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SEISMIC PERFORMANCE STUDY TMS SHOPPING CENTER ELASTIC TIME HISTORY ANALYSIS USING SAP90

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Ewing/Kariotis/Englekirk & Hart

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CHAPTER 1
EXECUTIVE SUMMARY

The SAP90 computer analysis of the TMS shopping center 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 TMS shopping center is a rectangular masonry shear wall structure with an open front and a steel deck 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 stiffness 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 stiffness 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 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.

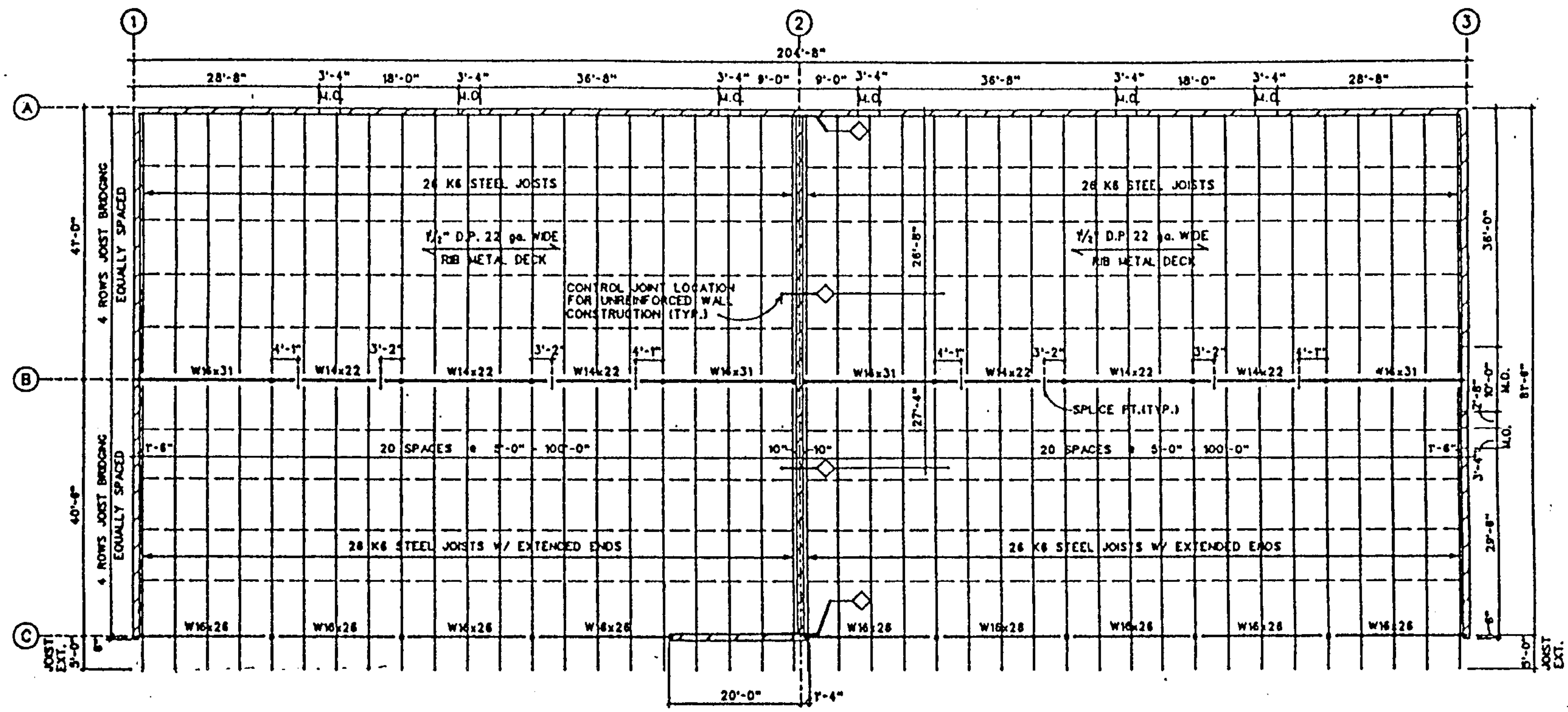
The building was then studied to evaluate its seismic performance. Several earthquake time histories were run using SAP90 and three displacement response quantities were recorded - a) the maximum wall deformation at the diaphragm level, b) the maximum relative diaphragm deformation and c) the maximum mid height wall deformation. These response quantities were also selected because these quantities were studied by the other members of the TCCMAR task 2 team using the LPM and FEM inelastic analysis programs.

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 of the research were met.

CHAPTER 2
BUILDING DESCRIPTION

The TMS shopping center is a one story rectangular box shaped structure. It is 205 feet long, 82 feet wide and 16 feet high. The southern side of the building has only a 20 feet long shear wall. There is a long shear wall running the entire north side of the building (i.e east-west). There are three north-south running walls at the east, middle and west side of the center. The building is shown in Figures 2.1 and 2.2.

All walls are 7.625 inch nominal thickness and are reinforced concrete masonry. All units were grouted solid. The roof diaphragm was studied for only one case - a steel deck with stiffness obtained from the ABK reports. Also, the behavior of the building was studied only for east-west motion. All walls were assumed to be hinged at the base.



