself changes - a linear elastic program cannot take this into account. Also, the coupling beams should be able to resist the shear corresponding to the yield moment in order for them to reach their yield moment capacity. The rather brittle moment-rotation behavior evidenced in Figures 3.2-A and 3.2-B indicates that the failure of the coupling beams will not substantially delay the onset of wall flexural failure degrading the overall system ductility. Therefore, the design criteria should account for the possibility of early failure for the lintels over door openings. It should also be noted that if the coupling beams are brittle, then the overall ductility of the structure is decreased and hence the essential character or composition of the beams must be a factor in design also.

REFERENCES

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

2. Kariotis, J.C., and Waqfi, O.M.,"Trial Designs Made in Accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings", Kariotis and Associates Report 9.1-2, Kariotis and Associates, South Pasadena, CA.

3. Wilson, E.L. and Habibullah, A., "SAP90 - A Series of Computer Programs for the

Static and Dynamic Finite Element Analysis of Structures, User's Manual", Computers and Structures, Inc., Berkeley, California, 1989.

4. Ewing, R.D, "Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard", EKEH Report 2.2-3, Ewing and Associates, Rancho Palos Verdes, California.

Table 3.1 Natural Periods
for the RCJ Hotel

Mode	Period (sec)
1	0.217
2	0.060
3	0.031
4	0.023
5	0.017

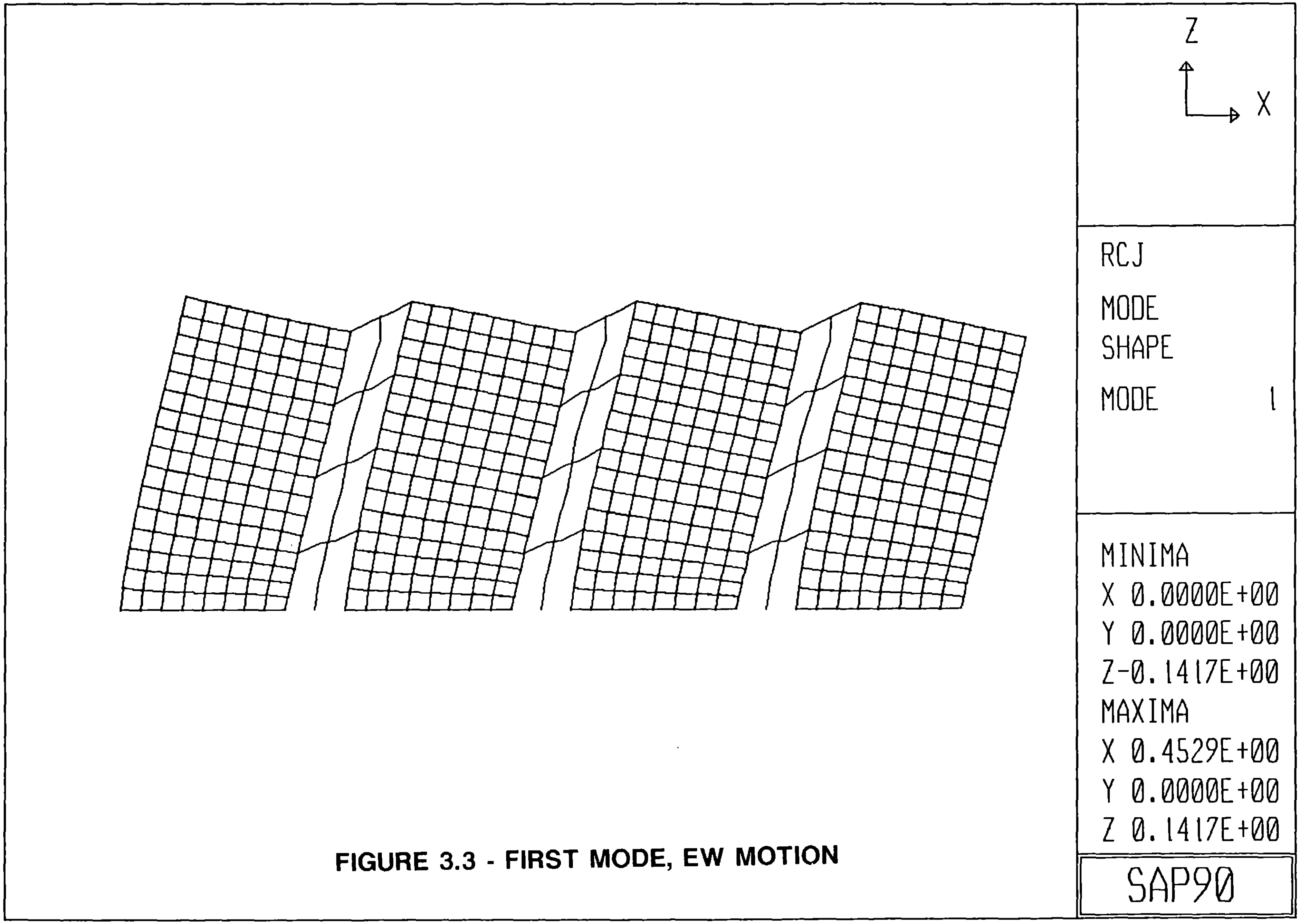


FIGURE 3.3 - FIRST MODE, EW MOTION

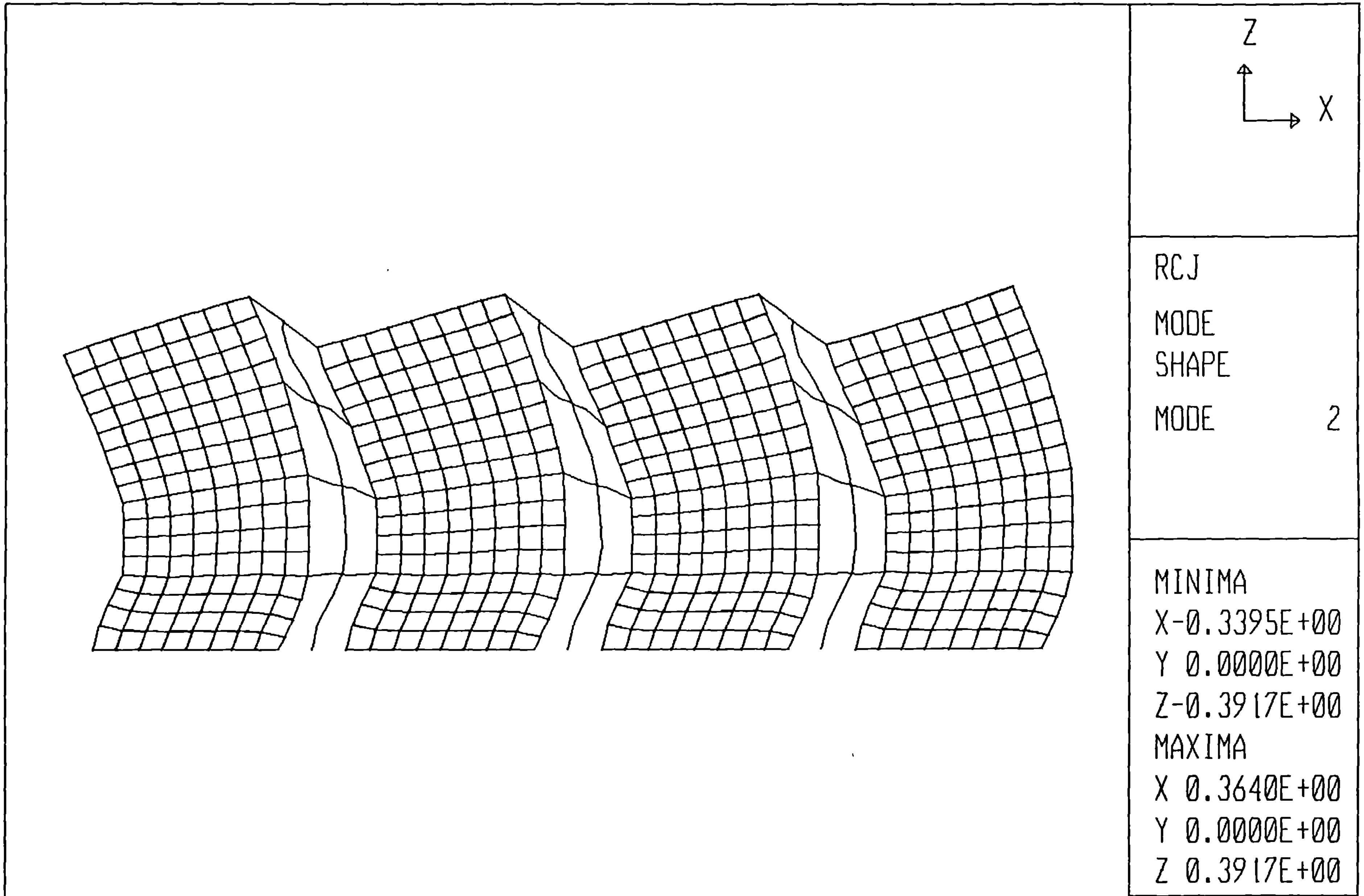


FIGURE 3.4 - SECOND MODE, EW MOTION

Table 3-2 Elastic Analysis Results

Using SAP90

EQ	Overall Drift (%)	Roof Drift (%)	4th Fl. Drift (%)	3rd Fl. Drift (%)	2nd Fl. Drift (%)	Roof Disp. (inches)	4th Fl. Disp. (inches)	3rd Fl. Disp. (inches)	2nd Fl. Disp. (inches)	Base Shear V/W (%)	Roof Accel. (g)	Max. Bm Rotation (rad)
G1	0.123	0.117	0.142	0.125	0.108	0.57	0.43	0.27	0.11	73	1.02	0.0108
G2	0.112	0.108	0.124	0.109	0.108	0.52	0.39	0.25	0.11	68	0.96	0.0124
G4	0.123	0.117	0.133	0.125	0.118	0.57	0.43	0.28	0.12	79	1.37	0.0162
G6	0.116	0.108	0.133	0.117	0.108	0.54	0.41	0.26	0.11	74	1.21	0.0180
G9	0.119	0.117	0.133	0.117	0.108	0.55	0.41	0.26	0.11	70	1.22	0.0145
Avg.	0.119	0.113	0.133	0.119	0.110	0.55	0.41	0.26	0.11	72.8	1.16	0.0144

Fig 3.5 Displacement at Roof

El Centro E-W Component (G1.DAT)

