



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

REPORT NO. 2.1-8



PB93-212959

**SEISMIC PERFORMANCE STUDY  
DPC GYMNASIUM  
ELASTIC TIME HISTORY ANALYSIS  
USING SAP90**

by

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**FEBRUARY 1992**

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**NATIONAL SCIENCE FOUNDATION**

**GRANT NO. BCS-8722869**

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**EKEH**

**Ewing/Kariotis/Englekirk & Hart**





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## **TABLE OF CONTENTS**

<b><u>CHAPTER</u></b>	<b><u>TITLE</u></b>	<b><u>PAGE</u></b>
1	Executive Summary	1-1
2	Building Description	2-1
3	The SAP90 Computer Model	3-1
	3.1 General	3-1
	3.2 Modeling the Shear Walls	3-1
	3.3 Modeling the Diaphragm	3-1
	3.4 Modeling Inelastic Behavior	3-5
	3.5 Iterative Process for Response Computation	3-9
4	Earthquake Response of the Building	4-1
	4.1 General	4-1
	4.2 Building Response Quantities	4-1
	4.3 Results	4-6
5	Conclusions	5-1
6	References	6-1
<b><u>APPENDIX</u></b>	<b><u>TITLE</u></b>	<b><u>PAGE</u></b>
A	Earthquake Ground Motions for Analysis	A-1
B	Interpretation of Diaphragm Stiffness Data	B-1



## **CHAPTER 1**

### **EXECUTIVE SUMMARY**

→ The SAP90 computer analysis of the DPC gymnasium was conducted as part of the ongoing TCCMAR program. Its primary purpose was to demonstrate the feasibility of using an elastic analysis with SAP90 to quantify the inelastic behavior of masonry structures. This research is divided into two main parts. The first part deals with the development of the computer model including the modeling of the flexible diaphragm. It also includes a quantification of the parametric study that is needed to compute the cracked behavior of masonry walls. The second part of the research is a study of the seismic performance of the building to different earthquake records. ←

The DPC gymnasium is a rectangular masonry shear wall structure with a plywood roof diaphragm. It is a single story structure and it has been designed according to the TMS draft LSDS criteria [1]. The building was modelled using the SAP90 elastic analysis computer program. The walls were modeled with elastic shell elements having both in-plane and out-of-plane stiffness. The small openings in the wall were not modelled for ease of modeling and because they will have no significant impact on building performance. Wall stiffnesses were varied to account for the response amplitude dependence of the moment of inertia. The effective moment of inertia was varied by changing the modulus of elasticity. The flexible roof diaphragm was modelled as a truss structure. Flexible truss diagonals were used to represent the shear deformation behavior of the diaphragm. The results of previous tests on diaphragm stiffnesses by ABK were used to determine the numerical values for the truss member properties [2].

To incorporate the inelastic behavior of the masonry walls and diaphragm panels, an iterative process was developed. The walls were subdivided into two zones of different stiffness. An initial estimate was made for the wall stiffness in each zone and a time history analysis performed. The resulting maximum out of plane bending moment along a reference wall strip was compared to the expected moment based on the assumed wall stiffness using the ACI formula for the effective moment of inertia. The stiffness values were then adjusted and another run performed. This was repeated until the resulting moment compared well with the expected moment for the assumed wall stiffness. It was found that three or four iterations were required to obtain convergence.

The building was then studied to evaluate its seismic performance. Several quantities were recorded - a) the maximum wall deformation at the diaphragm level, b) the maximum relative diaphragm deformation and c) the maximum mid height wall deformation. These response quantities were selected because they were studied by the other members of the TCCMAR Task 2 team using the LPM and FEM inelastic analysis models. A comparison of all results will be presented as a part of the next phase of the TCCMAR project.

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 DPC gymnasium is a one story rectangular box shaped structure. It is 128 feet long, 64 feet wide and 24 feet high (disregarding the roof truss). There are a total of two openings along adjacent walls that were not included in this analysis. The building is shown in Figures 2.1 and 2.2.

All walls are constructed using eight inch nominal thickness reinforced concrete masonry. All units were grouted solid. The roof diaphragm was studied for two cases. One was a half inch thick nailed plywood diaphragm and the other was a 20 gage metal deck. The floor at grade level is a concrete slab connected by dowels to the walls. No rotational restraint is assumed at the wall base.

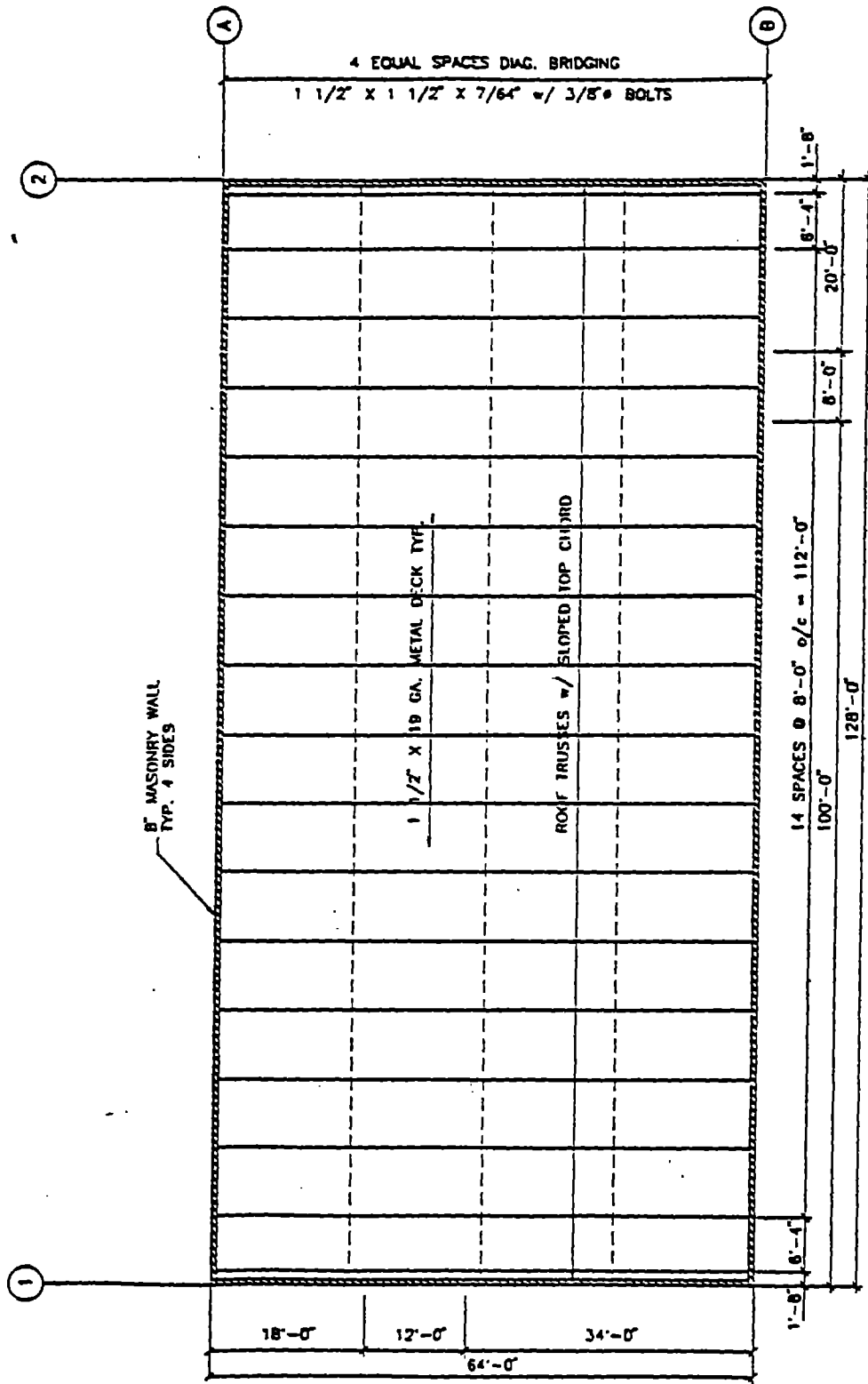
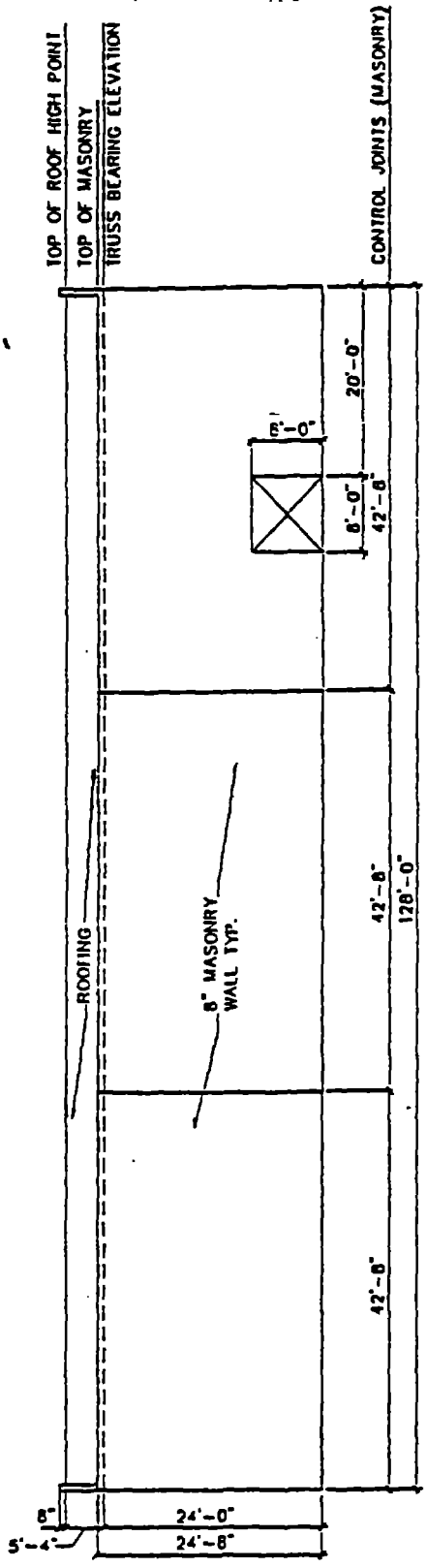


FIGURE 2.1 - DPC GYMNASIUM ROOF FRAMING PLAN



SOUTH ELEVATION (NORTH ELEVATION OPP. HAND)

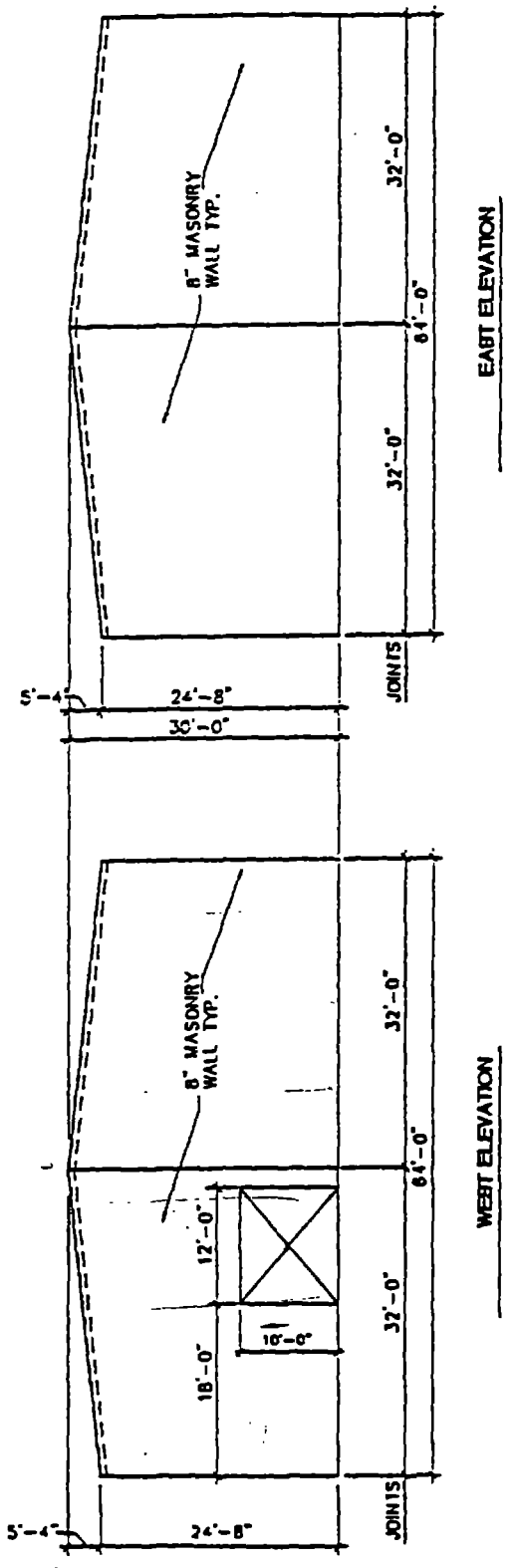


FIGURE 2.2 - DPC GYMNASIUM ELEVATIONS



## **CHAPTER 3**

### **THE SAP90 COMPUTER MODEL**

#### **3.1 General**

The SAP90 computer program was used to model the building [3]. The building model is a three dimensional model. The research had two objectives. The first research objective was to demonstrate how one could analyze a masonry structure with flexible diaphragms using a 3-D elastic model. In such a model we want to show how to model the inelastic behavior of the walls. Our second objective was to study the performance of the building to a specified set of earthquake time histories and compare them to our inelastic computer results with LPM and FEM computer programs for the same building. The discretized 3-D model of the building is shown in Figure 3.1.