ROOF FRAMING PLAN

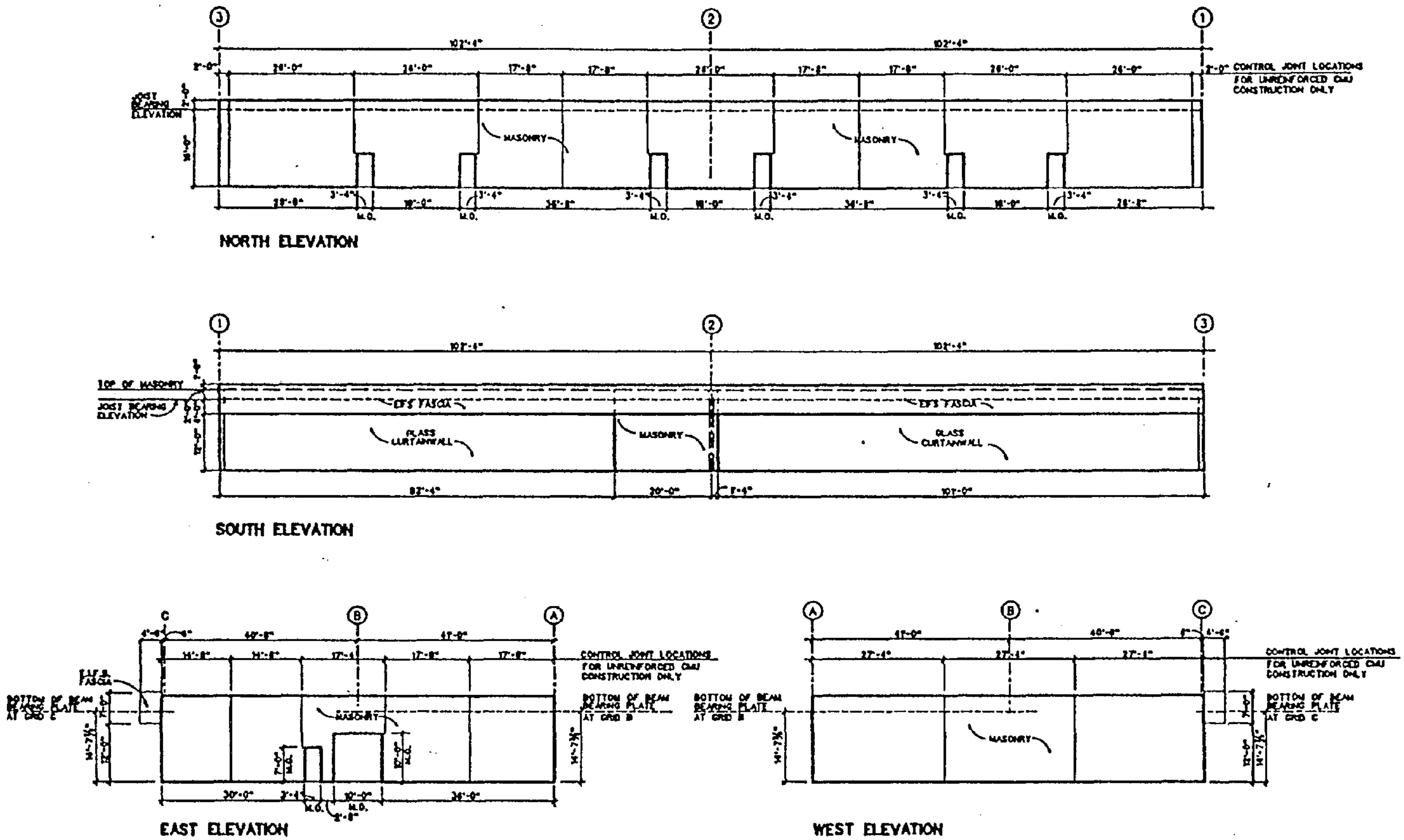
NORTH

TMS SHOPPING CENTER

NOTES:

1. ROOF DEAD LOAD - ~~20~~ 15 PSF
2. ROOF SNOW LOAD - 30 PSF
3. DESIGN WIND PRESSURE - 20 PSF
4. SEISMIC ZONE ≤ 1
5. COLUMNS ON GRIDLINE B ARE TS5x5x5/16
6. COLUMNS ON GRIDLINE C ARE TS4x4x1/4
7. BEAM BEARING PLATE AT B-1 AND B-3 IS 3/4x5x10
8. BEAM BEARING PLATE AT C-1,C-2 AND C-3 IS 3/8x5x7
9. ROOF CONSTRUCTION: BUILT-UP ROOFING, 2" RIGID INSULATN ON METAL ROOF DECK - 15 PSF (FLEXIBLE DIAPHRAM)
10. WALL CONSTRUCTION IS SINGLE WYTHE CMU. SEE ELEVATION VIEWS FOR EXTERIOR WALL CONTROL JOINT LOCATIONS WHEN CMU IS UNREINFORCED.
11. ALLOWABLE SOIL BEARING CAPACITY = 3000 PSF

FIGURE 2.1 - ROOF FRAMING PLAN



TMS SHOPPING CENTER

FIGURE 2.2 - 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 quantify how one could analyze a masonry structure with flexible diaphragms with a 3-D model. In such a model we want to show how we can take into account 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 four walls. Each element is seven feet long by four feet high and thus has an aspect ratio of 1:1.75. There are a total of 276 elements in the model. All of the elements stiffness is specified through the modulus of elasticity and the element thickness. Membrane stiffness is 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 rotational fixity to the walls. The finite element discretization scheme for the short, long and the 20 foot wall are shown in Figures 3-2A-C.