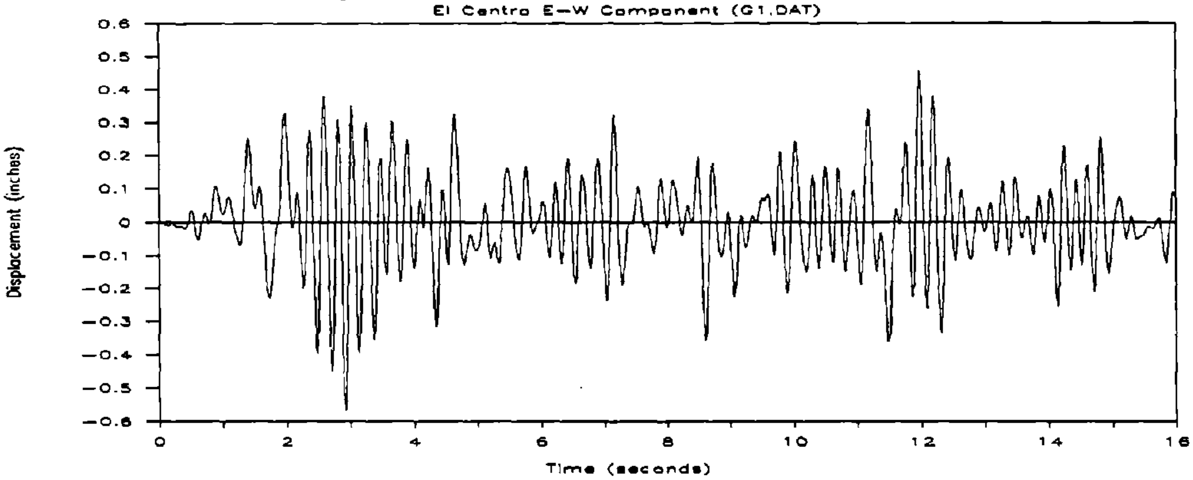


Fig 3.6 Displacement at Roof

El Centro N-S Component (G2.DAT)

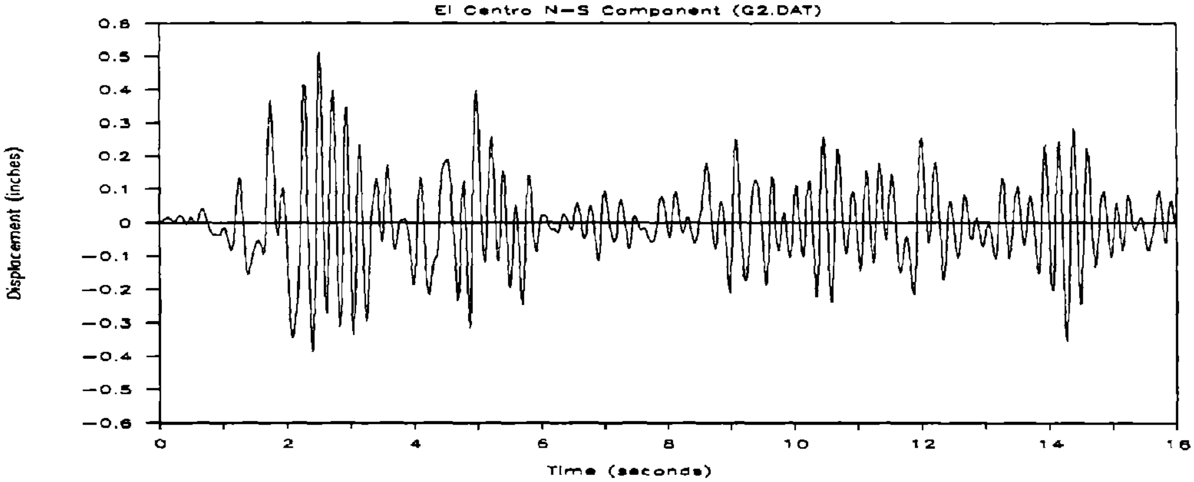


Fig 3.7 Displacement at Roof

Cruickshank Road, 230 (G4.DAT)

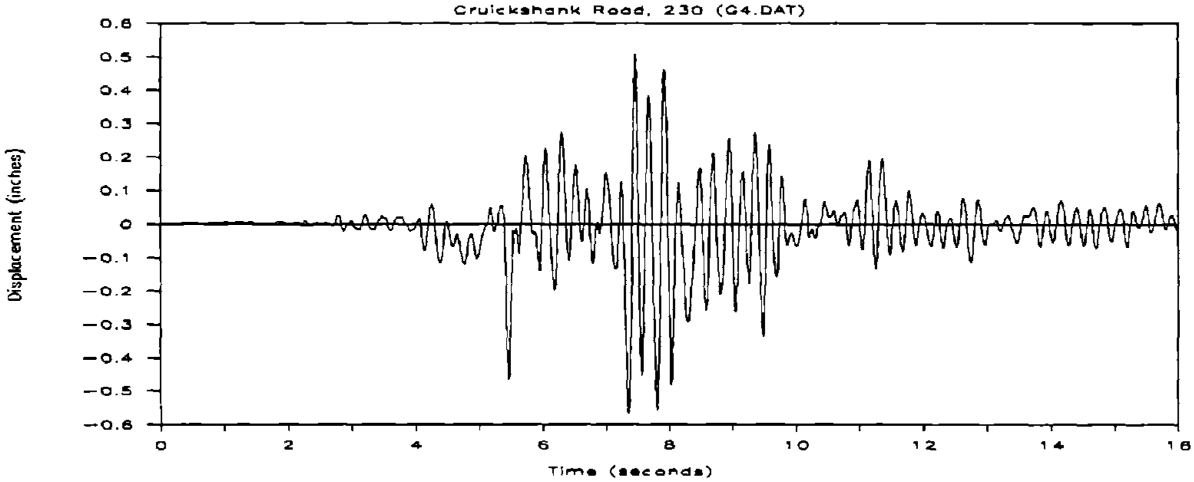


Fig 3.8 Displacement at Roof

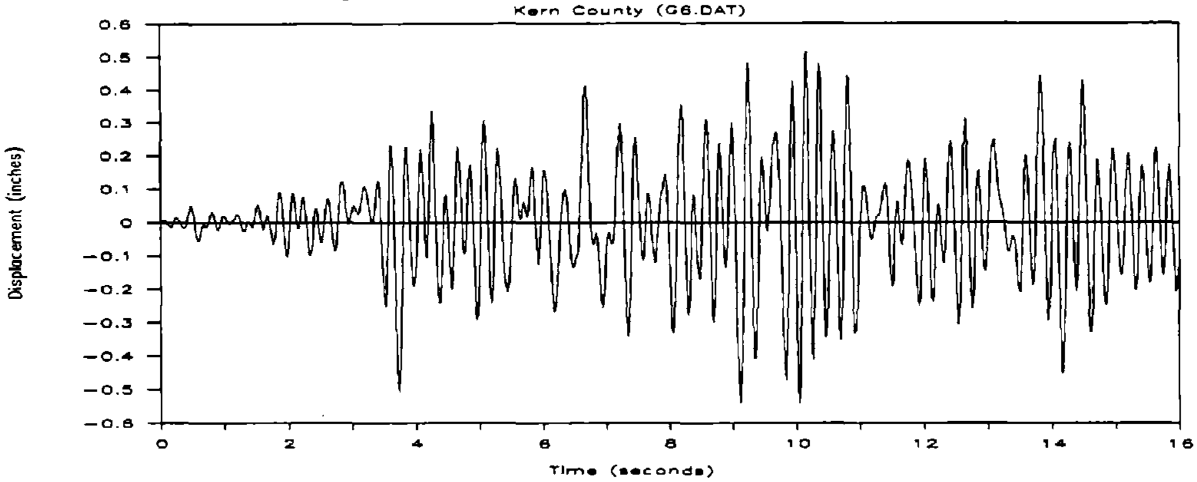
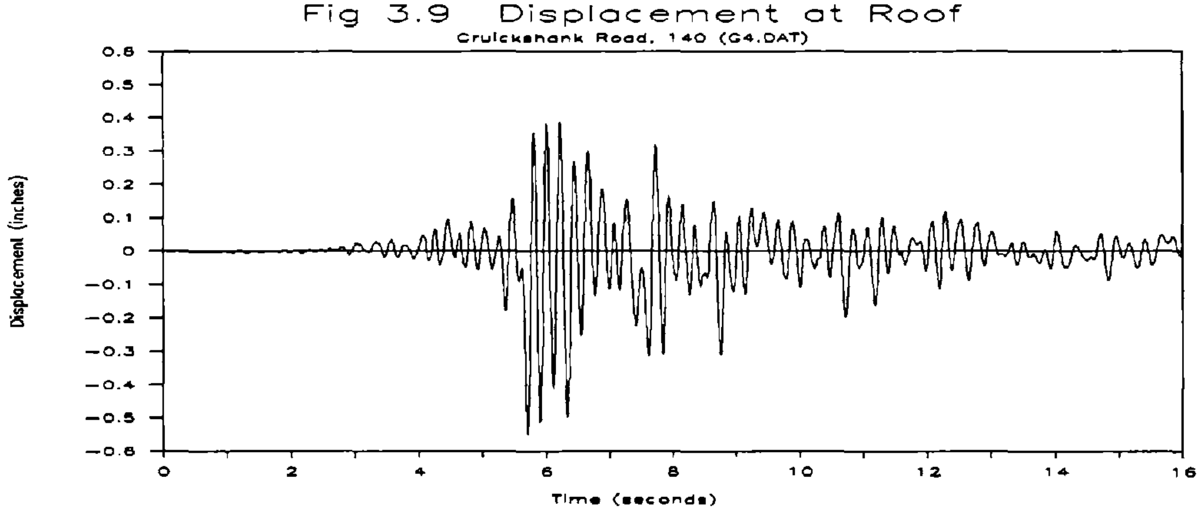


Fig 3.9 Displacement at Roof



CHAPTER 4
EXPECTED BUILDING PERFORMANCE USING DRAIN-2DX

4.1 General

As discussed earlier, the primary lateral load resisting system of the RCJ Hotel in the EW direction is a set of shear walls connected by coupling beams, which consist of a precast plank slab system with masonry lintels. The objective of this research were to examine how the in-plane seismic resistance of muti-story concrete masonry walls is affected by wall openings and floor system. Thus, nonlinear time history analyses were made for the ground motions in the EW direction to study the performance of the coupled wall system using the two-dimensional computer program DRAIN-2DX [1]. DRAIN-2DX is a new generation of PC-based general purpose computer program for static and dynamic analysis.

4.2 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. Yield moments and stiffness properties for the bilinear springs at the member ends were obtained from moment curvature relations. The stiffness of a member was the slope of the first line segment in the bilinear moment curvature curve, which can be approximated by utilizing effective moment of inertia for the member. The DRAIN-2DX analytical model for the RCJ Hotel is shown in Figure 4.1, where the coupled wall system was modeled as a planar frame. Shear walls were represented by columns placed at the centroids of the wall sections. The strength of the shear walls were computed based on expected material properties of concrete masonry and the reinforcing steel. The stiffness of shear walls were set at 60 percent of the initial stiffness calculated based on gross sections for the first floor only, corresponding to expected regions in the wall. The remaining portions of the walls were set to have initial stiffness. Coupling elements were represented by beams rigidly connected to the walls with the rigid ends of coupling beams equal in length to half of wall length. The strength and stiffness characteristics of the coupling beams were estimated from nonlinear finite element analyses performed by Ewing and Associates [2]. The analyses yielded moment rotation relationships for coupling beam as shown in

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ______
DATE ______
DESIGN ______
SHEET# ______