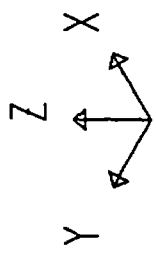
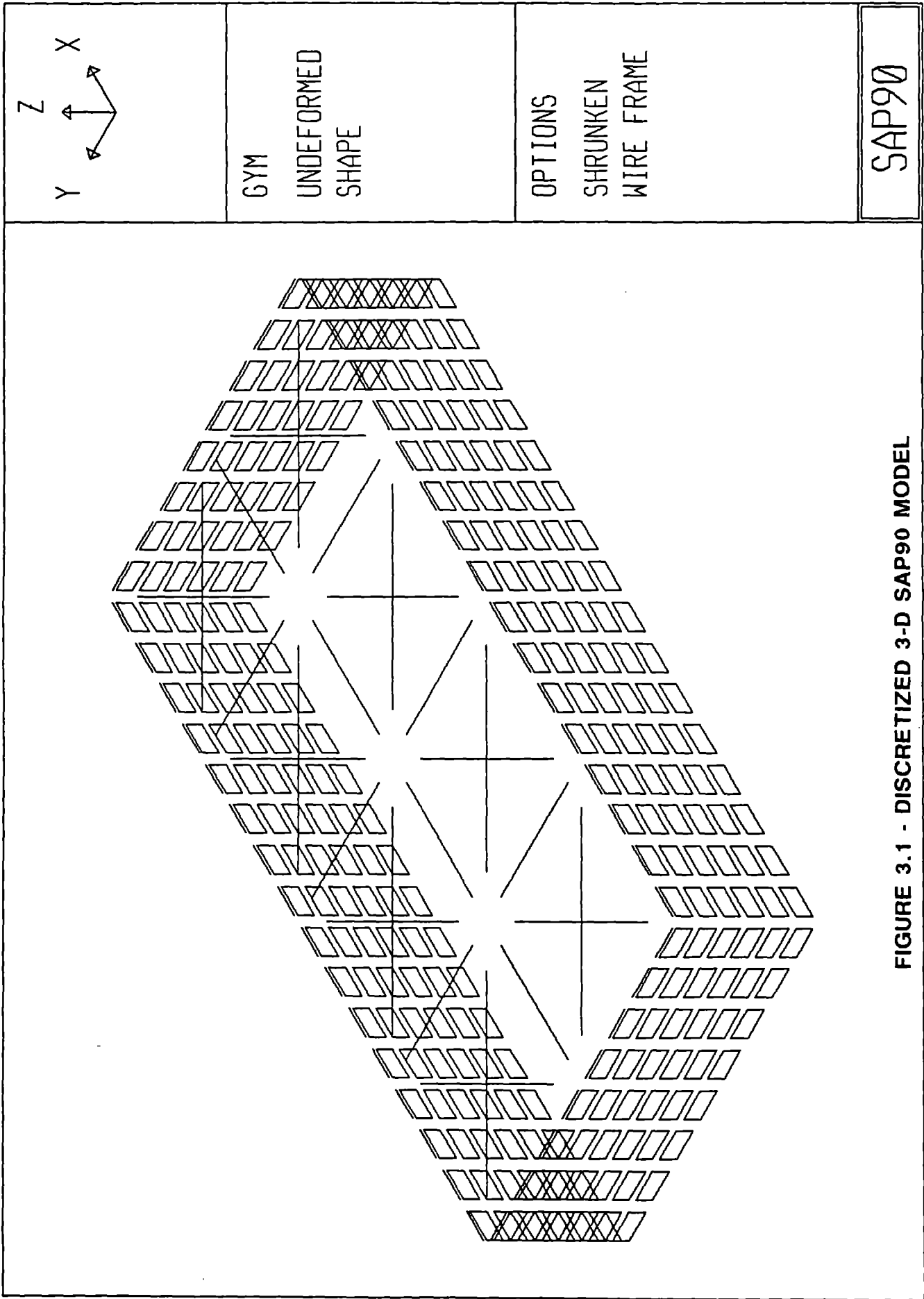
#### **3.2 Modeling the Shear Walls**

The shear walls are modeled as 7.625 inch thick shell elements. Since the existing openings are disregarded, a uniform mesh, both vertically and horizontally is used for all the walls. Each element is eight feet long by four feet high and thus has an aspect ratio of 1:2. There are a total of 288 elements in the model. All of the element stiffnesses are specified through the modulus of elasticity and the element thickness. Membrane stiffnesses are also considered. From the design data, the initial modulus of elasticity was assumed to be 2500 ksi. The walls were not allowed to move in the Z axis (vertically). Each wall node was also allowed two rotational degrees of freedom - the out of plane bending in two orthogonal directions.

As noted earlier, the floor slab was not allowed to impart any fixity to the walls. The long walls were therefore hinged in the X direction and the short walls in the Y direction. The finite element discretization scheme for the short and long walls are shown in Figures 3.2-A and 3.2-B.

#### **3.3 Modeling the Diaphragm**

The diaphragm for this gymnasium was plywood sheathing. However, in many applications a metal deck would be used and thus a metal deck diaphragm is also modelled in this research. Both of these diaphragms are flexible relative to the walls.



GYM  
UNDEFORMED  
SHAPE

OPTIONS  
SHRUNKEN  
WIRE FRAME

SAP90

FIGURE 3.1 - DISCRETIZED 3-D SAP90 MODEL

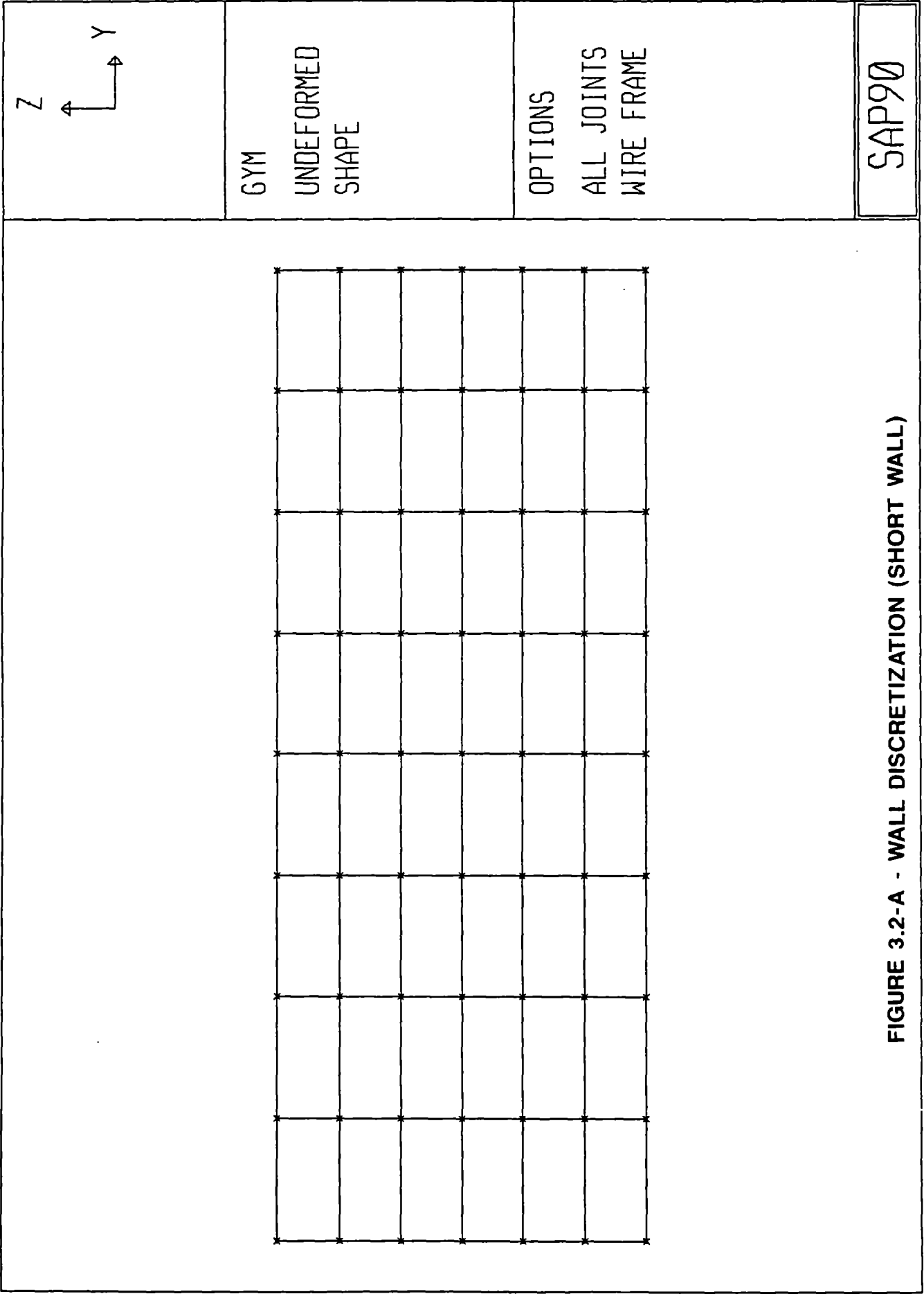
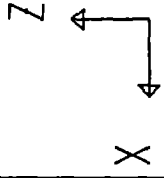


FIGURE 3.2-A - WALL DISCRETIZATION (SHORT WALL)



GYM  
UNDEFORMED  
SHAPE

OPTIONS  
ALL JOINTS  
WIRE FRAME

SAP90

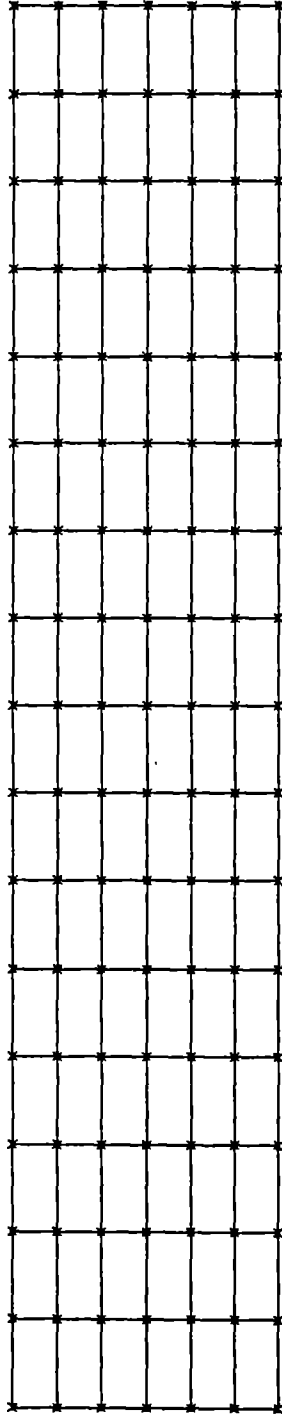


FIGURE 3.2-B - WALL DISCRETIZATION (LONG WALL)



(NS and EW). The 1:2 aspect ratio of the building plan dimensions indicated that the primary deformation of the diaphragm would be in shear, between adjacent panels. Several types of models for flexible roof diaphragms were studied. The truss analogy was selected because it gave a good picture of the actual diaphragm behavior i.e. panels shearing between each other for ground motion occurring in any direction.