3.3 Modeling the Diaphragm

The diaphragm for this building was a metal deck. This diaphragm is flexible relative to the walls. Therefore, the SAP90 model had to model the diaphragm stiffness in both directions (NS and EW). The 1:2.5 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

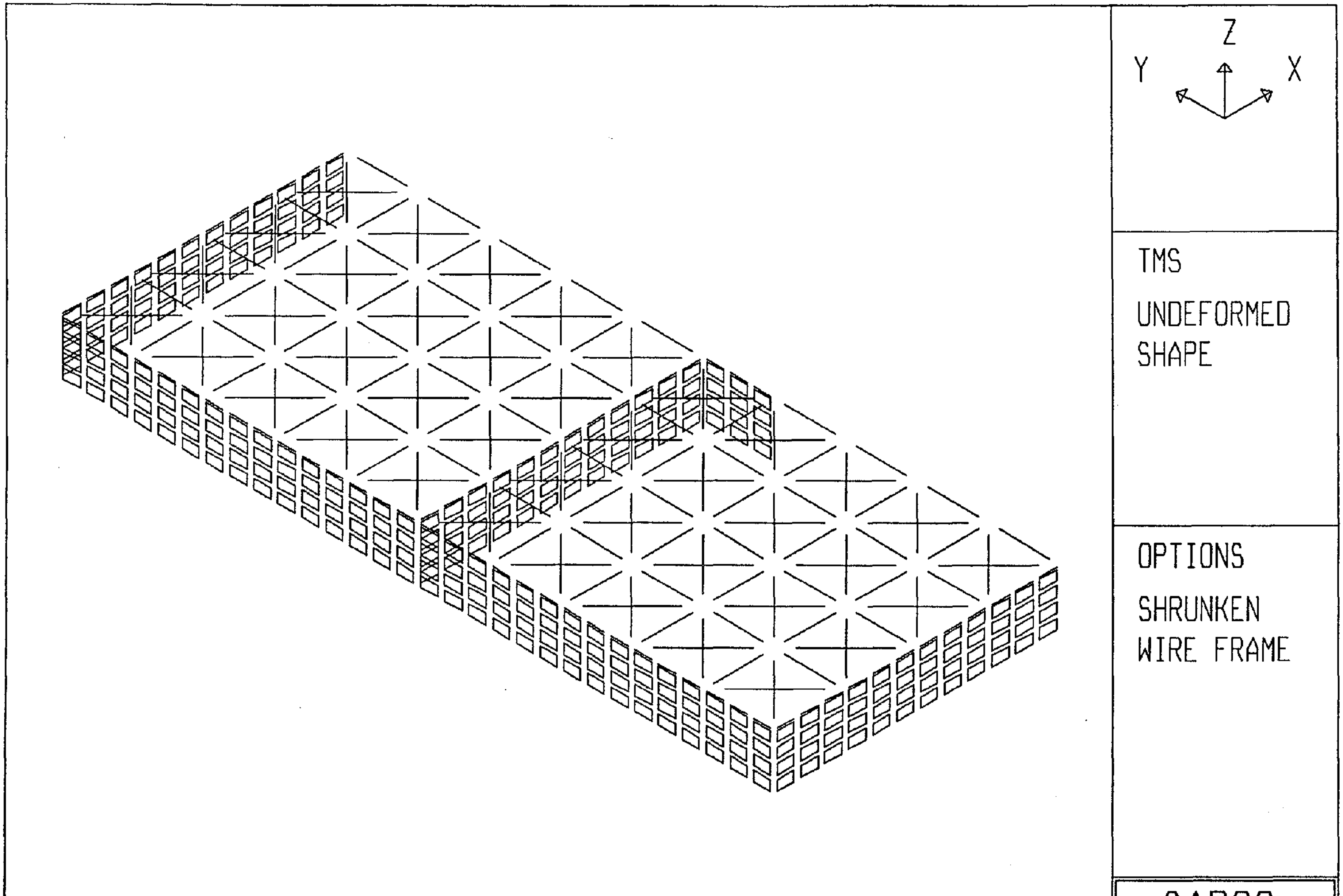


FIGURE 3.1 - DISCRETIZED 3-D SAP90 MODEL

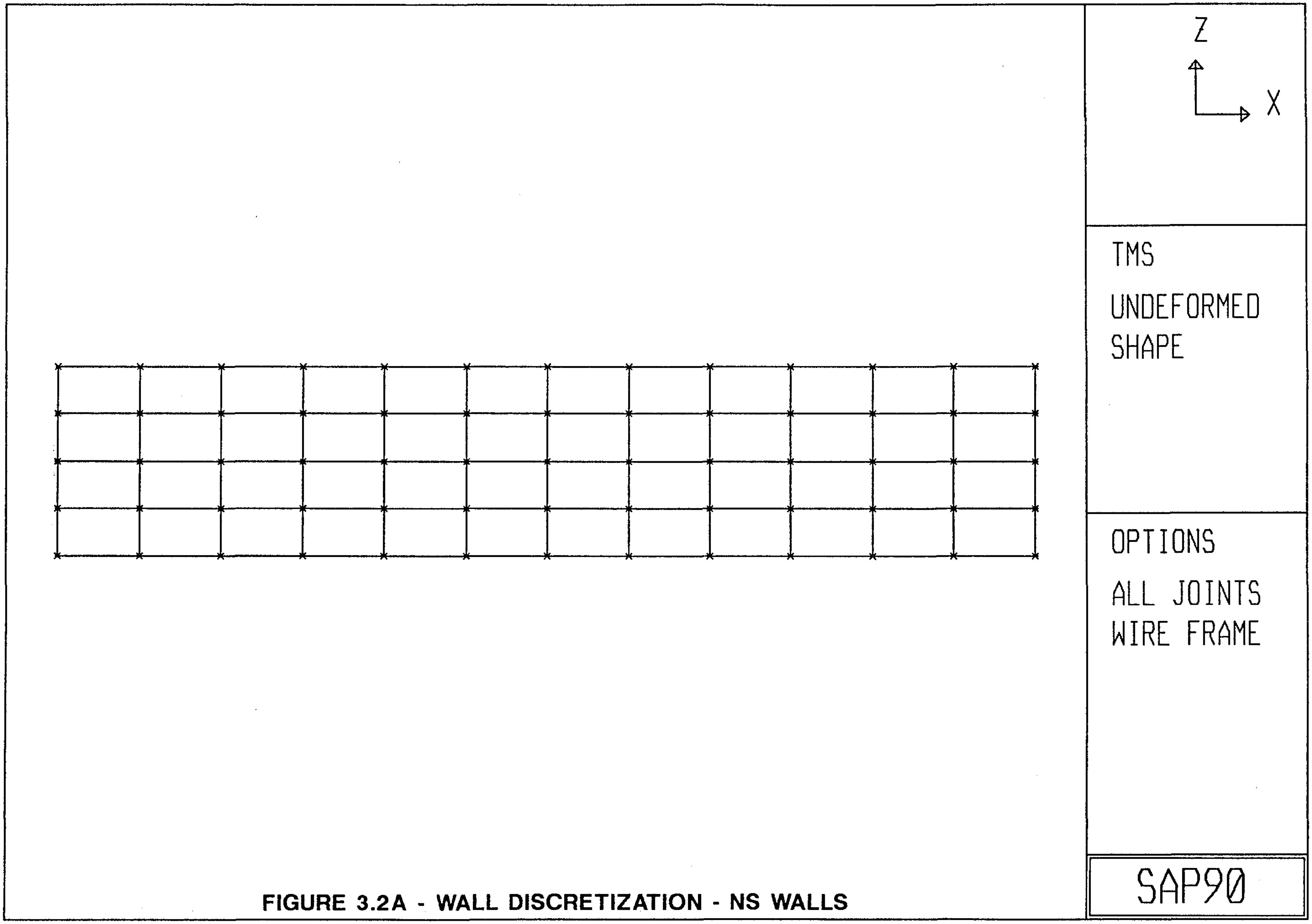


FIGURE 3.2A - WALL DISCRETIZATION - NS WALLS

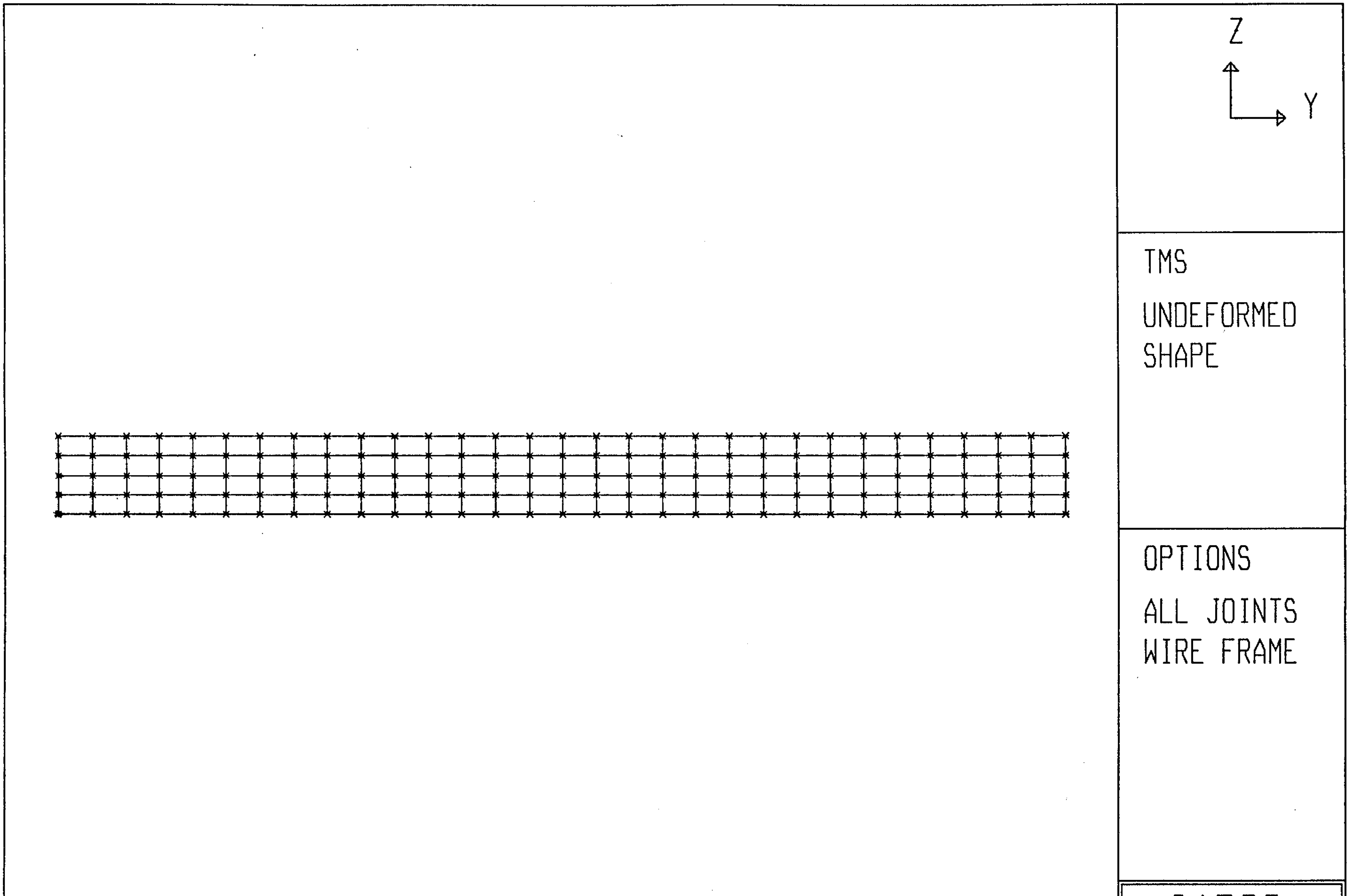


FIGURE 3.2B - WALL DISCRETIZATION - LONG WALL

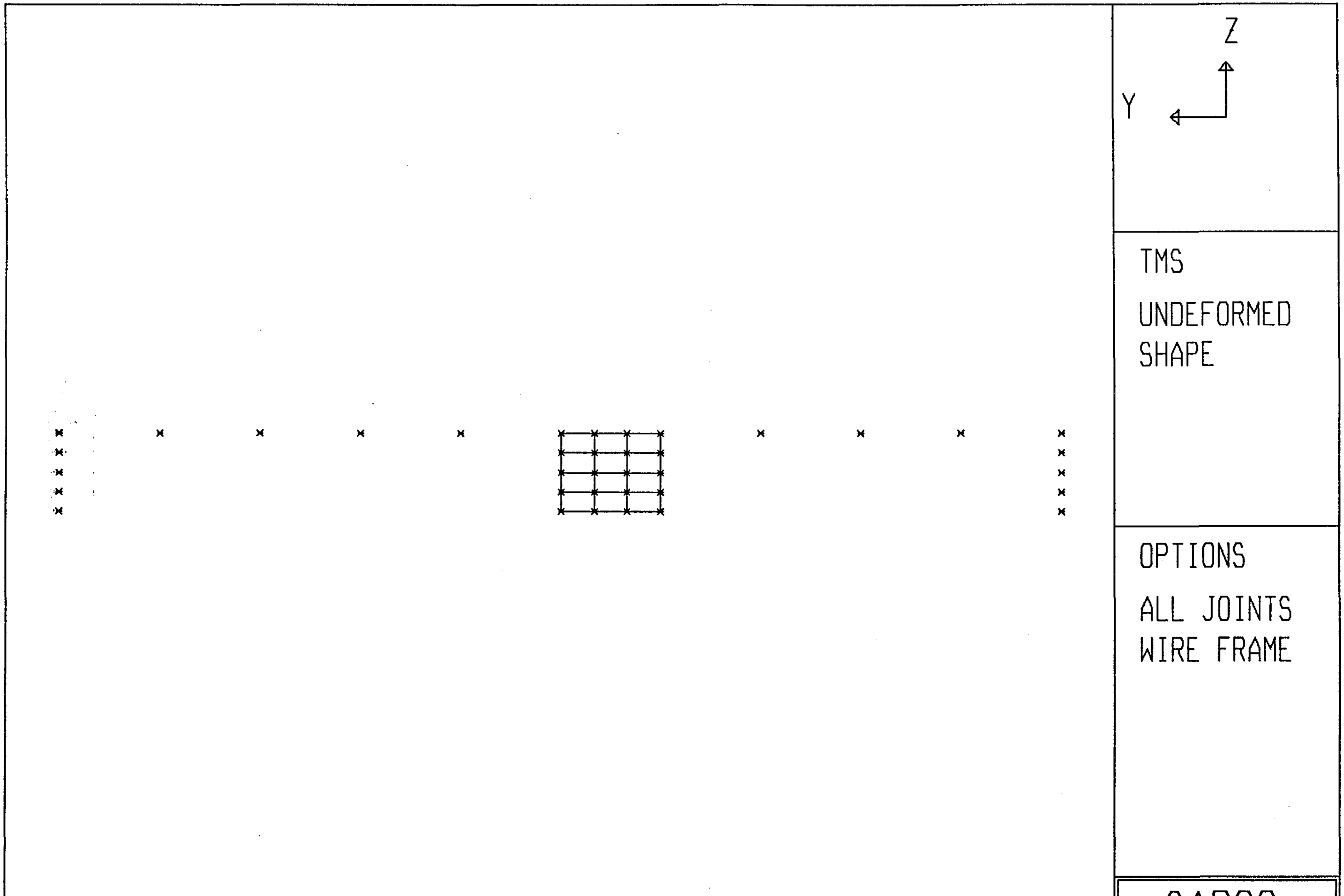


FIGURE 3.2C - WALL DISCRETIZATION - 20' WALL

truss analogy was selected because it gave a good picture of the actual diaphragm behavior i.e. panels shearing between each other for any direction of the ground motion input.

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 10 truss panels in the long direction and 4 in the short direction. Each truss panel is 20.5 feet by 20.5 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 is loaded at one end with a unit load and the other side of the truss is assumed pinned along all the nodes. 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 stiffness 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 members areas are scaled in either direction, the stiffness in the other direction is automatically scaled- as per the ABK report.

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 TMS shopping center. The diaphragm panels too could go into the inelastic range. Since the diaphragm panels closest to the wall deform the most (relatively), their stiffnesses were set at 60 percent of the initial stiffness and the panels in the center had their stiffness at 80 percent of the initial. 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.