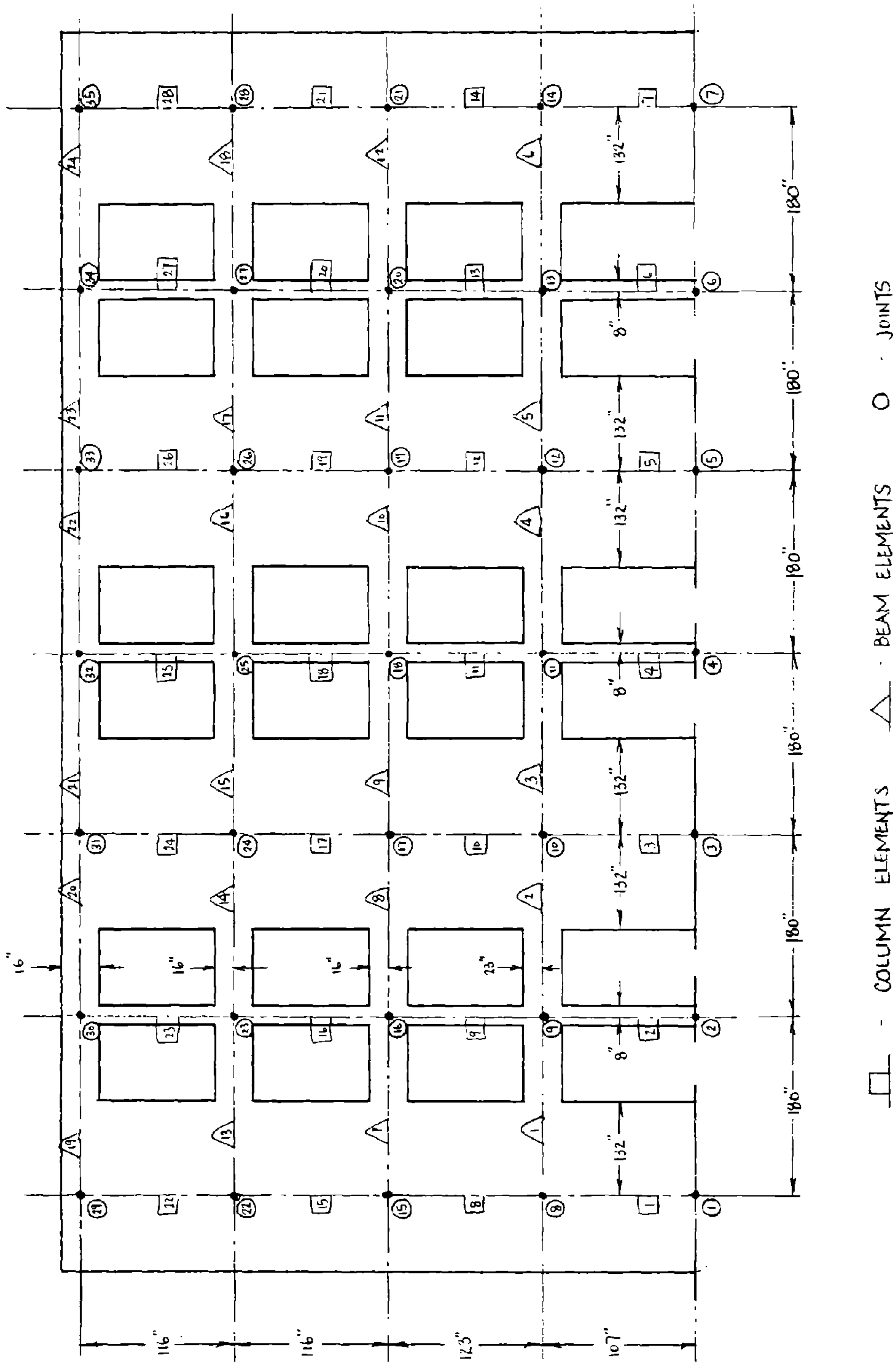


Figure 4.1 DRAIN-2DX Analytical Model

Figures 3.2-A and 3.2-B. From these curves, the equivalent bilinear moment rotation curves were made and in turn an estimate of the effective EI values for the coupling beam can be obtained for positive and negative bending. The average of these two EI values was used as an approximation of the coupling beam stiffness to model the load reversal effect on the member. It is noted that the EI values were determined at the yield rotation of 0.007 for positive and negative bending with the assumption that the beam would not sustain any more load/deformation, owing to a greater strength degradation for the beam under positive bending. However, different yield moments were used for positive and negative bending. As shown in Figure 4.1 , the coupling beam spanning between two shear walls is intersected by a door jamb at the middle of the beam. This jamb column provides a support to the beam and act as a point of inflection for the coupling beam. Therefore, the coupling beams were assumed to be pinned at the column whereas the columns and shear walls were assumed to be fixed at their base.

The DRAIN-2DX model for the coupled shear wall system 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 mode, 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.

4.3 Static Behavior State Analysis

An inelastic static behavior state analysis was performed to predict the strength of the coupled shear wall. Such a static analysis provides information on the actual strength and the location and sequence of plastic hinge formation. Furthermore, this static analysis is used as background information and serves as a check on the dynamic analysis.

The behavior state analysis used a lateral load with an inverted triangular load pattern consistent with the LSDS [3] seismic load distribution equation. The results of the analysis is shown in Figure 4.2 in terms of base shear versus roof displacement. It is noted from Figure 4.2 that the computed base shear strength of the coupled shear wall is 38% of the building weight. The design base shear as computed is 22.2% of the building weight. Considering a strength reduction factor of 0.85 required by the LSDS, a nominal base shear strength of 26.1% of the weight is estimated for a code design. Thus, the computed base shear strength is about 45% greater than the code strength for the inverted triangular load distribution. Thus, it can be concluded that the structural system strength is stronger than the minimum nominal strength resulting from the code requirements.

Figure 4.3 shows the location and the sequence of plastic hinge formation for the two frames. It can be observed from Figure 4.3 that the plastic hinge formed at coupling beam ends and at the base of the shear walls exactly as envisioned in the development of the design criteria. The plastic hinges formed earlier at ends of coupling beams than at base of shear walls further indicating that the coupling beam did behave as intended to delay the yielding of shear walls.

4.4 Results of Inelastic Time History Analysis

The elastic analysis results in Table 3.2 indicated that inelastic responses should occur for coupled wall system for ground motion 1,2,4,6 and 9 considered. 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 these earthquakes as described in Appendix A. The structural system responses considered in this study are base shear, roof acceleration, relative story displacements and drift ratios. 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 4.1 summarize the computed structural responses for the coupled wall system. Figure 4.4 shows the roof displacement time history for the earthquakes considered.

The maximum overall drift ratio as shown in Table 4.1 is about 0.2% which is well below the drift limit due to the much stiffer of shear walls in the structural system. Tables 3.2 and 4.1 give a comparison of the relative displacement for elastic and inelastic analysis. It is implicitly implied in the equal displacement design criteria approach that the

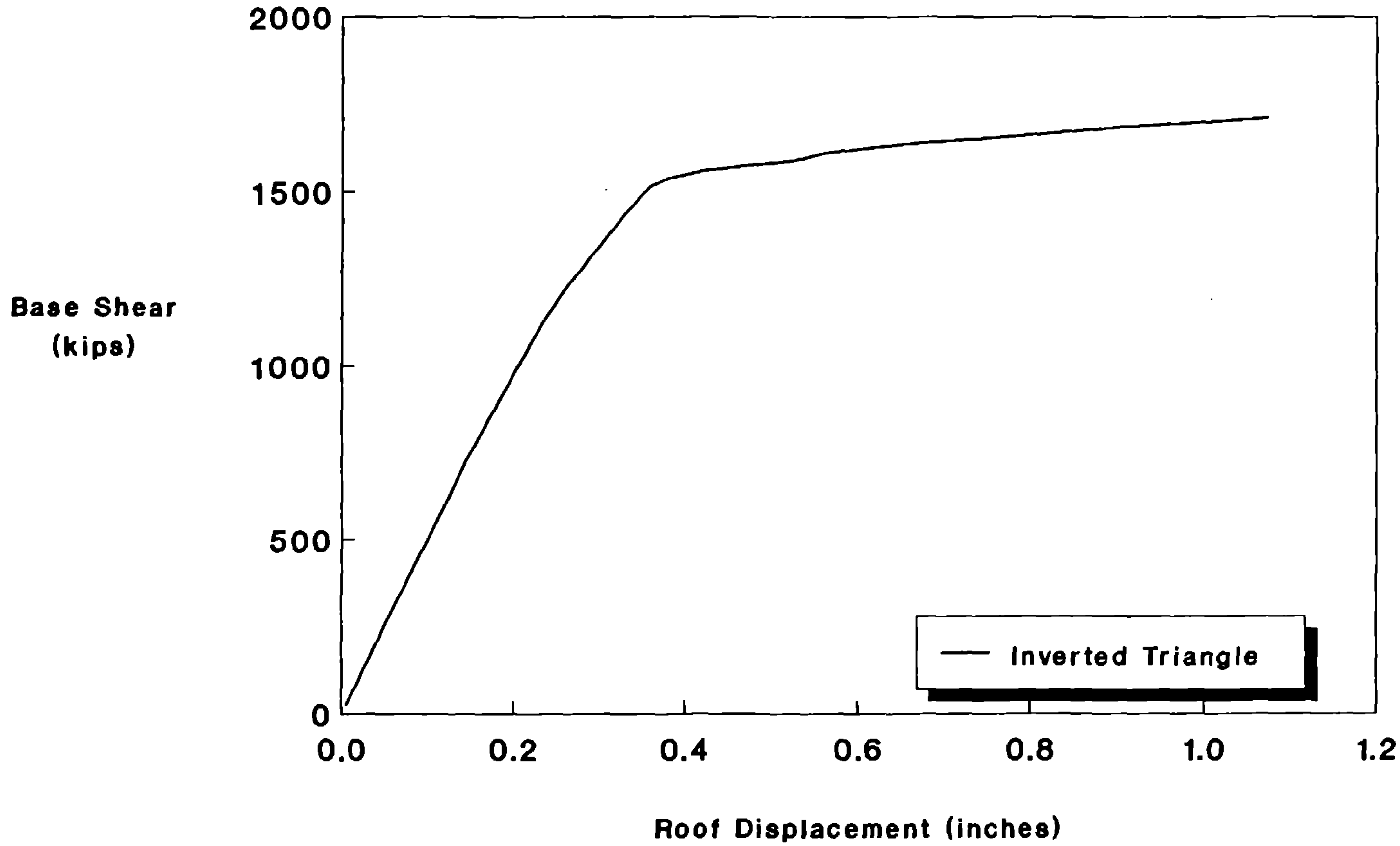
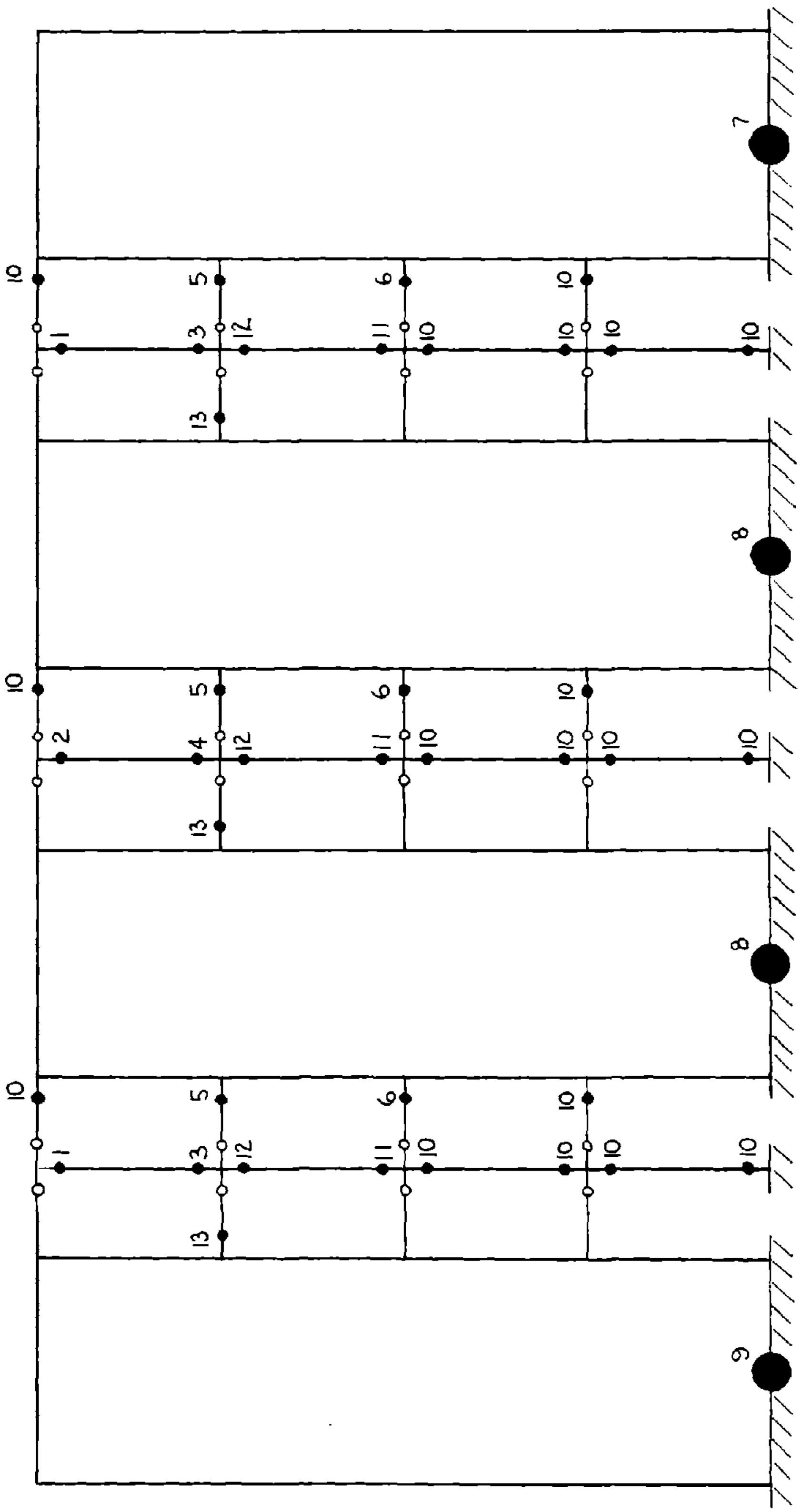