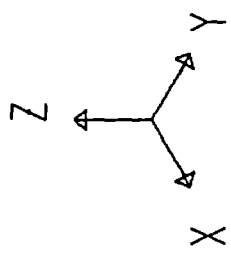
The truss model for the roof diaphragm is shown in Figure 3.3. All non-diagonal members in the model are links with a very large area so as to model them as rigid. All the connecting joints allow only the axial force to be transmitted. There are 4 truss panels in the long direction and 2 in the short direction. Each truss panel is 32 feet by 32 feet. The area of the diagonals was selected such that the truss would have a shear stiffness consistent with the ABK data. The diaphragm must be calibrated to reflect the actual stiffness of the roof diaphragm. For this, a unit area for the diagonals is assumed. The truss model only is loaded at one end with a unit load and the other side is assumed pinned. For this load condition, the deflection/unit load is computed. ABK associates performed stiffness tests on diaphragms and their topical report TR-05 [2] indicates the procedure for interpreting and scaling the stiffnesses for other diaphragm sizes. Using their data, the area of the diagonals was computed.

Appendix B gives a concise description of the actual diaphragm stiffness properties used for this report as well as the scaling rules from the ABK report. It should be noted that if the panel aspect ratio is kept at 1:1, and the diagonal member areas are scaled in either direction, the stiffness in the other direction is automatically scaled - as per the ABK scaling rules.

### **3.4 Modeling Inelastic Behavior**

Masonry structures will crack and experience inelastic behavior when subjected to earthquakes. Since SAP90 is an elastic analysis program, an approximate technique for quantifying inelastic response was used to model the DPC gymnasium. The diaphragm panels also respond in the inelastic range. Since the diaphragm panels closest to the wall shear the most, their stiffnesses were set at 60 percent of the elastic stiffness and the panels in the center were modeled with stiffnesses at 80 percent of the elastic stiffness. These can be changed if the nonlinear behavior of the diaphragms can be quantified, i.e. , the assumed stiffness must match the load-deflection curve for the diaphragm at the level of deformation computed in the run.





GYM  
UNDEFORMED  
SHAPE

OPTIONS  
ELEMENT PINS  
WIRE FRAME

SAP90

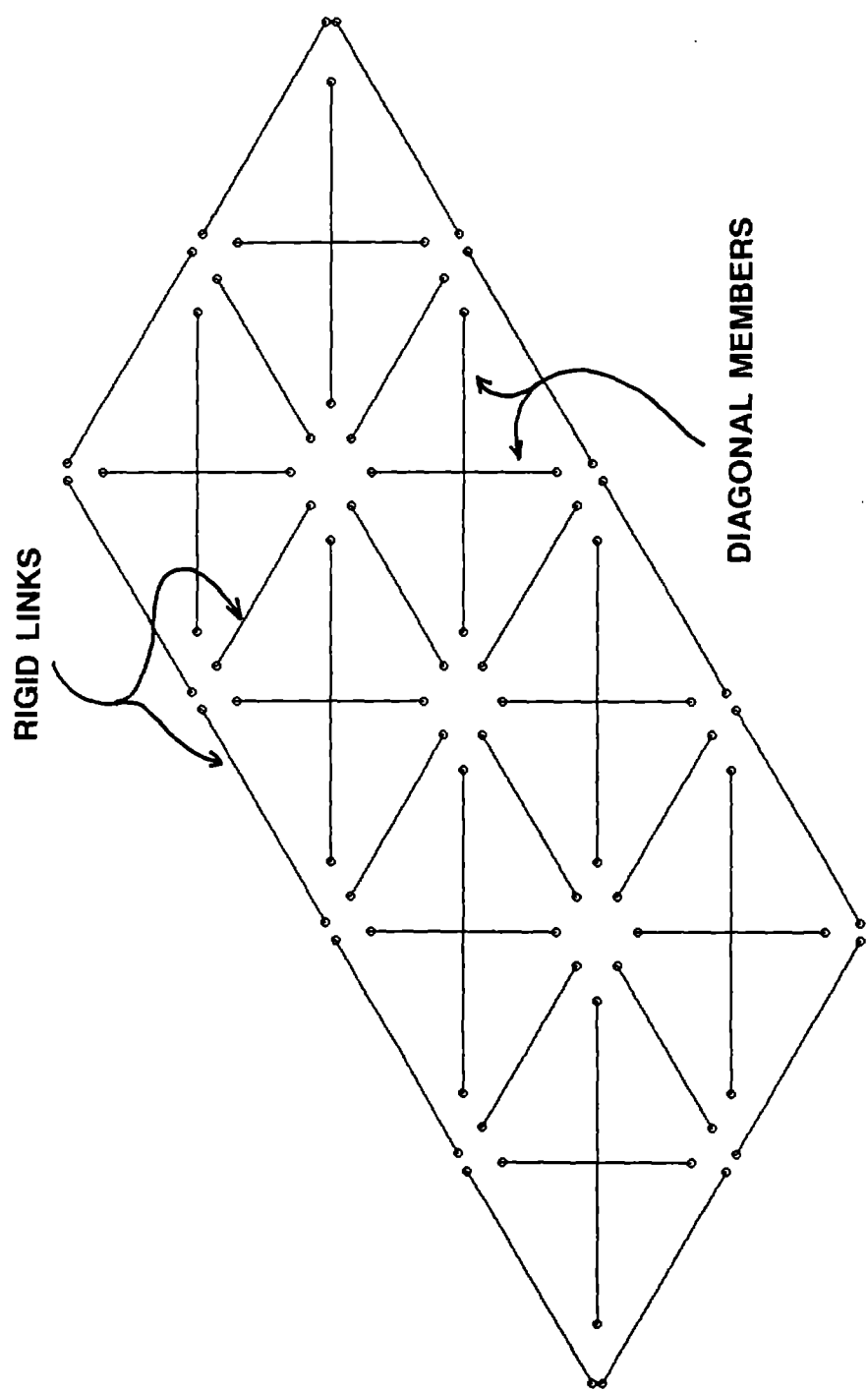


FIGURE 3.3 - TRUSS MODEL FOR THE FLEXIBLE DIAPHRAGM



The walls were subdivided into two zones - an inner and an outer. These two zones were assumed to have two different effective stiffnesses - the inner zone being more cracked due to the out of plane bending. The stiffness zones are shown in Figure 3.4. For example, the outer zone could have a stiffness 60 percent of the initial stiffness and the inner zone 40 percent of the initial stiffness.

Since all four walls had these zones of cracking, the effect of the crack zone in the in-plane walls on the building response (primarily derived from the out-of-plane displacement behavior) had to be studied. Two types of runs were conducted. First, a single wall with the double zoned stiffnesses (as shown in Figure 3.4) was subjected to a horizontal load at the wall top to study its in-plane deflections. As the wall stiffnesses was decreased, the wall deflection at the top also increased. However, it was found that the walls were so stiff that upon reducing the stiffnesses by a factor of 5, the increased roof displacement that would contribute to the out-of-plane response would be negligible given the magnitude of the peak out-of-plane responses. A second type of load test was conducted. This time, the entire three dimensional model was used. The effective stiffness in the in-plane walls was assumed to be in two parallel zones, the lower zone with only 20 percent of the initial stiffness. Actual time-history runs were conducted. It was found that the increase in building period was negligible and so was the change in the out-of-plane building displacement response. It should be noted that for a study of the in-plane wall response or for stress/moment quantities in any of the walls, such conclusions may not be valid.

A starting estimate for the stiffnesses in the two wall zones was 100 percent and 80 percent of the initial stiffness. The stiffness reduction for the walls was performed by changing the modulus of elasticity. A wall strip, running horizontally along the wall at the wall centerline was used as a reference. Since the wall is divided into eight strips horizontally, the outer two strips on either side are of greater stiffness than the center four strips.

Using the assumption that this strip behaves like a beam, the ACI formula was used to calculate effective moment of inertia. The formula is

$$I_{\text{eff}} = (M_{\text{cr}}/M_{\text{a}})^3 I_{\text{g}} + [1 - (M_{\text{cr}}/M_{\text{a}})^3] I_{\text{cr}} \quad (3-1)$$

where

$I_{\text{eff}}$  = effective moment of inertia

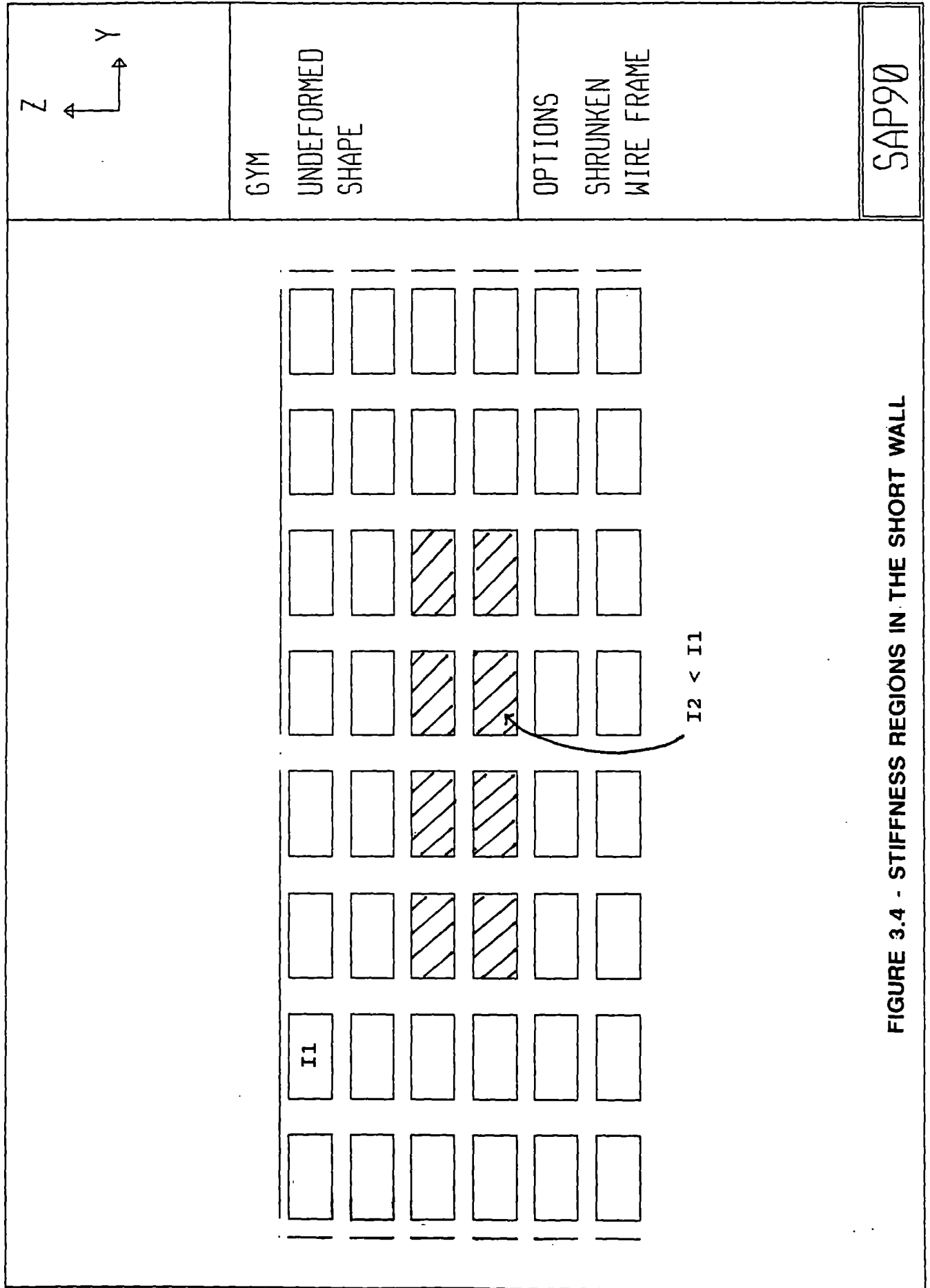


FIGURE 3.4 - STIFFNESS REGIONS IN THE SHORT WALL

- $I_g$  = gross moment of inertia
- $I_{cr}$  = cracked moment of inertia
- $M_{cr}$  = cracking moment
- $M_a$  = applied moment