The walls were subdivided into two zones - an inner and an outer. These two

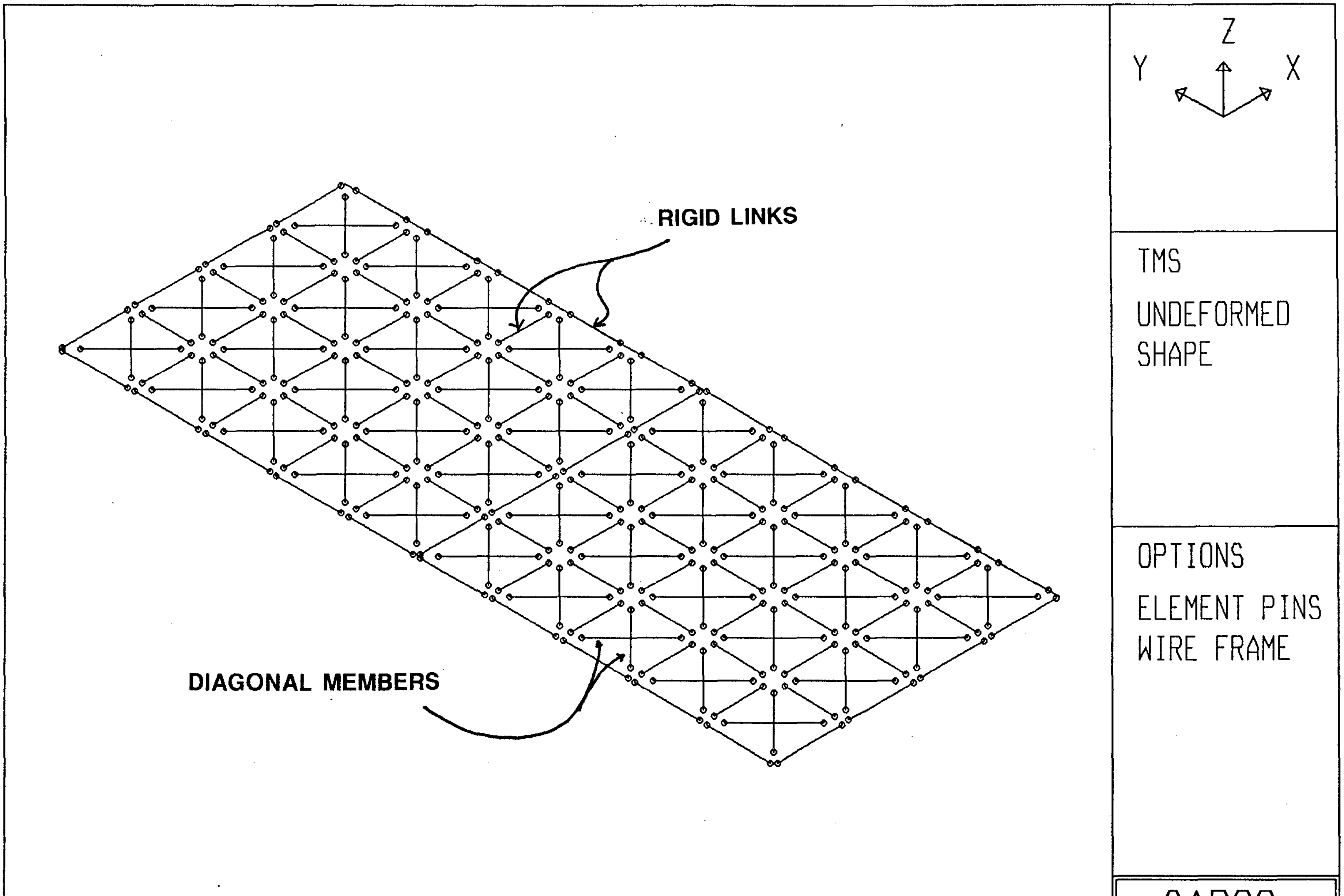


FIGURE 3.3 - TRUSS MODEL FOR FLEXIBLE DIAPHRAGM

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 stiffness (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 stiffness were decreased, the wall deflection at the top also increased. However, it was found that the walls were so stiff that upon reducing the stiffness 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 stiffness 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 height at the wall centerline was used as a reference. Since the wall is divided into twelve strips horizontally, the four end elements (two on either side) are of greater stiffness than the center eight elements.

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

$$I_{eff} = (M_{cr}/M_a)^3\, I_g + [1 - (M_{cr}/M_a)^3]\, I_{cr} \qquad (3\text{-}1)$$

where

I_{eff} = effective moment of inertia
I_g = gross moment of inertia
I_{cr} = cracked moment of inertia