Figure 4.2 Load-Deformation Response of Coupled Shear Wall

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ________
DATE ________
DESIGN ________
SHEET# ________



o REAL HINGE

• PLASTIC HINGE

Figure 4.3 Location and Sequence of Plastic Hinge Formations

Table 4.1 Inelastic Analysis Results

EQ	Overall Drift (%)	Roof Drift (%)	4th Fl. Drift (%)	3rd Fl. Drift (%)	2nd Fl. Drift (%)	Roof Disp. (inches)	4th Fl. Disp. (inches)	3rd Fl. Disp. (inches)	2nd Fl. Disp. (inches)	Base Shear V/W (%)	Roof Accel. (g)	Max. Bm Rotation (rad)
G1	0.084	0.133	0.133	0.101	0.069	0.51	0.35	0.20	0.07	39.0	0.83	0.0174
G2	0.108	0.148	0.137	0.105	0.083	0.50	0.36	0.22	0.09	45.6	0.95	0.0196
G4	0.178	0.222	0.211	0.183	0.165	0.82	0.62	0.40	0.18	50.3	1.24	0.0279
G6	0.179	0.184	0.193	0.179	0.160	0.82	0.62	0.40	0.18	47.6	0.73	0.0250
G9	0.202	0.217	0.223	0.195	0.170	0.93	0.68	0.42	0.18	48.4	0.96	0.0245

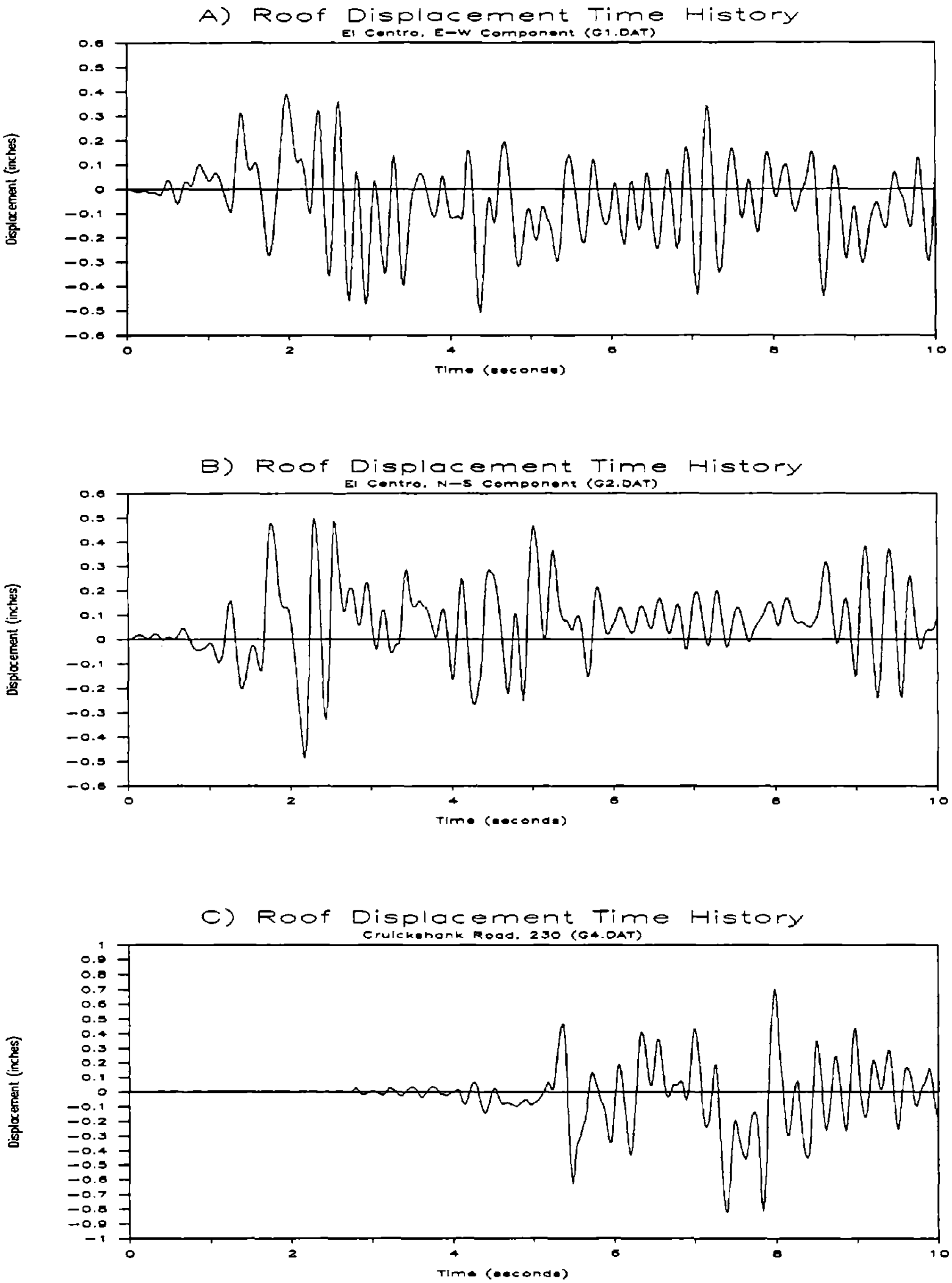
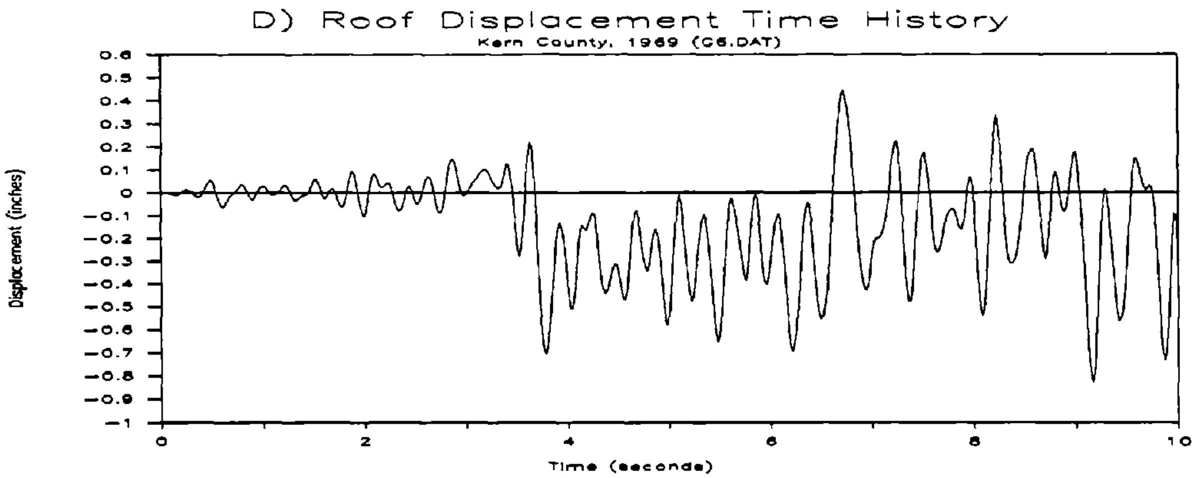


Figure 4.4 Roof Displacement Time History



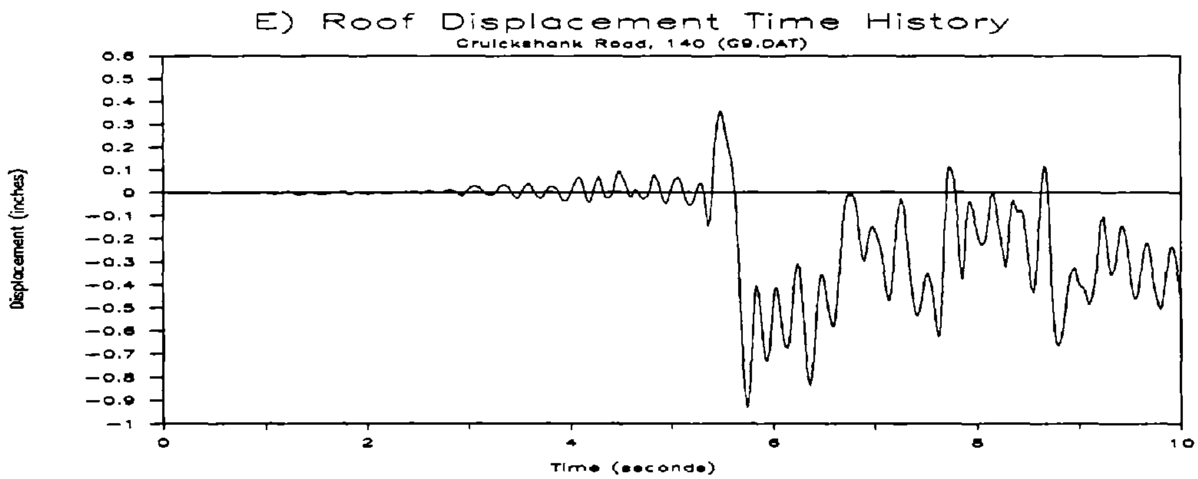


Figure 4.4 (Concluded)

displacement for the two analyses are identical. However, the ratios of displacements from the two analyses significantly deviate from the unity depending on the earthquake applied and can be seen to vary between 0.7 and 1.7.