The gross moment of inertia, the cracked moment of inertia and the cracking moment are constants for the section and depend on the material, geometry and the reinforcement in the walls. The applied moment at the section is obtained from the SAP90 time history analysis and it is the maximum bending moment along the reference strip of the wall. The iterative process for computing the elastic approximation to the inelastic response is described in the next section. It should also be noted that P-Delta effects are not included in this analysis since SAP90 lacks this capability.

### **3.5 Iterative Process for Response Computation**

- Step 1. Assume a set of values for the effective moments of inertia of the wall in the two zones shown in Figure 3.4. It should be noted that the SAP90 model can be set with as many zones as the engineer believes is necessary.
- Step 2. Assume a reference strip for computing convergence. In this case, the strip chosen was at the wall centerline. It was assumed that this vertical strip behaved like a beam and hence the cracking was directly a result of the out-of-plane bending moments on the section. The ACI formula relating the applied moment to the effective moment of inertia was used to determine the effective moment of inertia.
- Step 3. Run the time-history analysis for the model. Find the maximum bending moments along the reference strip for the two zones. Compute the effective moment of inertia using the ACI formula.
- Step 4. If the computed effective moments of inertia in the two zones do not match the assumed moments of inertia, change the values for the moments of inertia of the two wall zones in the model and re-run the time history analysis. Repeat Steps 2 and 3 until the computed and assumed effective moments of inertia converge.

**Step 5.** This is the "elastic approximation" to the inelastic response. For this converged set of values, compute the response quantities that are required.



## **CHAPTER 4**

### **EARTHQUAKE RESPONSE OF THE GYMNASIUM**

#### **4.1 General**

The time history runs were performed on the SAP90 model for four different time histories - records 1, 2, 6, and 9. The earthquake records used in this study are described in Appendix A. For each time history, two runs were performed for each record - one for each direction. Both positive and negative maximum responses were calculated. All of the time histories were scaled for a seismic zone with a ZPA of 0.4g.

#### **4.2 Building Response Quantities**

For this building, three different displacement response quantities, all of them connected with the out-of-plane wall behavior were computed. They are:

1. **The Maximum Relative Panel Displacement:** This is the maximum relative displacement of any diaphragm panel and is found by computing the maximum displacement of the diaphragm panel nearest to the walls parallel to the ground motion.
2. **The Maximum Roof Displacement:** This is found by computing the displacement response of the diaphragm along its centerline. For the east-west direction, this is the same as the maximum relative panel displacement since there are only two panels spanning the wall.
3. **The Maximum Mid-Height Wall Deflection:** This is computed as the difference between the deflection of the wall at mid-height and half the deflection at the top of the wall - both at the wall centerline. This removes the displacement term associated with the diaphragm displacement and estimates the relative curvature of the wall (since the displacement contribution of the diaphragm displacement to the wall at mid height is half the diaphragm displacement at the top of the wall).

These response quantities are shown in Figure 4.1. An eigenvalue analysis was performed as a preliminary to the time history analysis. Figures 4.2 and 4.3 show the first modes for the north-south and east-west directions for the gymnasium. Table 4.1 shows a comparison of the periods of vibration of the gymnasium for both the wood and steel

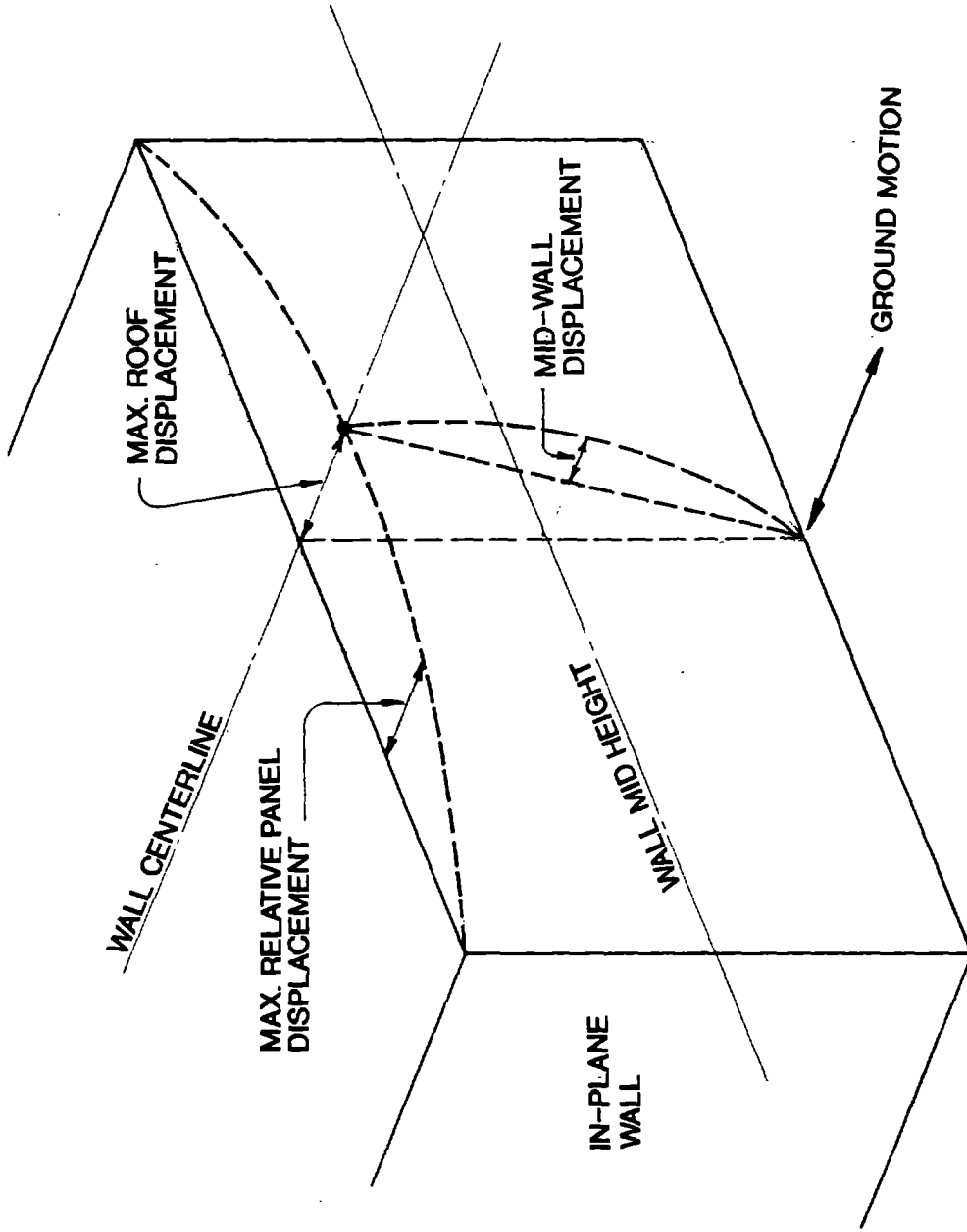
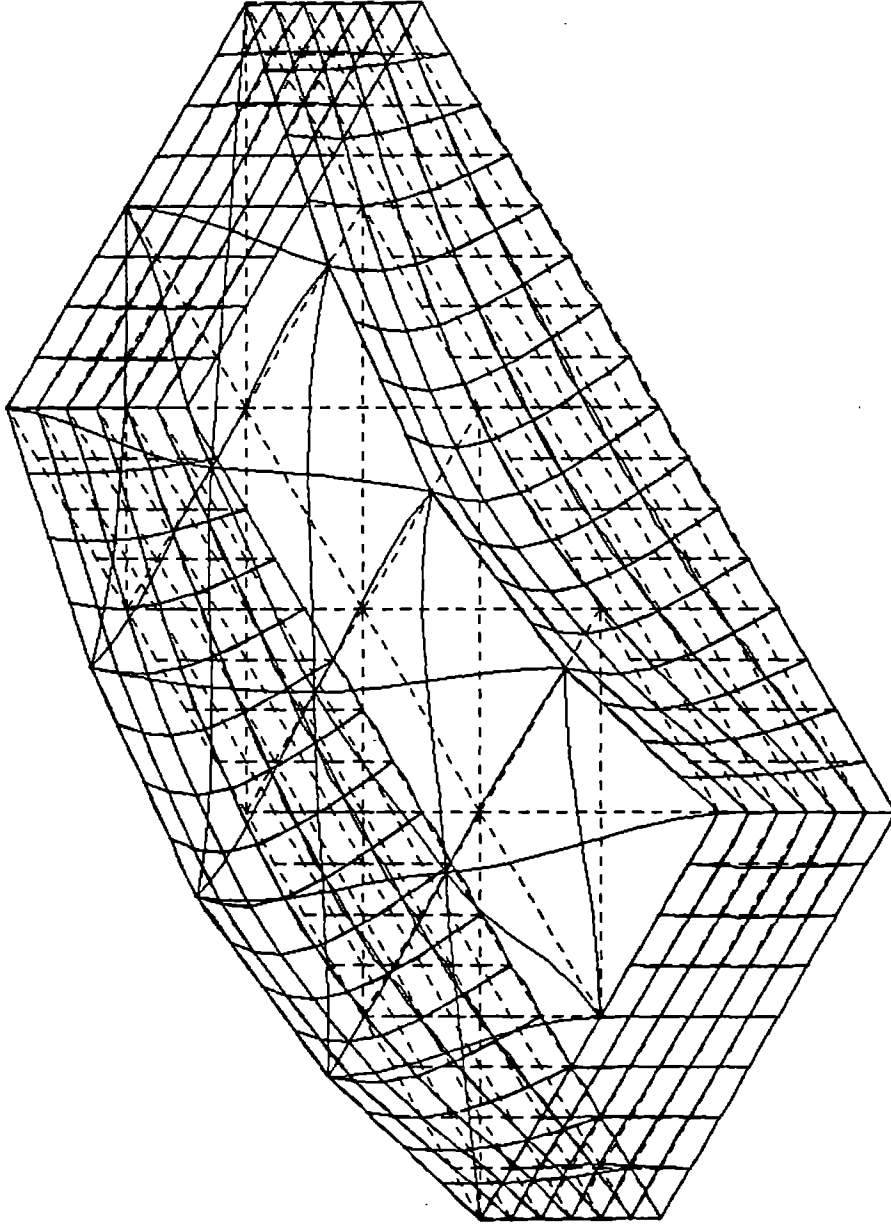
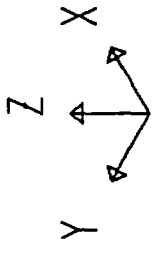