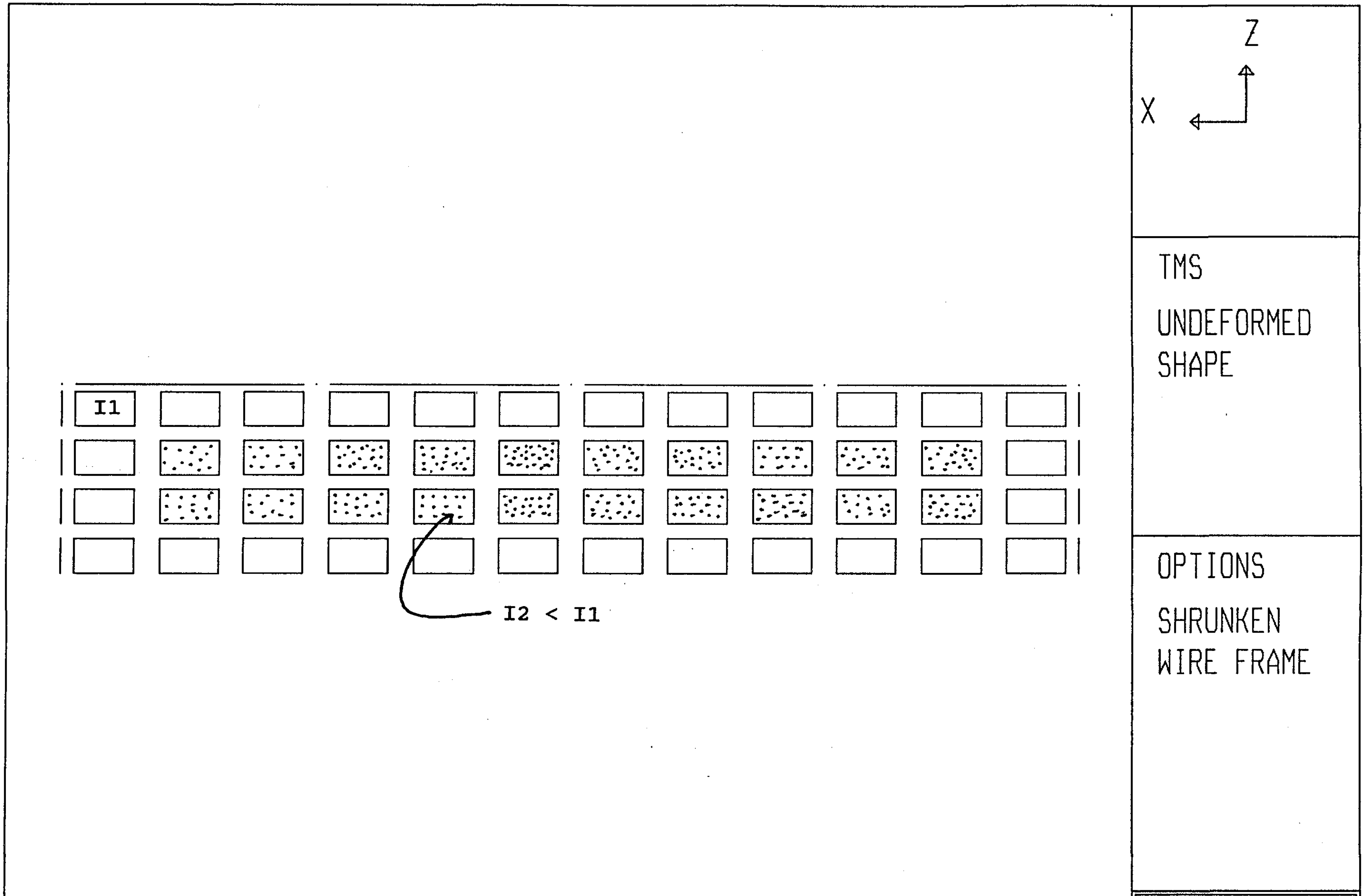


FIGURE 3.4 - STIFFNESS ZONES - NS WALLS

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 engineers believe 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 does 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 TMS SHOPPING CENTER

4.1 General

The time history runs were performed on the SAP90 model for nine different time histories - records 1-6, and 9-11. The earthquake records used in this study are described in Appendix A. For each time history, one run was performed for each record- for the east-west motion. Both positive and negative maximum responses were calculated. All 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.

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-A, 4.2-B, 4.3-A and 4.3-B show the first two modes for the north-south and east-west directions for the TMS center. Table 4.1 shows the first three periods of vibration of the TMS center for the steel deck diaphragm in the NS and EW directions. The final iterated wall stiffness did not vary much for the nine earthquakes for a given direction (EW or NS), all being between 10%