The ratio of maximum base shear from an elastic analysis (Table 3.2) to that from an inelastic analysis (Table 4.1) can be used to provide insight into reasonable design values for response modification factors, i.e. R factor as defined in the NEHRP document [4]. The ratios typically vary with the different ground motion with an approximate value of 1.5 for earthquakes considered. It can be shown that the ratio of elastic analysis to inelastic analysis for the roof acceleration is approximately equivalent to the base shear ratio. The ratios also vary with earthquake intensity level.

The maximum rotations experienced for the coupling beams are also listed in Table 4.1. These values are about 2 to 4 times greater than the rotation corresponding to the beam yield, indicating that the coupling beams have gone into inelastic range with limited ductility. The moment rotation behavior of the coupling beam was rather brittle as evidenced by the significant strength degradation shown in Figure 3.2. This will reduce the effectiveness of the coupling beam, and thus not allow the coupled wall system to deform a great amount after the yield.

4.5 References

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.

2. Ewing, R. D., "Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard," EKEH Report 2.2-3, Ewing and Associates, Rancho Palos Verdes, California.

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

4. NEHRP, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, FEMA, Washington D.C.

CHAPTER 5
EXPECTED PERFORMANCE OF BUILDING WITH UNCOUPLED SHEAR WALLS

5.1 General

The lateral force resisting system in the RCJ Hotel as envisioned is a coupled shear wall system. The performance of the coupled walls is governed by the characteristics of individual shear walls and also the effectiveness of the coupling systems between the walls. Under lateral loads, the coupled system develops shears and moments and transmits these to the walls. The shear transferred between the walls varies with the strength of the coupled system, which in turn depends on the coupling effectiveness of the floor system with lintels. If the coupling system is weak, the amount of shear transfer between walls will be small and thus the walls behave more like independent cantilevers. In other words, the coupling wall system becomes an uncoupled wall system. Thus, this chapter is focused on the design and the performance assessment of these uncoupled shear walls.

5.2 LSDS Design of Uncoupled Shear Wall

A design of the uncoupled shear wall system in the East-West direction of the RCJ Hotel was performed using Limit States Design Standard [1]. The four identical walls were assumed to be uncoupled and to act independently of each other with each wall resisting one quarter of the total seismic force. Detailed design calculations are shown in Appendix B and a brief summary of the design is outlined below.

The structure was initially assumed to have a fundamental period of vibration of less than 0.5 seconds. With this assumed period, the base shear force and distribution of lateral force to each floor were determined. The computer program SAP90 [2] was then used to calculate the displacement at the each floor of the building under the applied lateral forces using an effective moment of inertia of the wall recommended by Priestly and Hart [3] to characterize the effective stiffness of the building system. The results of the SAP90 analysis were then used in Rayleigh's Equation [4] to obtain a more accurate estimate of the fundamental period of vibration. With the new fundamental period of vibration, a new base shear force and distribution of lateral force to each floor were obtained. Again, the displacement at each floor were computed to revise the estimate of the fundamental period of vibration. This procedure was repeated until the period of vibration of the wall converged. The analysis resulted in a fundamental period of vibration of 0.59 seconds and a base shear of 247 kips on each wall.

The base of the wall was designed to resist the overturning moment resulting from the static lateral force applied at each floor level. The expected compressive stress of the masonry was assumed to be 2500 psi and the expected yield stress of the steel was assumed to be 66 ksi. An unconfined stress-strain curve with an ultimate strain of 0.0026 was used for the masonry. The flexural capacity was calculated using the computer program IMFLEX [5] and compared with the applied moment. Vertical reinforcement of #8 bars at 16 inches on center was found to be adequate.

To ensure that a flexural yield limit state occurred before a shear yield limit state the base of the wall was designed to have a shear capacity greater than the ductile shear force. The ductile shear force is the base shear force corresponding to the development of the flexural capacity at the base of the wall. The shear capacity of the wall was computed with the equation recommended by Fattel [6] and horizontal reinforcement of #6 bars at 8 inches on center was determined to be adequate.

The reinforcement configuration was changed at the third floor. The flexural capacity and shear capacity of the wall at that level were designed to exceed the forces at that level corresponding to the ductile base shear force. This was to ensure that a plastic hinge could only occur at base of the wall. The resulting design was #5 bars at 16 inches on center for vertical reinforcement and #4 bars at 8 inches on center for horizontal reinforcement.

5.3 Results of SCM Analysis

Analysis to evaluate the performance of the uncoupled shear wall was performed using computer program IMFLEX. In the IMFLEX analysis, the behavior of the wall was assumed to be controlled by the flexural capacity due to the fact that the wall was designed with shear capacity larger than the shear force associated with the development of the flexural strength. It was further assumed that the development of plastic hinge at the wall base. The wall was assumed to rotate about their compression toe and all deformation were assumed concentrated at the plastic hinge. The plastic hinge length was assumed to be equal to one-half the wall length.

A realistic moment curvature was developed considering the expected axial force level, expected material properties of concrete masonry and the reinforcing steel. Figure

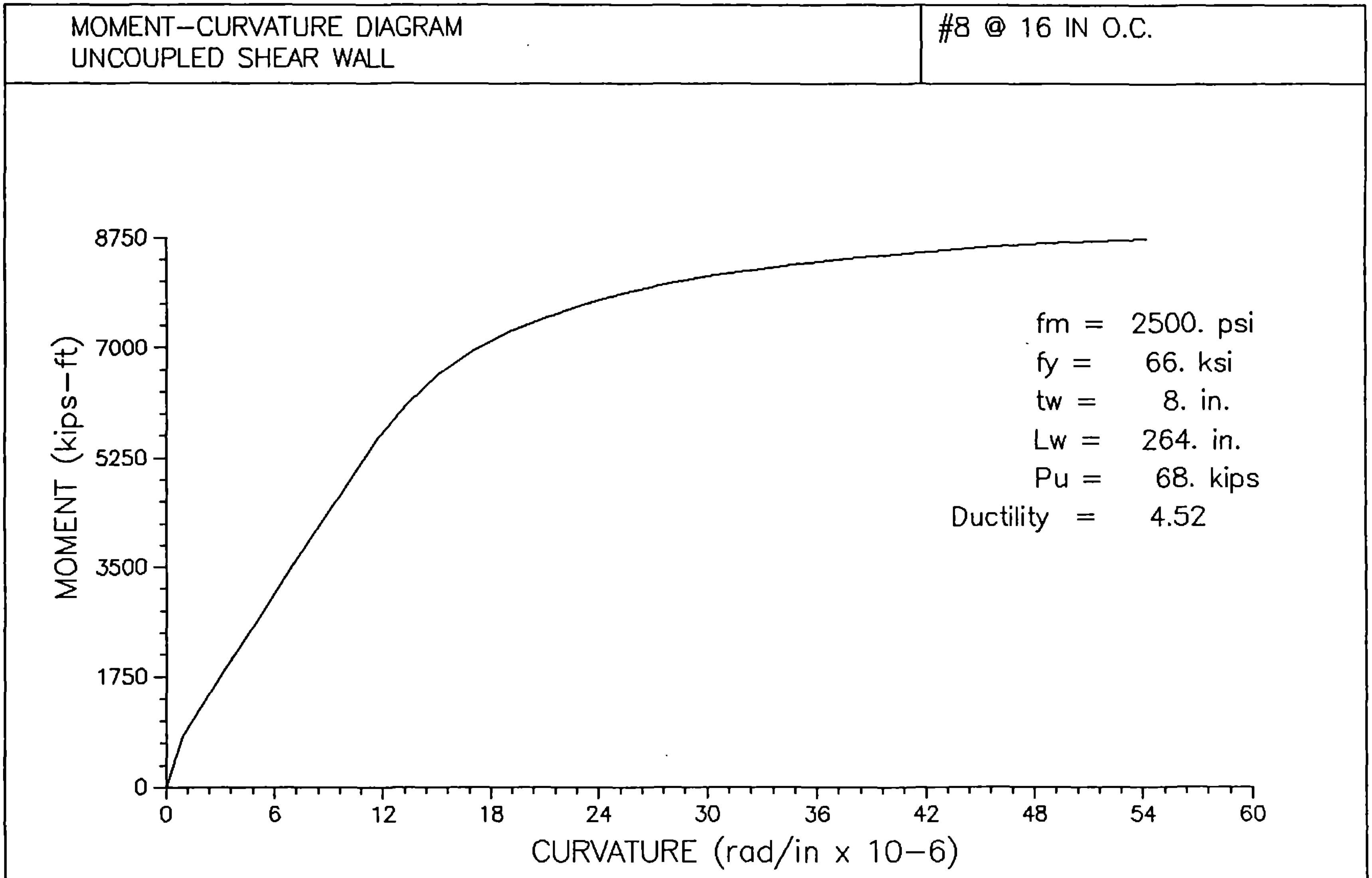


Figure 5.1 Moment–Curvature Relation of Uncoupled Shear Wall

5-1 shows the analysis results with the 1.81 inches displacement at the top of the wall equivalent to the displacement ductility of 4.5. The corresponding flexural capacity of the wall base section is 8730 k-ft. Since there is no axial force due to the coupling system, the moments at the bases of walls are equal. This results in a overturning moment capacity of 34920 k-ft for uncoupled shear walls. The corresponding base shear is 1325 kips. The base shear for the coupled wall system, as shown in Figure 4-2, was 1710 kips corresponding to 1.1 inches roof displacement. Comparison of base shear of coupled wall system with uncoupled wall systems indicates that an 29% increase in base shear capacity if the walls were coupled. Therefore, transfer of shears and moments between the walls through coupling system resulted in the greater lateral load and flexural capacity of the system.

5.4 References

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

2. Wilson, E. L. and Habibullah, A., "SAP90 - A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures, User's Manual, "Computers & Structures, Inc., Berkeley, California, 1989.

3. Priestly,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.

4. Uniform Building Code (UBC), "Uniform Building Code, 1991 Edition", International Conference of Building Officials, Whittier, CA 1991.

5. Hart, G.C., Sajjad, N. A., and Basharkhah, M. A., "Inelastic Flexural Shear Wall Analysis Computer Program (IMFLEX; Version 1.01)", January 1989.

6. Fattel, S.G., and Todd, D.R., Ultimate Strength of Masonry Shear Walls: Predictions vs Test Results. National Institute of Standards and Technology, NISTR 4633, Gaithersburg, MD, 1991.

CHAPTER 6
DISCUSSION

The results of the elastic and inelastic analyses have already been presented in Chapters 3 and 4. From a comparison of the results, it is seen that a "pseudo" elastic analysis using SAP90 gives good results as far as the displacement response/drifts are concerned. Even the failure of the coupling beams is predicted. The deficiencies of the SAP90 model are stronger when comparing the base shear coefficients and the beam rotations. As progressive nonlinear behavior cannot be modeled, any cumulative quantity such as rotation that is large for the specific ground motion cannot be modeled accurately with SAP90. Also, failure paths from a static lateral load analysis are not easily predicted with the elastic approximation - areas in which the DRAIN-2DX model is superior. Therefore, the choice of a linear or nonlinear analysis depends on the objectives of the analysis - displacement based responses are predicted fairly well by an elastic approximation to the nonlinear response.