FIGURE 4.1 - OUT OF PLANE RESPONSE QUANTITIES



GYM  
 MODE  
 SHAPE  
 MODE 1

MINIMA  
 X-0.5193E-01  
 Y 0.0000E+00  
 Z 0.0000E+00  
 MAXIMA  
 X 0.5193E-01  
 Y 0.1543E+01  
 Z 0.0000E+00

SAP90

FIGURE 4.2 - FIRST MODE N-S DIRECTION

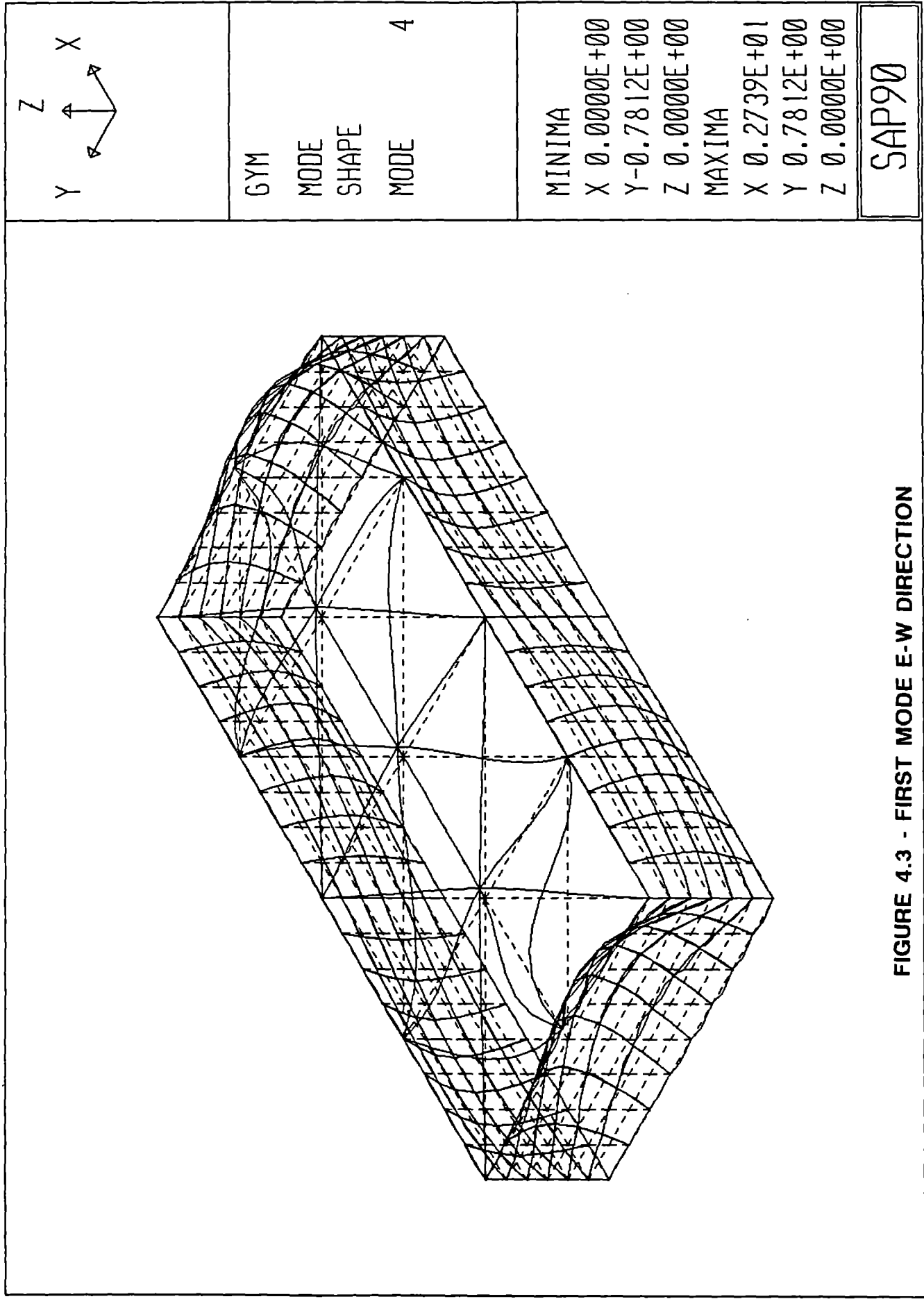


FIGURE 4.3 - FIRST MODE E-W DIRECTION

	GYM MODE SHAPE MODE 4	MINIMA X 0.0000E+00 Y -0.7812E+00 Z 0.0000E+00 MAXIMA X 0.2739E+01 Y 0.7812E+00 Z 0.0000E+00	SAP90
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**TABLE 4.1**

**COMPARISON OF NATURAL PERIODS OF VIBRATION**

Direction	Mode	Plywood Diaphragm	Metal Deck Diaphragm
		Period (s)	Period (s)
EW	1	0.21	0.13
	2	0.20	0.11
	3	0.07	0.07
NS	1	0.50	0.27
	2	0.25	0.14
	3	0.17	0.09

deck. The final iterated wall stiffnesses did not vary much for the four earthquakes for a given direction (EW or NS), all being between 15 and 30 percent of the gross stiffness and hence, the natural period of the structure also did not change for each earthquake. Therefore, the numbers in Table 4.1 represent the overall structural behavior.

### **4.3 Results**

The results of the analyses are shown in Tables 4.2 to 4.5 Figures 4.4-4.9 also show the time history of selected responses for the two types of diaphragms studied.

**TABLE 4.2 - DISPLACEMENT RESPONSE RESULTS**

Plywood Deck, NS Motion

Motion No.	Building Centerline		Panel Nearest NS Wall	Drift (%)	Base Shear Coeff.
	Roof Displ (in)	Mid-Wall Displ (in)	Roof Displ (in)		
1	-2.85	-2.15	-1.10	0.99	0.63
	3.25	2.47	1.26	1.13	
2	-3.37	-2.62	-1.77	1.17	0.71
	3.55	2.61	2.10	1.23	
6	-2.62	2.00	-2.20	0.91	0.53
	3.02	2.27	2.48	1.05	
9	-1.94	-1.48	-0.85	0.67	0.40
	1.39	1.02	0.60	0.48	
<b>Average</b>	<b>2.75</b>	<b>2.08</b>	<b>1.55</b>	<b>0.95</b>	<b>0.57</b>

**Notes:**

1. Refer to Figure 4.1 for explanation of response quantities
2. Drift is the Roof Displacement/Wall Height at building centerline
3. Base Shear Coefficient is Base Shear/Building Weight

## TABLE 4.3 - DISPLACEMENT RESPONSE RESULTS

Plywood Deck, EW Motion

Motion No.	Building Centerline		Panel Nearest NS Wall	Drift (%)	Base Shear Coeff.
	Roof Displ (in)	Mid-Wall Displ (in)	Roof Displ (in)		
1	-0.46	-1.29	-	0.16	0.42
	0.39	1.21	-	0.14	
2	-0.40	-1.48	-	0.14	0.38
	0.48	1.40	-	0.17	
6	-0.65	-2.14	-	0.23	0.39
	0.51	1.96	-	0.18	
9	-0.57	-1.92	-	0.20	0.52
	0.36	0.98	-	0.13	
Average	0.48	1.55	-	0.17	0.43

**Notes:**

1. Refer to Figure 4.1 for explanation of response quantities
2. Drift is the Roof Displacement/Wall Height at building centerline
3. Base Shear Coefficient is Base Shear/Building Weight

2.1.1



**TABLE 4.4 – DISPLACEMENT RESPONSE RESULTS**

Steel Deck, NS Motion

Motion No.	Building Centerline		Panel Nearest NS Wall	Drift (%)	Base Shear Coeff.
	Roof Displ (in)	Mid-Wall Displ (in)	Roof Displ (in)		
1	-0.82	-1.65	-0.59	0.28	0.46
	0.69	1.41	0.50	0.24	
2	-0.63	-1.96	-0.38	0.22	0.59
	0.82	1.77	0.50	0.28	
6	-0.79	-2.00	-0.56	0.27	0.63
	0.81	1.85	0.59	0.28	
9	-0.78	-1.44	-0.57	0.27	0.54
	0.52	1.25	0.35	0.18	
Average	0.73	1.67	0.51	0.25	0.55

**Notes:**

1. Refer to Figure 4.1 for explanation of response quantities
2. Drift is the Roof Displacement/Wall Height at building centerline
3. Base Shear Coefficient is Base Shear/Building Weight

**TABLE 4.5 – DISPLACEMENT RESPONSE RESULTS**

Steel Deck, EW Motion

Motion No.	Building Centerline		Panel Nearest NS Wall	Drift (%)	Base Shear Coeff.
	Roof Displ (in)	Mid-Wall Displ (in)	Roof Displ (in)		
1	-0.12	-1.26	-	0.04	0.39
	0.20	1.28	-	0.07	
2	-0.10	-1.40	-	0.03	0.38
	0.13	1.52	-	0.05	
6	-0.13	-1.84	-	0.05	0.38
	0.16	1.62	-	0.06	
9	-0.20	-1.83	-	0.07	0.60
	0.10	1.00	-	0.03	
Average	0.14	1.47	-	0.05	0.44

**Notes:**

1. Refer to Figure 4.1 for explanation of response quantities
2. Drift is the Roof Displacement/Wall Height at building centerline
3. Base Shear Coefficient is Base Shear/Building Weight