APPENDIX A
EARTHQUAKE GROUND MOTIONS FOR ANALYSIS

Nine sets of ground motion records were chosen for the TCCMAR project by the TCCMAR Task 2 Team. Table A shows a list of the ground motion records along with their salient characteristics. The earthquake time histories are shown in the accompanying figures. A detailed description of these ground motion records is given in Kariotis and Associates Report 9.1-2 [1].

REFERENCES

1. Kariots, J.C., and Waqfi, O.M, "Trial Designs Made in Accordance with Tentative Limit States Design Standards For Reinforced Masonry Buildings", Kariotis and Associates Report 9.1-2, February 1992, Kariotis and Associates, South Pasadena, California.

TABLE A

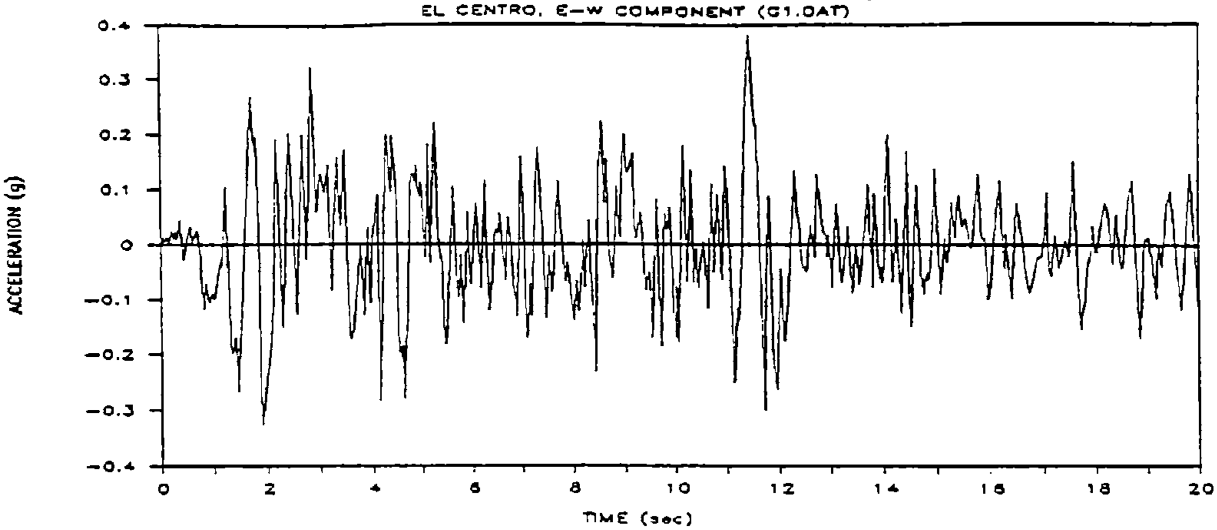
Earthquake Ground Motions

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

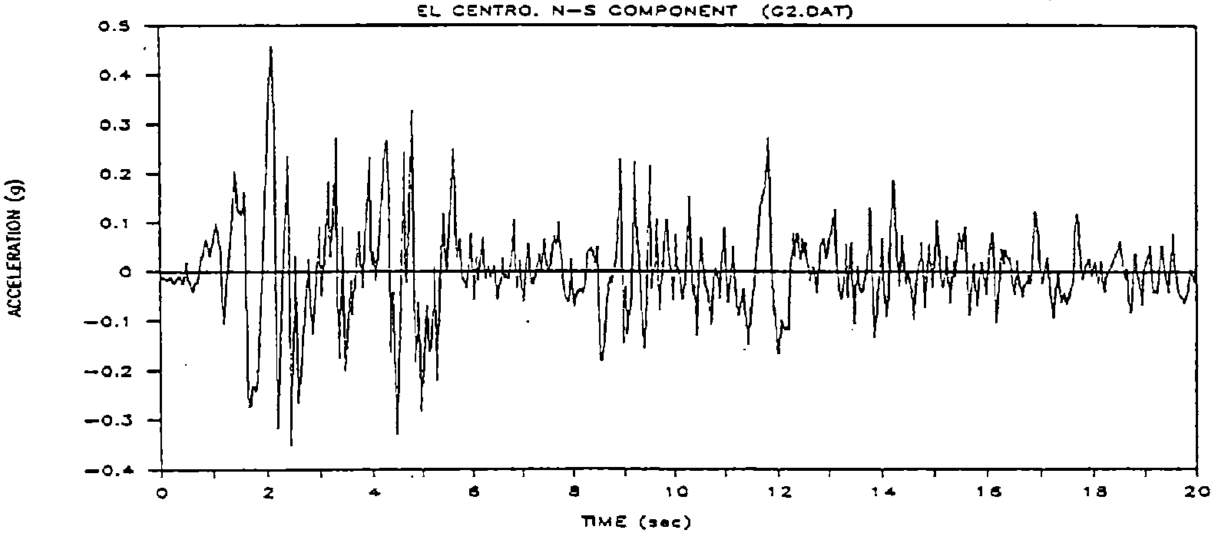
ACCELERATION TIME HISTORY (ZONE 4)

EL CENTRO, E–W COMPONENT (G1.DAT)



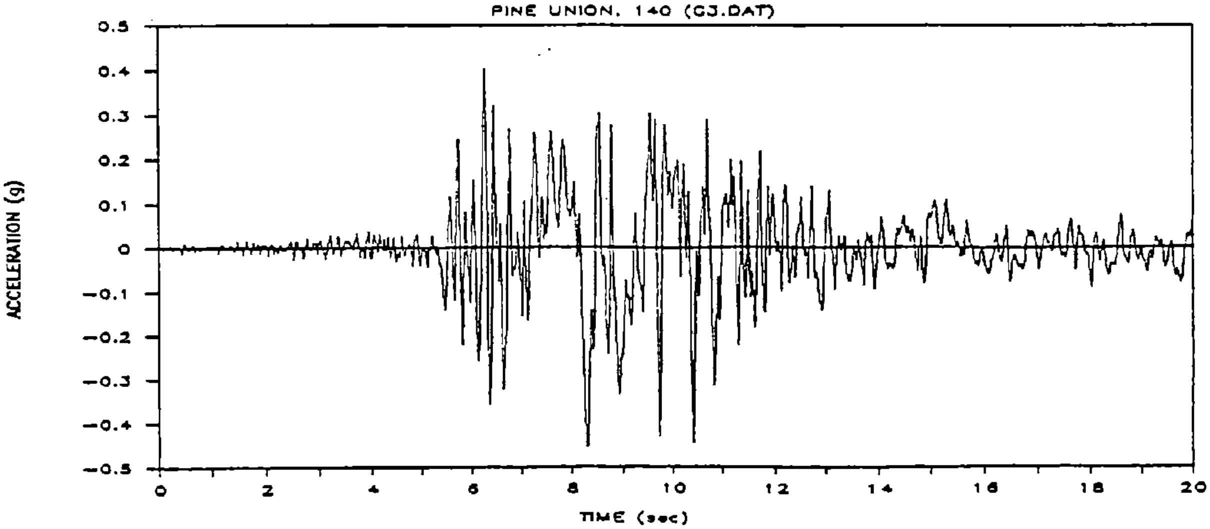
ACCELERATION TIME HISTORY (ZONE 4)

EL CENTRO, N–S COMPONENT (G2.DAT)



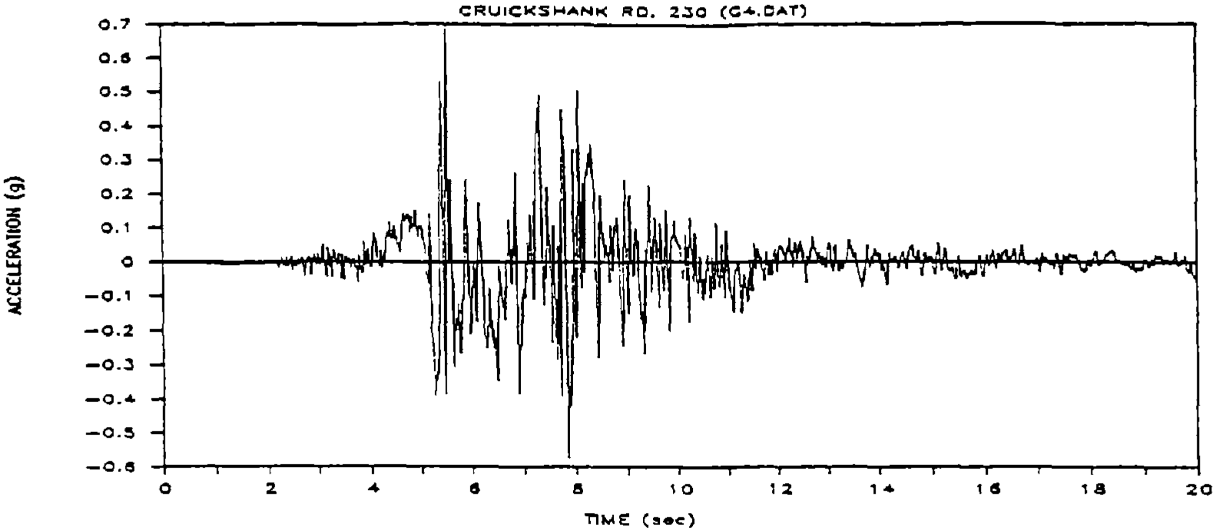
ACCELERATION TIME HISTORY (ZONE 4)

PINE UNION, 140 (G3.DAT)



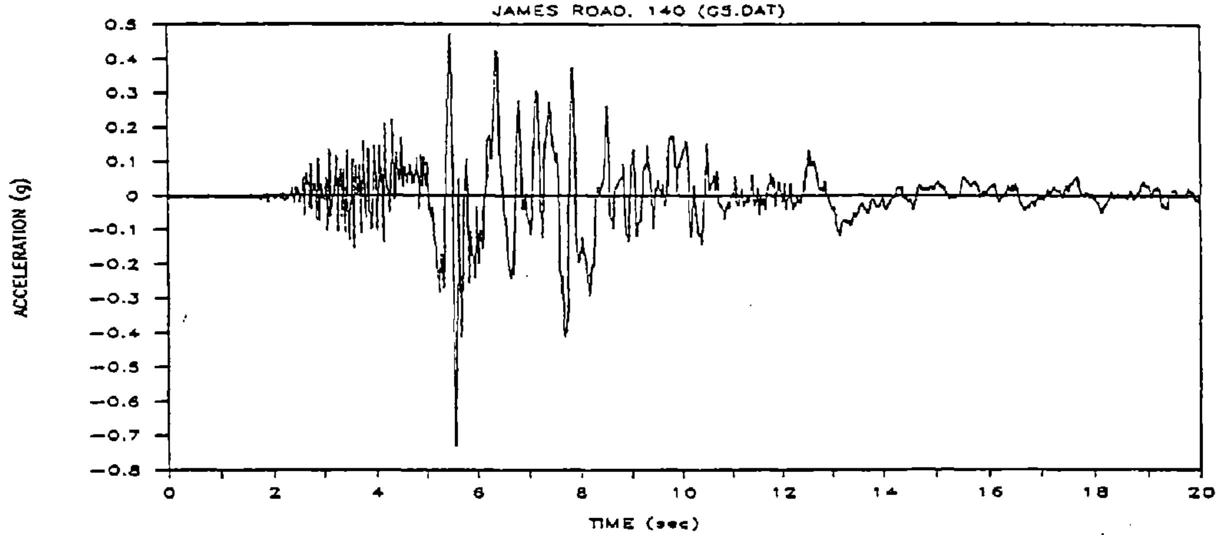
ACCELERATION TIME HISTORY (ZONE 4)