FIGURE 4.4 — Roof Displacement  
Plywood Deck, NS, Motion 2

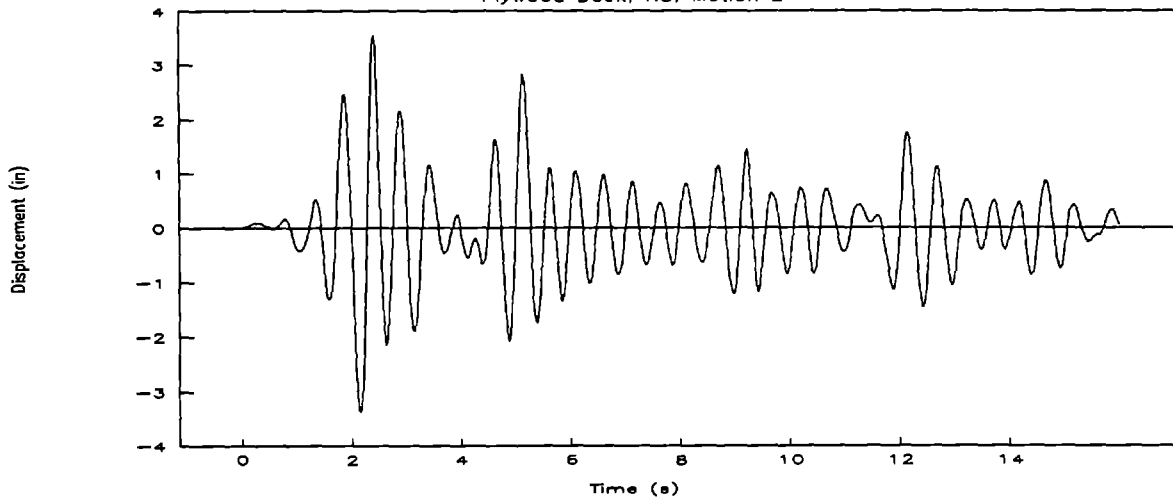


FIGURE 4.5 — Mid Ht Displacement  
Plywood Deck, NS, Motion 2

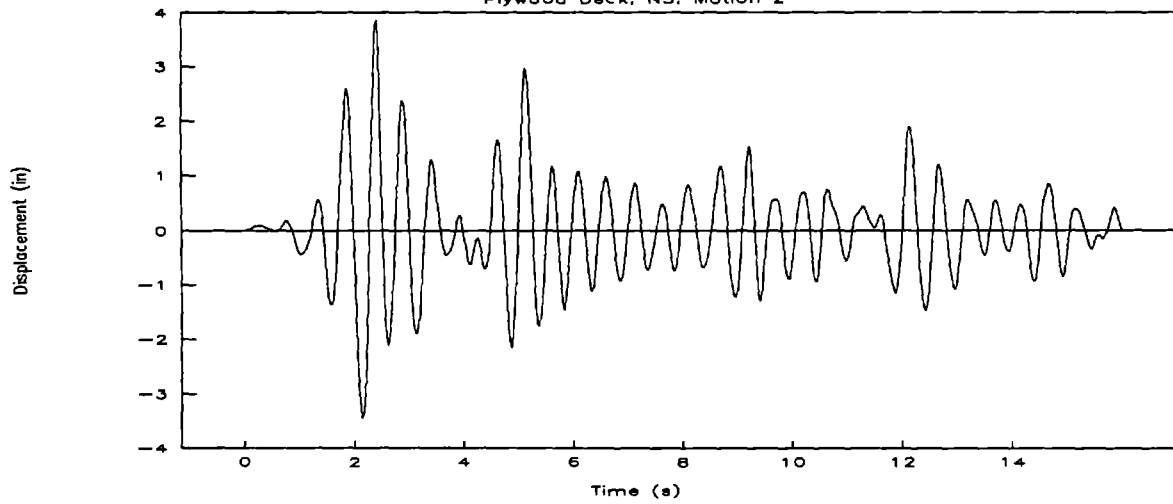


FIGURE 4.6 — Relative Panel Motion  
Plywood Deck, NS, Motion 2

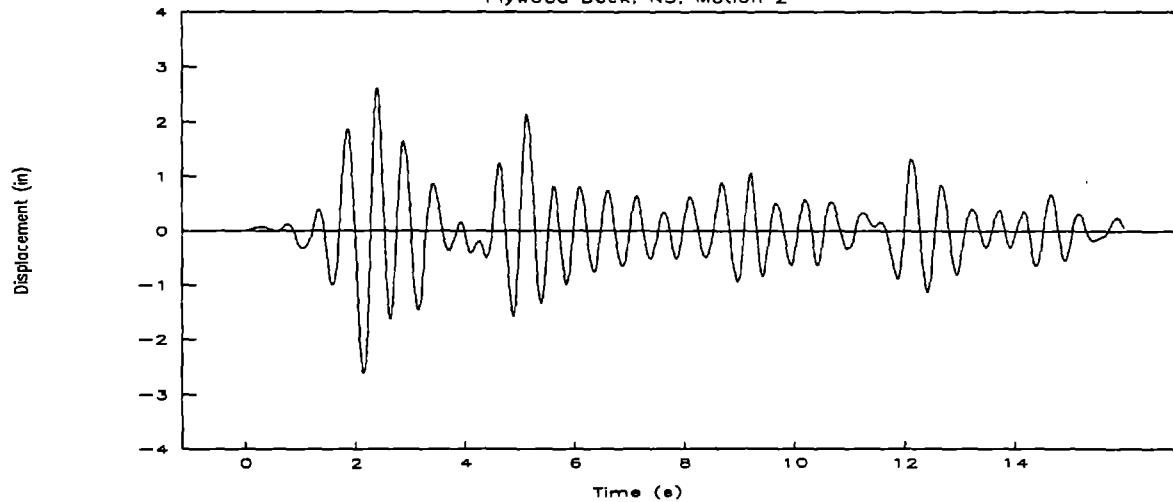


FIGURE 4.7 — Roof Displacement  
Metal Deck, NS. Motion 2

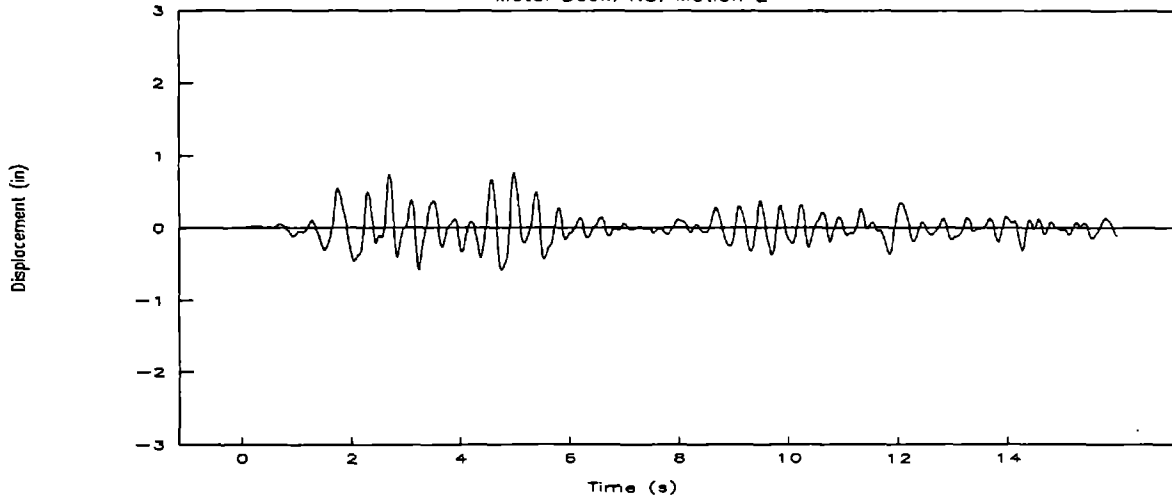


FIGURE 4.8 — Mid Ht Displacement  
Metal Deck, NS. Motion 2

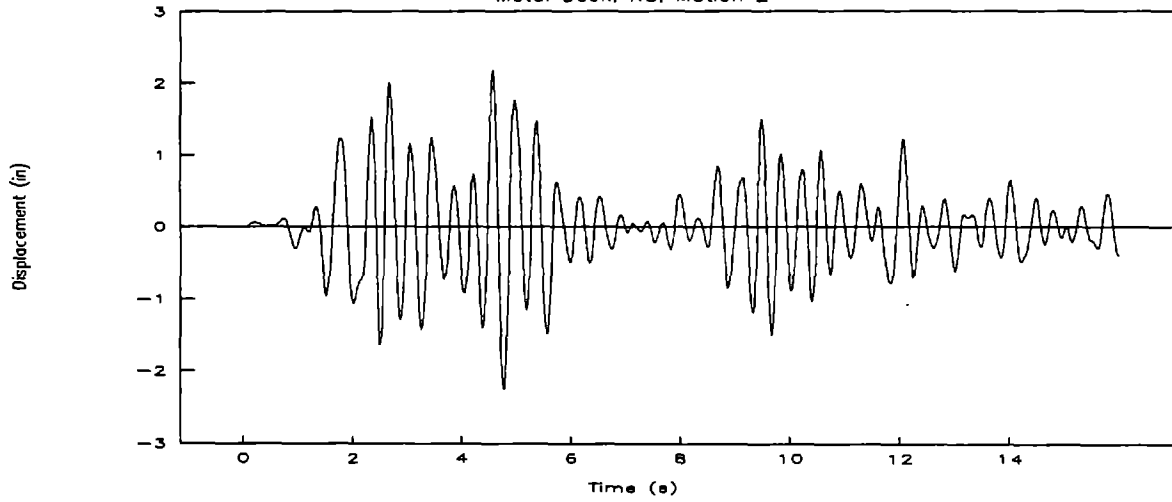
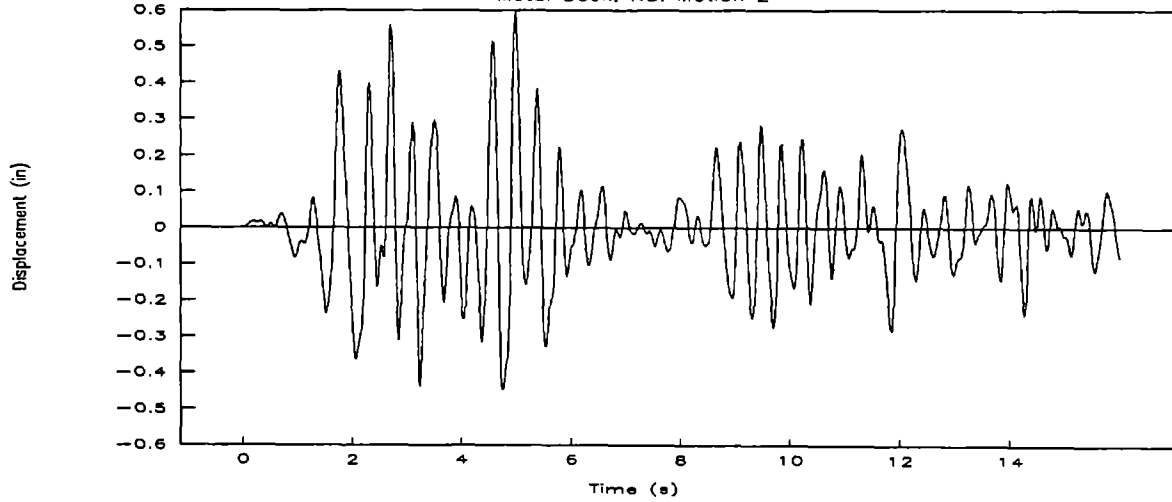


FIGURE 4.9 — Relative Panel Motion  
Metal Deck, NS. Motion 2



## **CHAPTER 5 CONCLUSIONS**

Results from the analysis can be divided into those quantifying the methodology and those representing building performance. The former includes the iterative technique for a "pseudo" nonlinear response and the diaphragm modeling technique. The latter is comprised of the displacement responses discussed earlier as well as the drift and the base shear.

The iterative technique developed here is dependent to a great extent on the regions of the wall that are checked for deformation/force levels. The more refined the mesh, the greater the complexity of the analysis and larger the number of runs to be made. It may be helpful to conduct a simple sensitivity study to estimate the pattern or numbers of stiffness zones that would be adequate. However, some amount of engineering judgement will surely help in reducing the size of the task.

The flexible truss model for the diaphragm worked very well in quantifying the overall displaced shape of the roof. It is also very easy to model and scale. However, it is necessary to have a good estimate of the actual stiffness of the diaphragm to be modeled since it will control the sizing of the diagonal members of the truss. It will be of great interest in applying this model to diaphragms that are of different shapes such as L shaped roofs as well as to diaphragms with openings.

The performance of the steel and plywood diaphragms can be seen in the results in Tables 4.2 to 4.5. As expected, the steel diaphragm makes the structure a lot stiffer and greatly reduces the out of plane response. It can also be seen from Table 4.1 that the actual value of the diaphragm stiffness is a significant factor in the computation of the natural period as well as the peak response. It was found from all the four earthquake records that the steel deck diaphragm response went into the inelastic range (by checking the diaphragm shear with its capacity). It may be useful therefore to have the diaphragm type factor into the actual value of the shear used in its design - i.e. the relative stiffness of the diaphragm and the walls is an important factor to consider in the design of both the walls and the diaphragm.

The base shear coefficients that were computed from the computer runs were about the same for both the steel and plywood diaphragms. This may be an area where an actual nonlinear run may show the difference in the base shears more effectively. It should be noted that the force-deflection model for both types of diaphragms were the same i.e. linear.

The drifts in both directions were higher for the plywood deck as compared to the steel deck. The ratio of the averaged drift ratios in the two directions for the two different

types of diaphragms was approximately equal to the ratio of the shear stiffnesses for the two diaphragms.

A comparison of the elastic and inelastic model results will be presented in the next phase of the TCCMAR research project with a discussion of the results and various techniques.

## **CHAPTER 6 REFERENCES**

1. **LSDS, Masonry Limit States Design Standards - Draft, The Masonry Society, February, 1991.**
2. **Interpretation of Diaphragm Tests - ABK Topical Report TR-05, ABK Associates, El Segundo, California, 1981.**
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.**





## **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. 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, February 1992, Kariotis and Associates, South Pasadena, California.

TABLE A  
Earthquake Ground Motions

Earthquake	$\Delta t$ (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

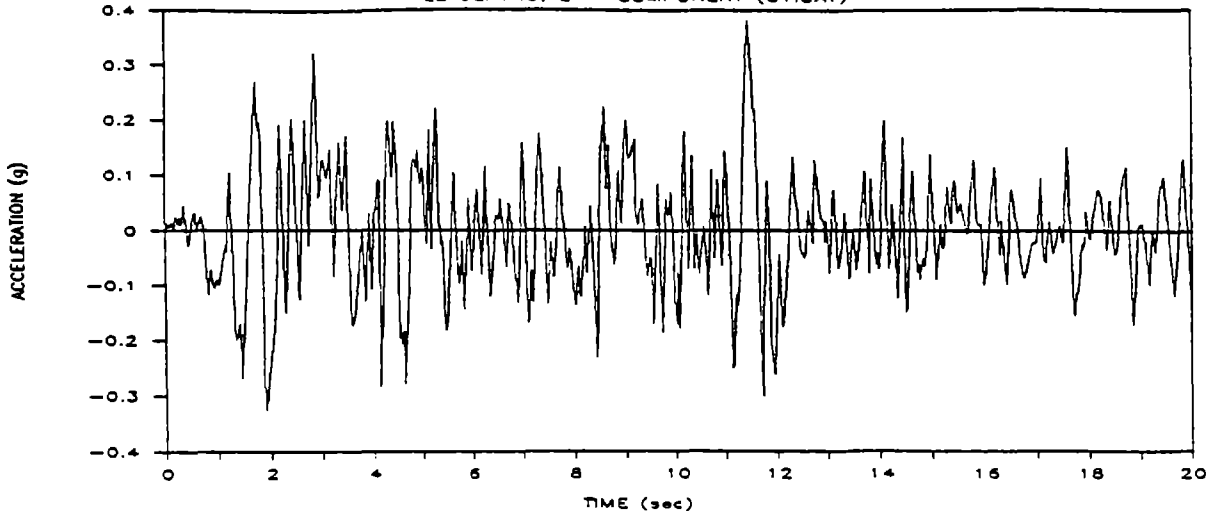
<sup>1</sup>  $C_s$  = Scaling factor for converting acceleration units to in/s/s.

<sup>2</sup>  $C_1$  = Scaling factor for Seismic Zone 2

<sup>3</sup>  $C_2$  = Scaling factor for Seismic Zone 4

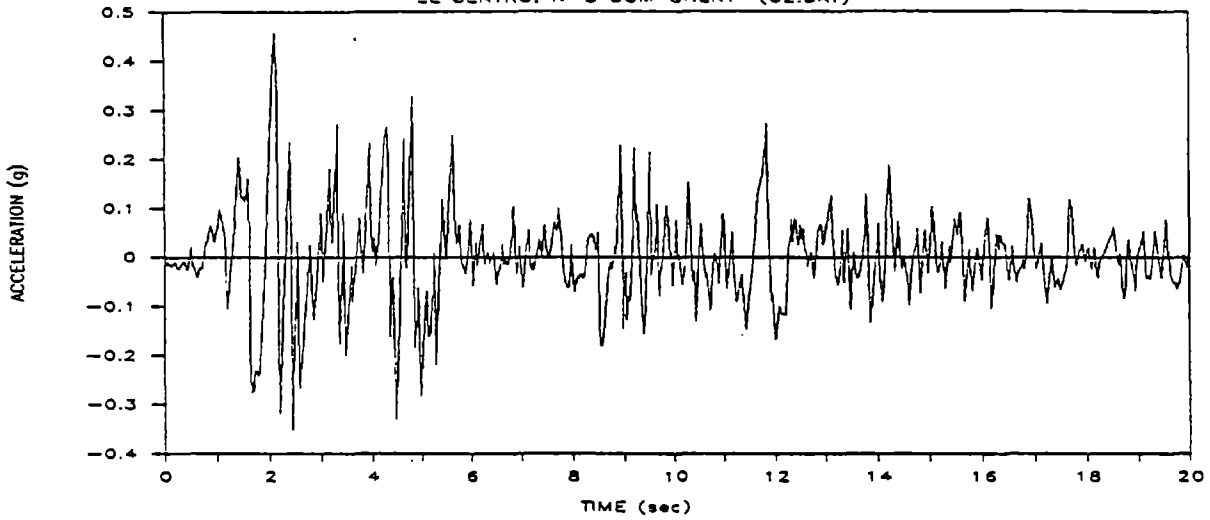
### ACCELERATION TIME HISTORY (ZONE 4)

EL CENTRO, E-W COMPONENT (G1.DAT)



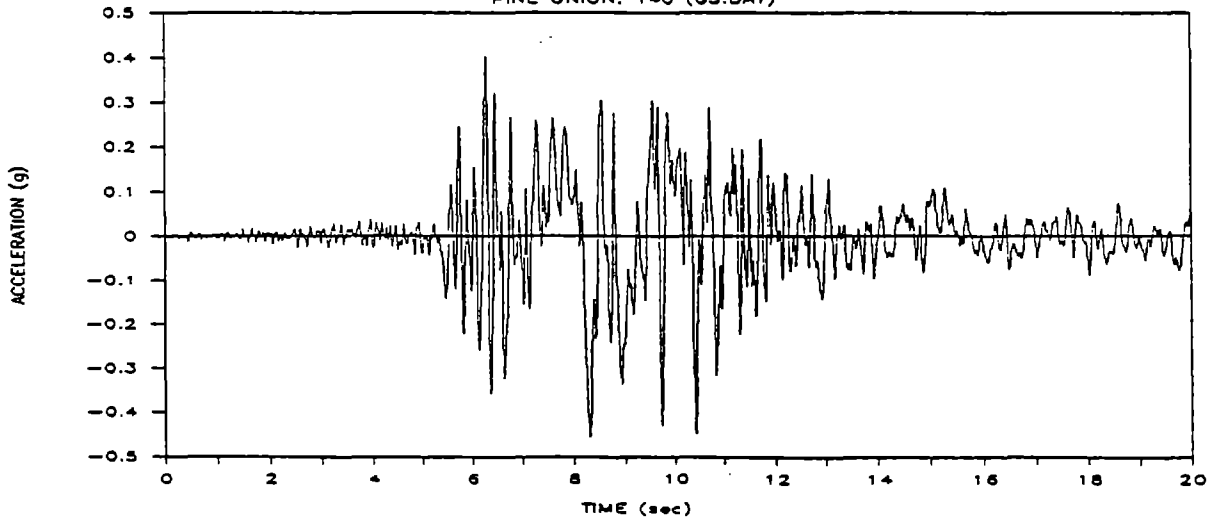
### ACCELERATION TIME HISTORY (ZONE 4)

EL CENTRO, N-S COMPONENT (G2.DAT)



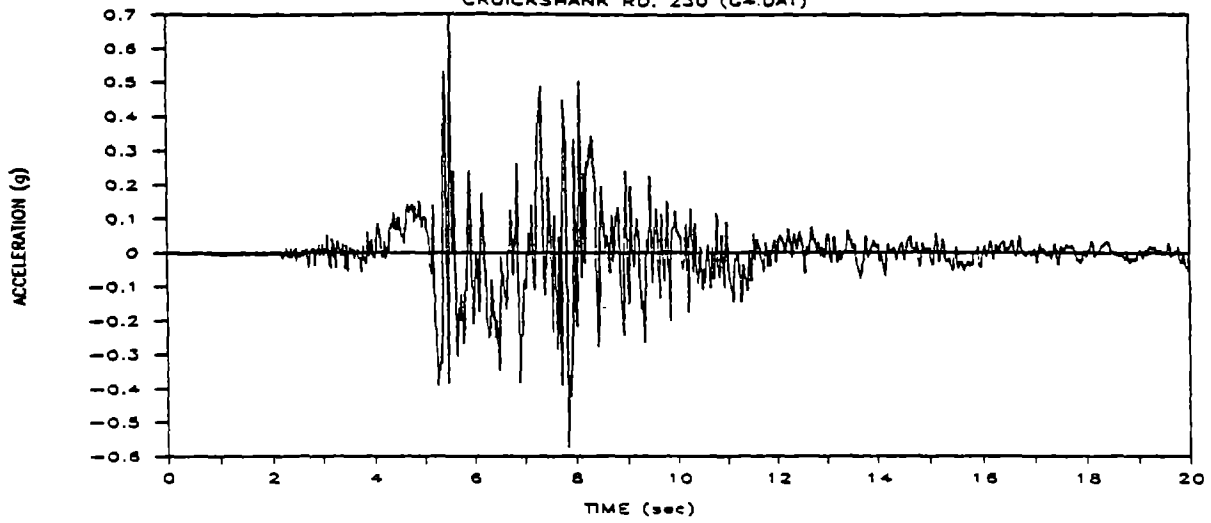
### ACCELERATION TIME HISTORY (ZONE 4)

PINE UNION, 140 (G3.DAT)



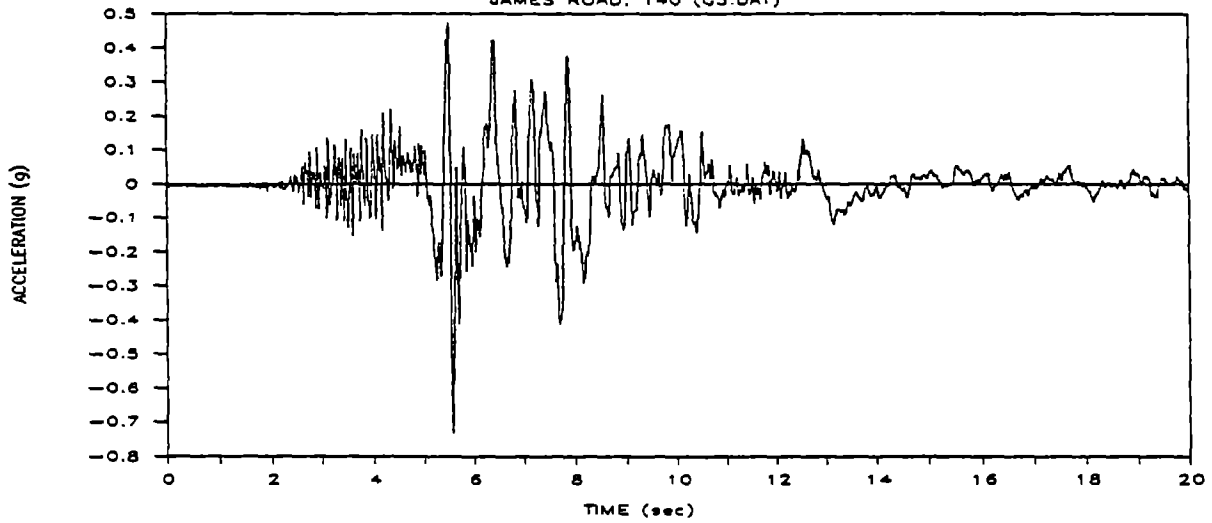
### ACCELERATION TIME HISTORY (ZONE 4)

CRUICKSHANK RD. 230 (G4.DAT)