CRUICKSHANK RD. 230 (G4.DAT)



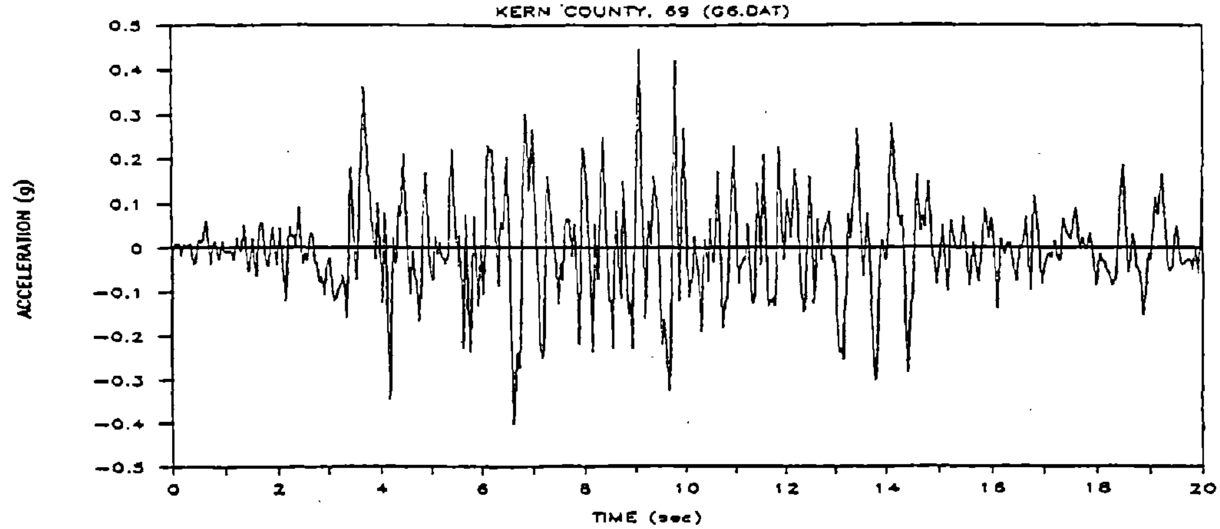
ACCELERATION TIME HISTORY (ZONE 4)

JAMES ROAD, 140 (G5.DAT)



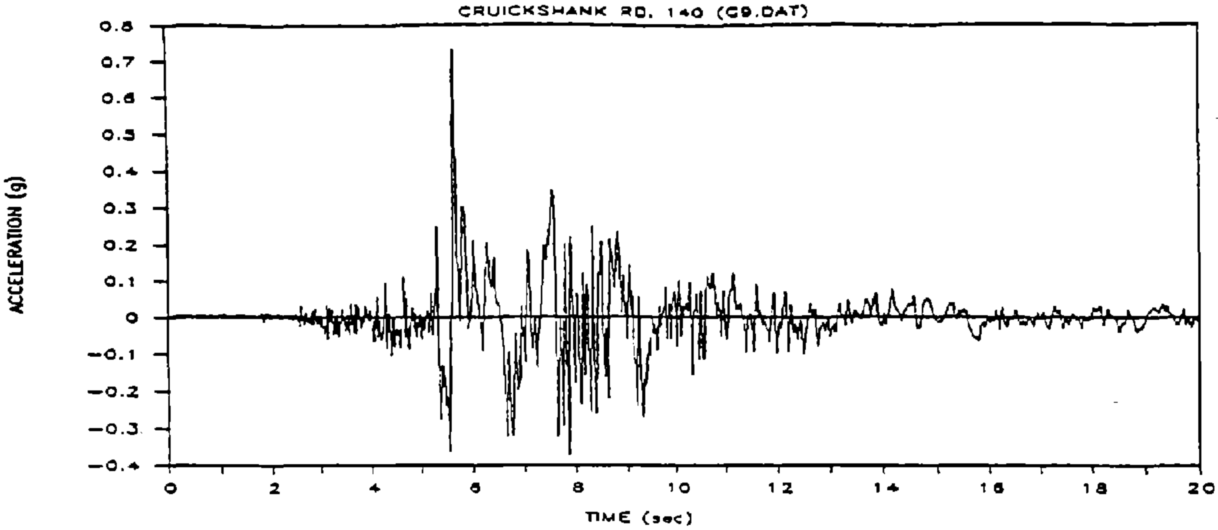
ACCELERATION TIME HISTORY (ZONE 4)

KERN COUNTY, 69 (G6.DAT)



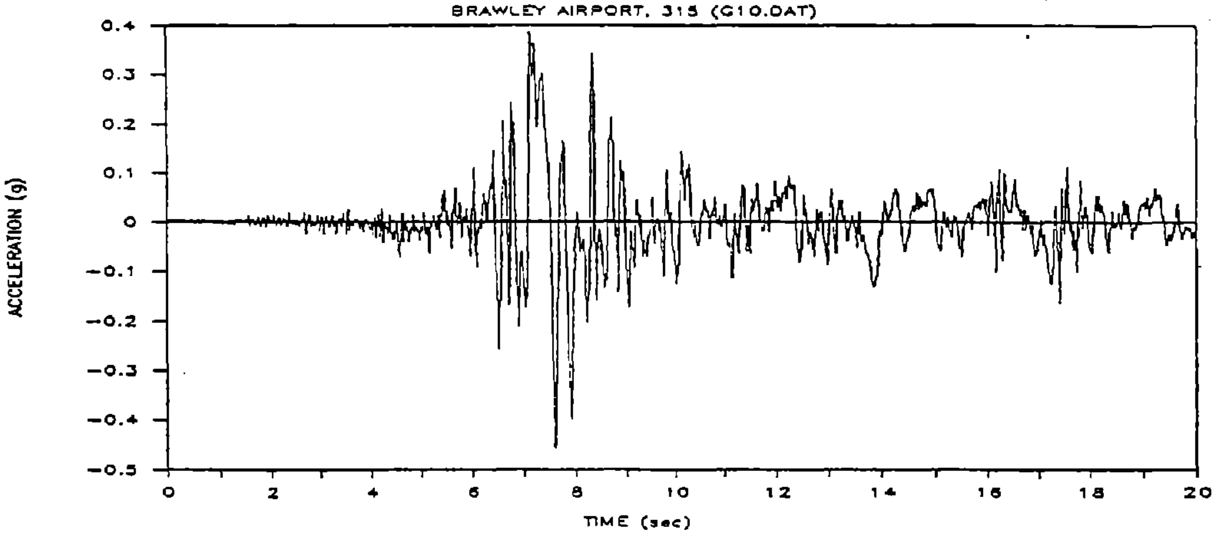
ACCELERATION TIME HISTORY (ZONE 4)

CRUICKSHANK RD. 140 (G9.DAT)



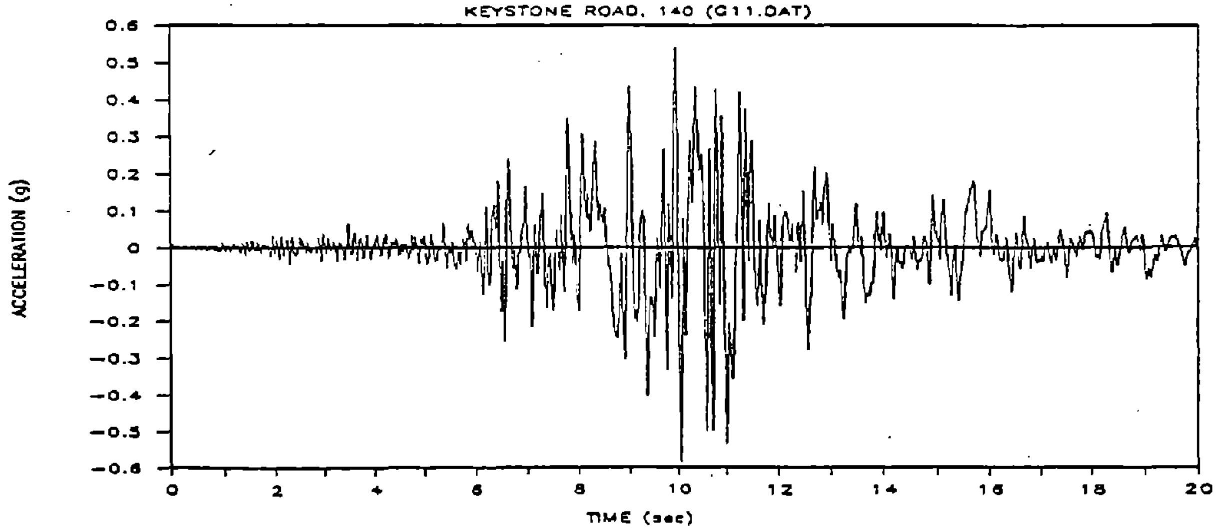
ACCELERATION TIME HISTORY (ZONE 4)

BRAWLEY AIRPORT, 315 (G10.DAT)



ACCELERATION TIME HISTORY (ZONE 4)

KEYSTONE ROAD, 140 (G11.DAT)



APPENDIX B
LSDS DESIGN OF UNCOUPLED SHEAR WALLS

The accompanying calculation sheets detail the LSDS design of the uncoupled shear walls referred to in the earlier chapters of this report.

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB#
DATE
DESIGN
SHEET#

LSDS DESIGN OF UNCOUPLED SHEAR WALL

WEIGHT CALCULATION:

FLOOR & ROOF AREA = (29′8″ + 9′4″ + 28′8″) + (4 × 30′)
= 8120 ft^2

ROOF WT = (AREA) w = 8120 × 0.095 = 771.4^K

FLOOR WT. = 8120 × 0.11 = 893.2^K

WALL WEIGHTS:
(w = 77 psf)

EAST-WEST WALL WT. = [(9′8″ + 120′) − (8 × 7′ × 3′4″)] 0.077
= 75^K

5 NORTH-SOUTH WALLS WT. = 5 (28′8″ + 29′8″)(9′8″)(0.77)
= 217^K

FLOOR	SLAB	E-W WALL	N-S WALLS	TOTAL
R	771.4	40.2	108.5	920.1^K
4	893.2	75	217	1185.2^K
3	893.2	75	217	1185.2^K
2	893.2	84.3	230	1270.5^K
				4498^K

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ________
DATE ________
DESIGN ________
SHEET# ________

TOTAL SEISMIC WT. FOR BLDG = 4498^K

BASE SHEAR, $V = C_s W$ (NEHRP, 1991)

$$C_s = \frac{S_{a(1.0)}\, S}{R\, T_{es}^{n}}$$

$S_{a(1.0)}$ = 0.58

S = 1.0

n = 1.0 (ASSUME $T_{es} \leq 0.5$ sec)

R = 4.5

THEN FOR $T_{es} = 0.5$ sec

$$C_s = \frac{(0.58)(1.0)}{(4.5)(0.5)} = 2.58 > \frac{S_{a(0.3)}}{R} = \frac{1.0}{4.5} = 0.222$$

USE $C_s = 0.222 \Rightarrow V = 0.222(4498) = 998.56^K$

LATERAL FORCE DISTRIBUTION

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{i=n} w_i h_i^k} \quad ; \text{ FOR } T < 0.5\text{sec} \quad k = 1$$

FLOOR	W	h	Wh	V	Vh
R	920^K	39.83′	$36646^{K'}$	334^K	$13303^{K'}$
4	1185^K	30.16′	$35747^{K'}$	326^K	$9834^{K'}$
3	1185^K	20.50′	$24293^{K'}$	221^K	$4531^{K'}$
2	1208^K	10.83′	$13087^{K'}$	119^K	$1289^{K'}$
			109773	1000^K	$28957^{K'}$

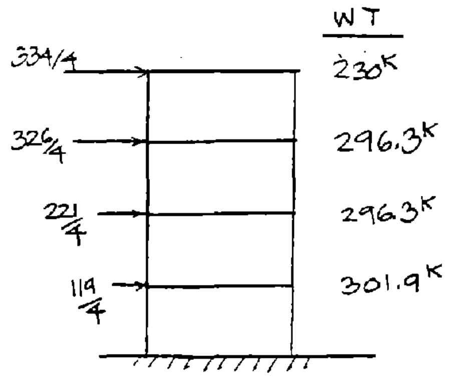
FOR 4 IDENTICAL WALLS, BASE SHEAR/WALL = $1000/4 = 250^K$

MOMENT @ BASE/WALL = 7240 K-ft

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ______
DATE ______
DESIGN ______
SHEET# ______



USE RAILEIGH'S EQUATION TO DETERMINE PERIOD, T (UBC '91 EQ. 31-5)

$$E = 550 f'_{me} = 550(2500) = 1375 \text{ psi}$$

$$I_{gross} = \frac{(7.625)(22 \times 12)^3}{12} = 11.69 \times 10^6 \text{ in}^4$$

AXIAL LOAD = SELF WT. OF WALL (NON-BEARING WALL)

$$P = \frac{77}{1000} \times 39.83 \times 22 = 67.5^K$$

THEN

$$\frac{I_{eff}}{I_{gross}} = \frac{15}{f_y} + \frac{P}{f'_{me} A_g} = \frac{15}{60} + \frac{67.5}{(2.5 \times 7.625 \times 22 \times 12)} = 0.263$$

THUS

$$I_{effective} = 0.263(I_g) = 0.263(11.69 \times 10^6) = 3.08 \times 10^6 \text{ in}^4$$

USING SAP90

FLOOR	f	W	DISP., δ	$W_i \delta_i^2$	$f \delta_i$
R	83.5	230	1.348	417.77	112.54
4	81.5	296.3	0.893	236.48	72.81
3	55.25	296.3	0.475	66.90	26.25
2	29.75	301.9	0.151	6.92	4.52
				728.06	

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB#
DATE
DESIGN
SHEET#

RAYLEIGH'S FORMULA,

$$T = 2\pi \sqrt{(\Sigma w_i \delta_i^2) \div (g \Sigma f_i \delta_i)}$$

$\Rightarrow$ T = 0.587 sec.

THEN

$$C_s = \frac{S_{a(1.0)}\, S}{R\, T_{es}^{\eta}}$$

$$= \frac{(0.58)(1.0)}{(4.5)(0.587)^1} \qquad ; \eta = 1.0 \text{ SINCE } T_{es} < 1.0s$$

$$= 0.2196$$

AND

$$V = \frac{0.2196(4498)}{4} = 246.94^K$$

LATERAL FORCE DISTRIBUTION

AT EACH FLOOR,

$$F_x = C_{vx} V$$

WHERE

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

k VARIES FROM 1 TO 2 B/W T = 0.5s TO T = 2.5 sec

FOR T = 0.587s,

$$k = 1 + \frac{(2.0 - 1.0)}{(2.5 - 0.5)}(0.587 - 0.5) = 1.0435$$

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB#
DATE
DESIGN
SHEET#

	w	h	wh^k	f	SHEAR
R	230	39.83	10753.43	83.85	83.85
4	296.3	30.16	10363.78	80.81	164.65
3	296.3	20.5	6927.03	54.01	218.67
2	301.9	10.83	3626.29	28.28	246.94
			31670.52	246.94	

CHECK T AGAIN, USING RAYLEIGH'S FORMULA,

FROM SAP90 OUTPUT,

	δ	$w\delta_i^2$	$f\delta_i$
R	1.342	414.29	112.53
4	0.889	234.33	71.86
3	0.473	66.21	25.83
2	0.150	6.83	4.25
		721.66	82758.80 / 3684

$T = 0.5867s \simeq 0.587s$ — OK

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ______
DATE ______
DESIGN ______
SHEET# ______

DESIGN OF WALL AT GROUND FLOOR

APPLIED MOMENT, $M_u = (39.83)(83.85) + (30.16)(80.81) + (20.5)(54.01) + (10.83)(28\cdot$

$= 7190.45$ k-ft

AXIAL LOAD, $P_u = 77\text{psf}\ (39.83) \times 22 = 67.5$ KIPS

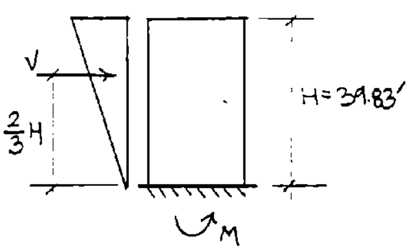
TRY VERTICAL REINFORCEMENT, # 8 @ 16 INCHES O.C.

FROM IMFLEX OUTPUT, (WALL5. OUT)

$M_n = 8729.9$ KIP-FT

$\phi M_n = 0.85(8729.9) = 7420.4$ KIP-FT $> M_u = 7190.5$ KIP-FT — OK

SHEAR DESIGN



$\frac{2}{3} VH = M$

$\Rightarrow V = \frac{3}{2}\frac{M}{H}$

DUCTILE SHEAR FORCE $= \frac{3}{2} \times \frac{8796.2}{39.83} = 331.27$ KIPS

SHEAR CAPACITY

$$v_n = \left[\left(\frac{0.76}{\gamma_d + 0.7} + 0.012\right)(4.04\,\rho_{ve})\sqrt{f_{me}} + 0.01575(\rho_n f_y f_{me})^{1/2}\delta + 0.175\sigma_0\right]\frac{d}{L}$$

$$\rho_n = \frac{331.27}{(22 \times 12 \times 7.625 \times 66)} = 0.0025 > \rho_{min} = 0.0015$$

USE $\rho_n = 0.0025$, $S = 8'' <$ MAX SPACING $= \frac{1}{4}(22 \times 12) = 66''$

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ______
DATE ______
DESIGN ______
SHEET# ______

$A_{sv} = 0.0025(8)(7.625) = 0.153 in^2$

TRY #6 @ 8" HORIZONTAL REINF.

$$r_d = h/l = \frac{39.83 \times 12}{(22\times 12) - 4} = 1.838$$

$$\rho_{ve} = \frac{A_{ve}}{tL} = \frac{0.79}{(7.625)(22\times 12)} = 0.00039 = 0.039\%$$; ONE BAR @ EACH END AS ENCORE REBAR.

$$f_{me} = 2.5 ksi = 2.5(6.89476) = 17.237 Mpa$$

$$\rho_h = \frac{A_h}{S_h t} = \frac{0.44}{8(7.625)} = 0.00721 = 0.721\%$$

$$f_{yh} = 66 ksi = 455.05 Mpa$$

$$\delta = 1.0$$

$$\sigma_0 = \frac{P}{A} = \frac{67.5}{(22)(12)(7.625)} = 0.033 KSI = 0.231 Mpa$$

$$\rho_v = \frac{A_v}{S_v t} = \frac{0.79}{(7.625)(16)} = 0.0065 = 0.65\%$$

THEN

$$v_n = \left[\left(\frac{0.76}{1.838+0.7} + 0.012\right)(4.04\times 0.039)\sqrt{17.237} + 0.01575(0.721\times 455.05\times 17.237)^{1/2}(1.0) + (0.175)(0.231)\right]\frac{260}{264}$$

$$= \left[0.204 + 1.184 + 0.0404\right]\frac{260}{264} = 1.406 Mpa$$

$$= 0.204 KSI$$

Robert Englekirk
Consulting Structural Engineers, Inc.

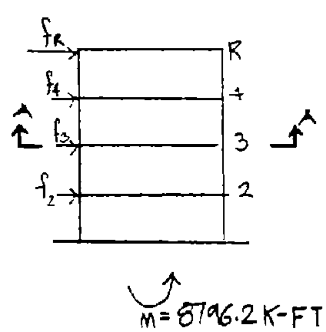
2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

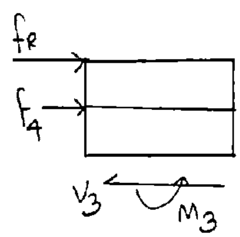
JOB# ______
DATE ______
DESIGN ______
SHEET# ______

$\phi V_n = 0.9(0.204)(22 \times 12)(7.625) = 369.59 \text{ KIPS} > V_u = 331.27 \text{ KIPS}$ OK

USE #8 @ 16" VERTICAL REINF.
#6 @ 8" HORIZONTAL REINF.

DESIGN OF WALL AT 3RD FLOOR





$$\frac{\text{DUCTILE BASE SHEAR}}{\text{ELASTIC DESIGN BASE SHEAR}} = \frac{331.27}{246.94} = 1.342$$

THUS

$f_R = (1.342)83.85 = 112.48^K$

$f_4 = (1.342)80.81 = 108.41^K$

AND

$M_3 = (112.48)(19.33) + (108.4)(9.67) = 3222.56 \text{ K-FT}$

$P_3 = 0.077(22)(19.33) = 32.75^K$

$V_3 = 112.48 + 108.41 = 220.89^K$

$P = 0.077(19.33 \times 22) = 32.75^K$

Robert Englekirk
Consulting Structural Engineers, Inc.

2116 Arlington Avenue
Los Angeles, California 90018-1398
(213) 733-6673 FAX (213) 733-8682

JOB# ______
DATE ______
DESIGN ______
SHEET# ______

TRY #5 @ 16" O.C.

FROM IMFLEX OUTPUT, (WALL6.OUT)

$$M_n = 3839.0 \text{ K-ft} \;;\; \phi M_n = 0.85(3839.0) = 3263.2 \text{ K-FT} > 3222.56$$

— OK

SHEAR

TRY #5 @ 8"

$$\rho_h = \frac{A_{vh}}{S_n t} = \frac{0.31}{(8)(7.625)} = 0.327\,\%$$

$$\sigma_o = \frac{P}{A} = \frac{32.75}{(22)(7.625)(12)} = 0.162\,\text{ksi} = 0.112\,\text{Mpa}$$

THEN

$$v_u = \left[0.204 + 0.01575(0.327 \times 455.05 \times 17.237)^{1/2}(1.0) + (0.175)(0.112)\right]\frac{260}{264}$$

$$= \left[0.204 + 0.795 + 0.0196\right] = 1.019\,\text{Mpa}$$

$$= 0.148\,\text{ksi}$$

$$\phi V_n = (0.9)(0.148)(22 \times 12)(7.625) = 268.13^{K} > V_u = 220.89^{K} \quad \text{OK}$$

USE #5 @ 16" VERTICAL REINF.
#4 @ 8" HORIZONTAL REINF