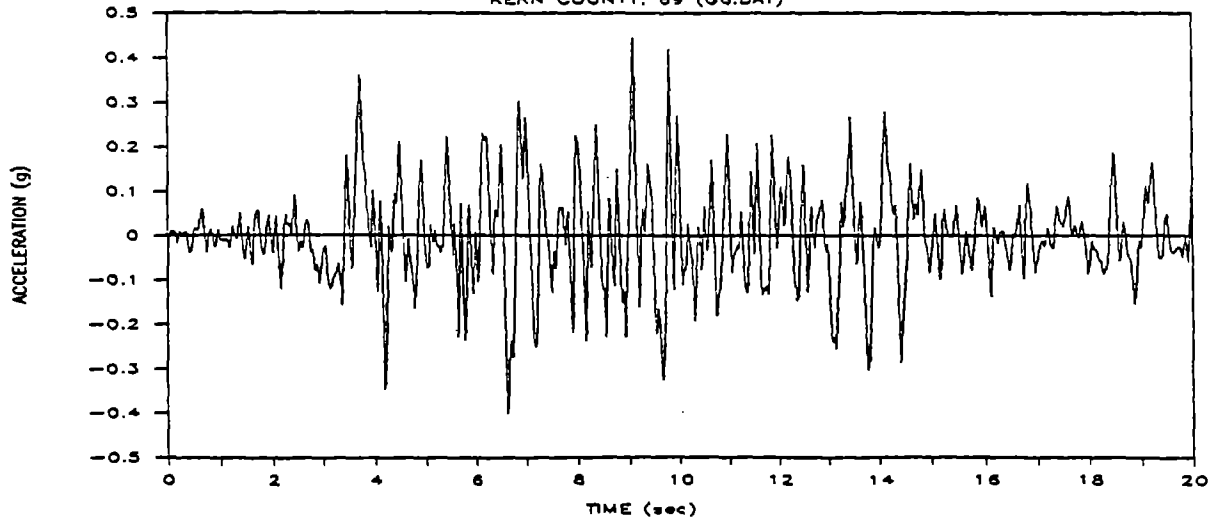
### ACCELERATION TIME HISTORY (ZONE 4)

JAMES ROAD. 140 (G5.DAT)

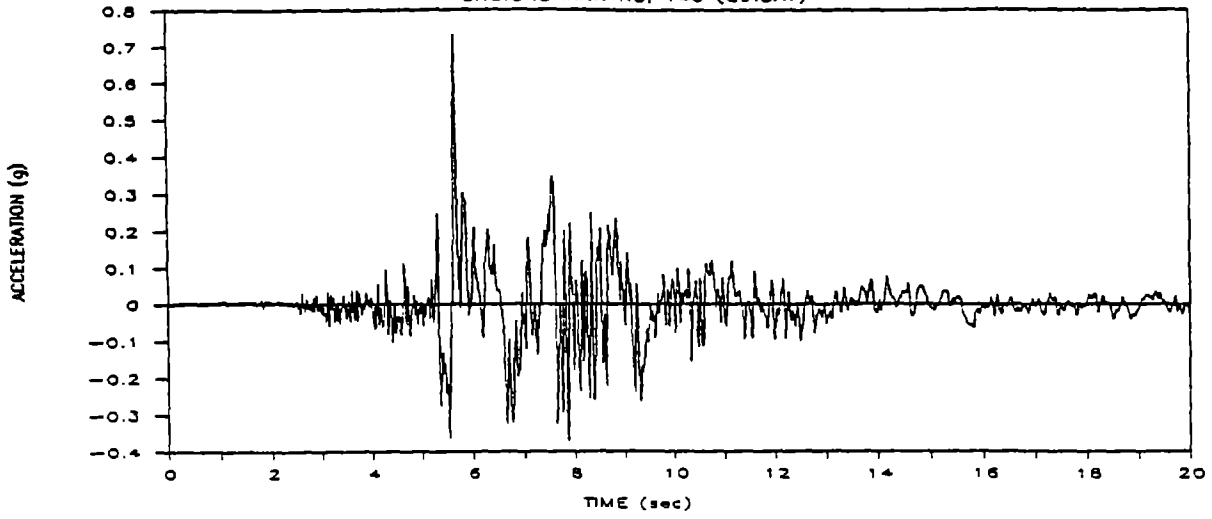


### ACCELERATION TIME HISTORY (ZONE 4)

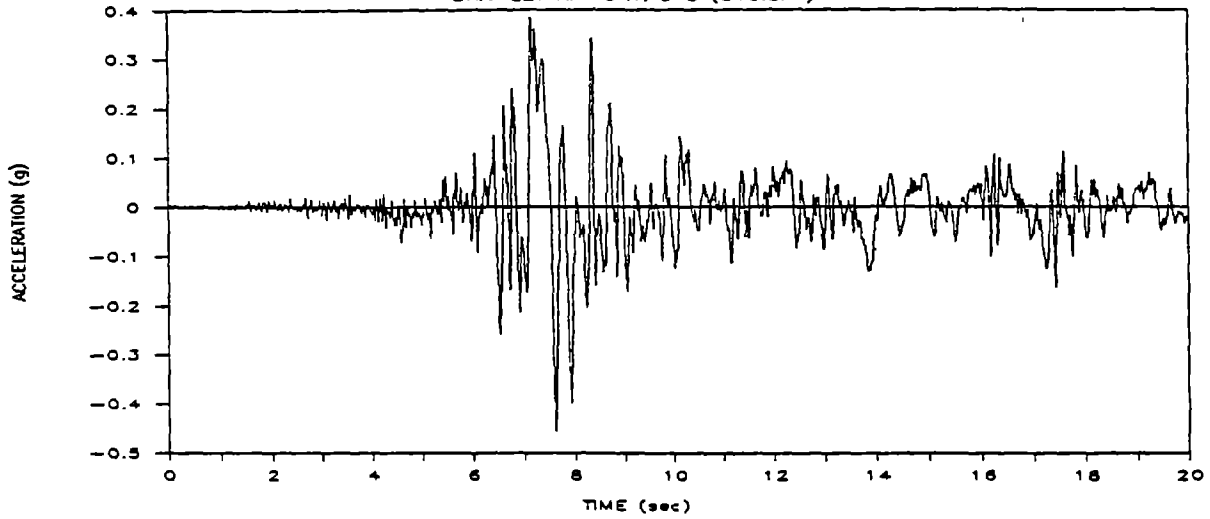
KERN COUNTY. 69 (G6.DAT)



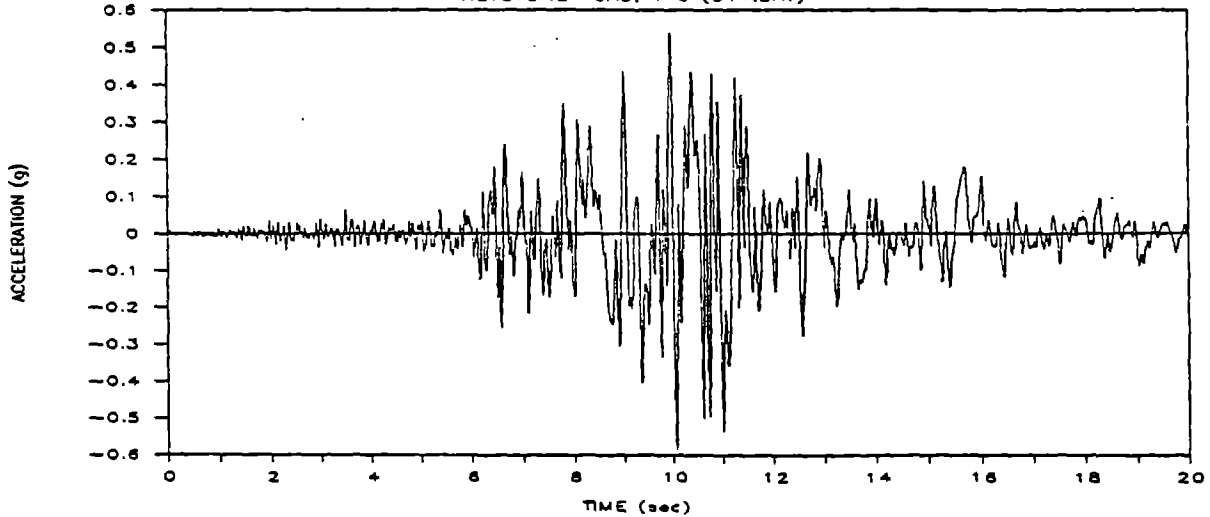
ACCELERATION TIME HISTORY (ZONE 4)  
CRUICKSHANK RD. 140 (G9.DAT)



ACCELERATION TIME HISTORY (ZONE 4)  
BRAWLEY AIRPORT, 315 (G10.DAT)



ACCELERATION TIME HISTORY (ZONE 4)  
KEYSTONE ROAD, 140 (G11.DAT)





## **APPENDIX B**

### **INTERPRETATION OF DIAPHRAGM STIFFNESS DATA**

This appendix is a concise description of the data available for estimating the in-plane stiffness of various types of diaphragms and is based on the ABK Joint Venture Topical Test Report TR-05 [1]. This report presents the interpretation of the quasi-static and dynamic tests on full-scale diaphragms. A nonlinear hysteretic element was developed for use in the nonlinear dynamic analysis of diaphragms. Properties for the model were obtained from the quasi-static test data and the model was correlated with the dynamic tests.

#### **Diaphragm Spring Element**

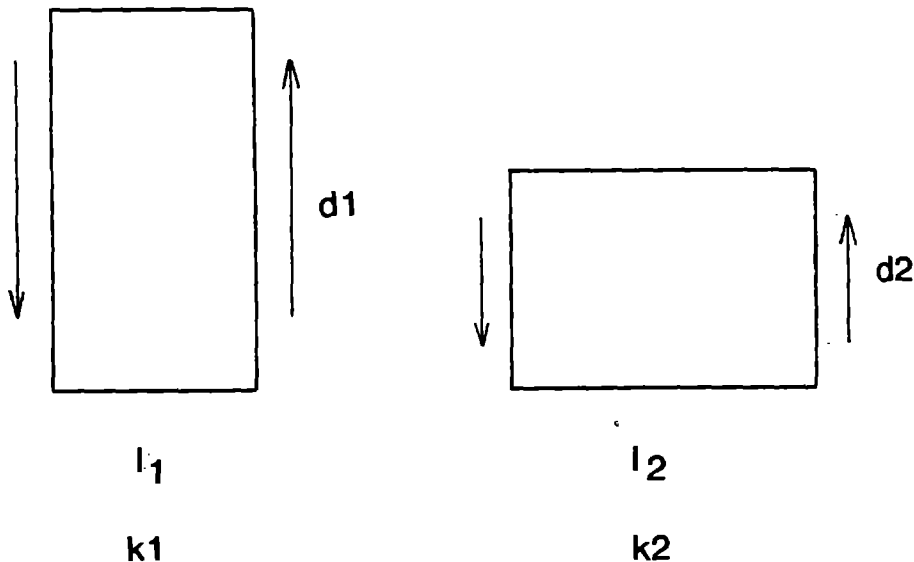
The diaphragm spring element used in the analysis represents the stiffness of a given panel of the diaphragm or of the entire diaphragm. The tests were conducted on a 20 feet by 20 feet diaphragm panel. The spring constant that is of most use in the SAP90 linear elastic analysis is the initial spring stiffness. Various different types of diaphragms were used in the tests. Two were chosen to represent the plywood and steel deck options. They were:

1. Type N - 1/2" Plywood Deck, Blocked and Chorded
2. Type Q - 20 gage Steel Deck, button punched, seams at 18" o.c.

From the ABK Topical Report TR-03 [2], estimates for the initial spring stiffness for the 20 feet by 20 feet diaphragm section were made. They are given in Table B.1. Since the diaphragms for the various buildings studied in the TCCMAR Task 2 study had different sizes, a set of scaling rules was developed to convert from the estimates for the 20 foot by 20 foot deck to that for the appropriate building. These rules are given next:

## Scaling Rules

Scaling of the properties given in Table B.1 for diaphragms with other sizes and aspect ratios are accomplished using the following relationships:



$$k_2 = (d_2/d_1) \times (l_1/l_2) \times k_1$$

## REFERENCES

1. Interpretation of Diaphragm Tests - ABK Topical Report TR-05, ABK Associates, El Segundo, CA.
2. Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing - ABK Topical Report TR-03, December 1981, ABK Associates, El Segundo, CA



**TABLE B.1**

**Stiffness Properties for Diaphragms**

Type	Description	Initial Stiffness (kips/inch)
N	1/2" Plywood Deck, Blocked & Chorded	10.0
Q	20 gage Steel Deck, button punched	32.5