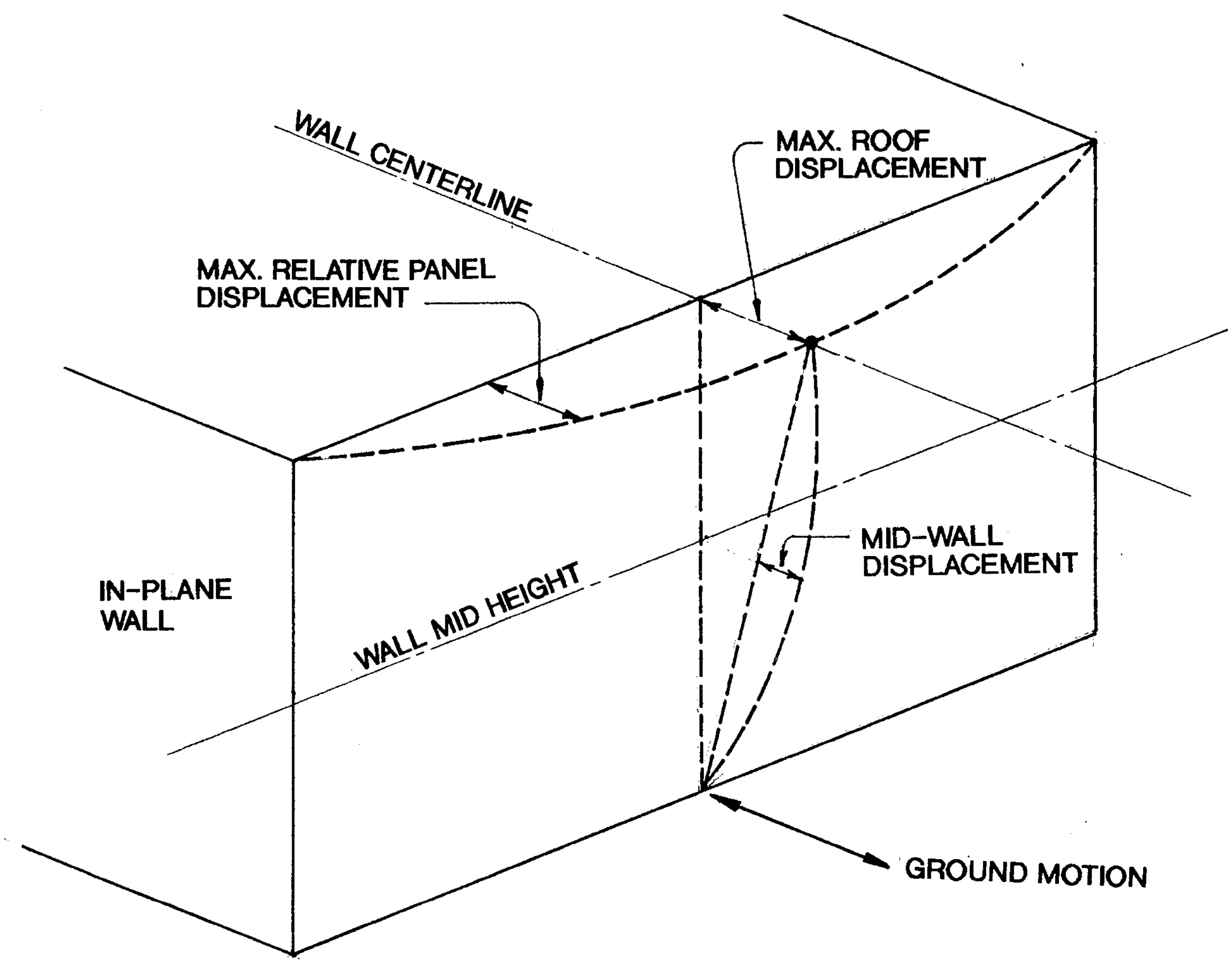


FIGURE 4.1 - OUT OF PLANE RESPONSE QUANTITIES

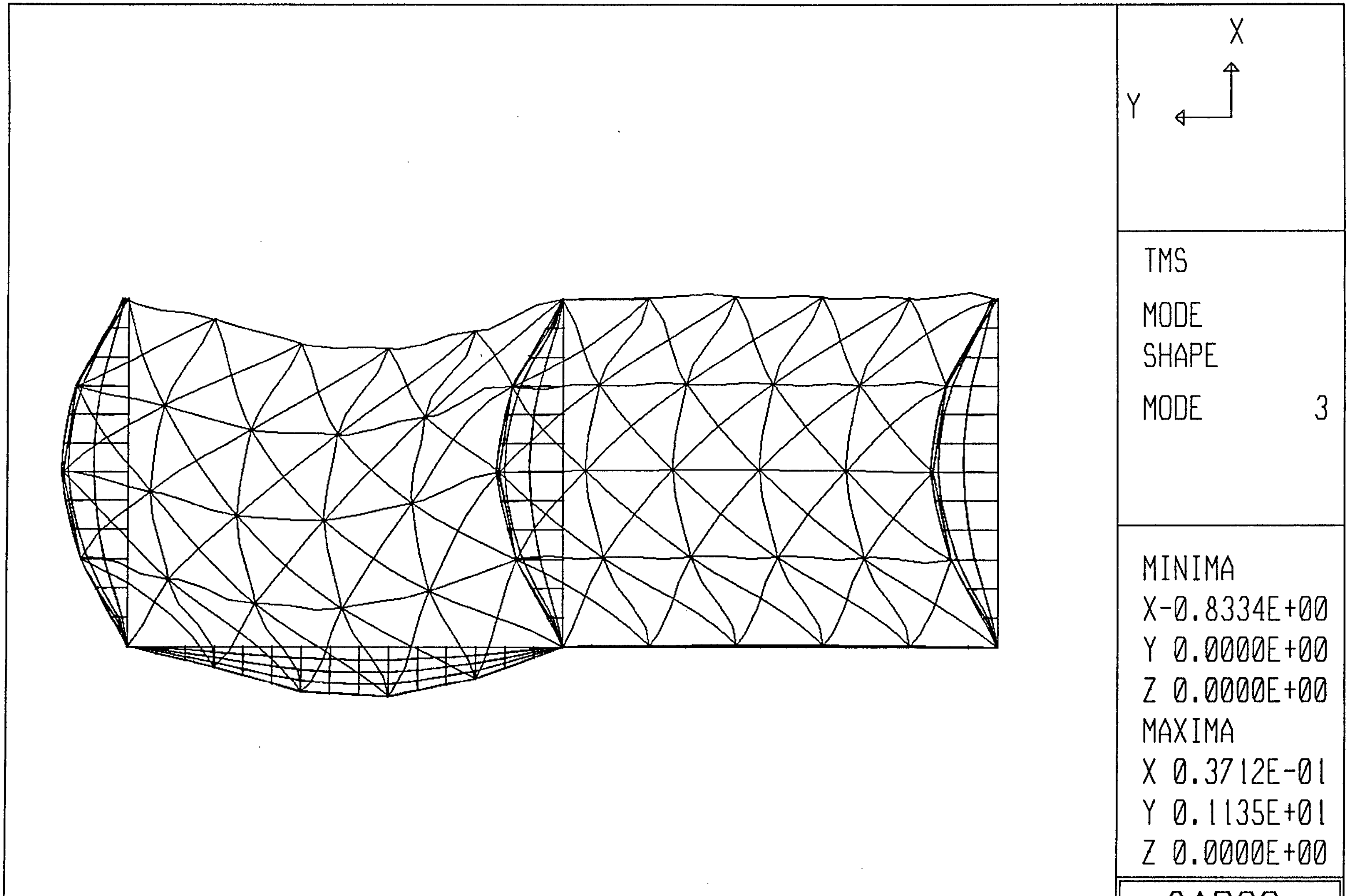


FIGURE 4.2-A - FIRST MODE - EW DIRECTION

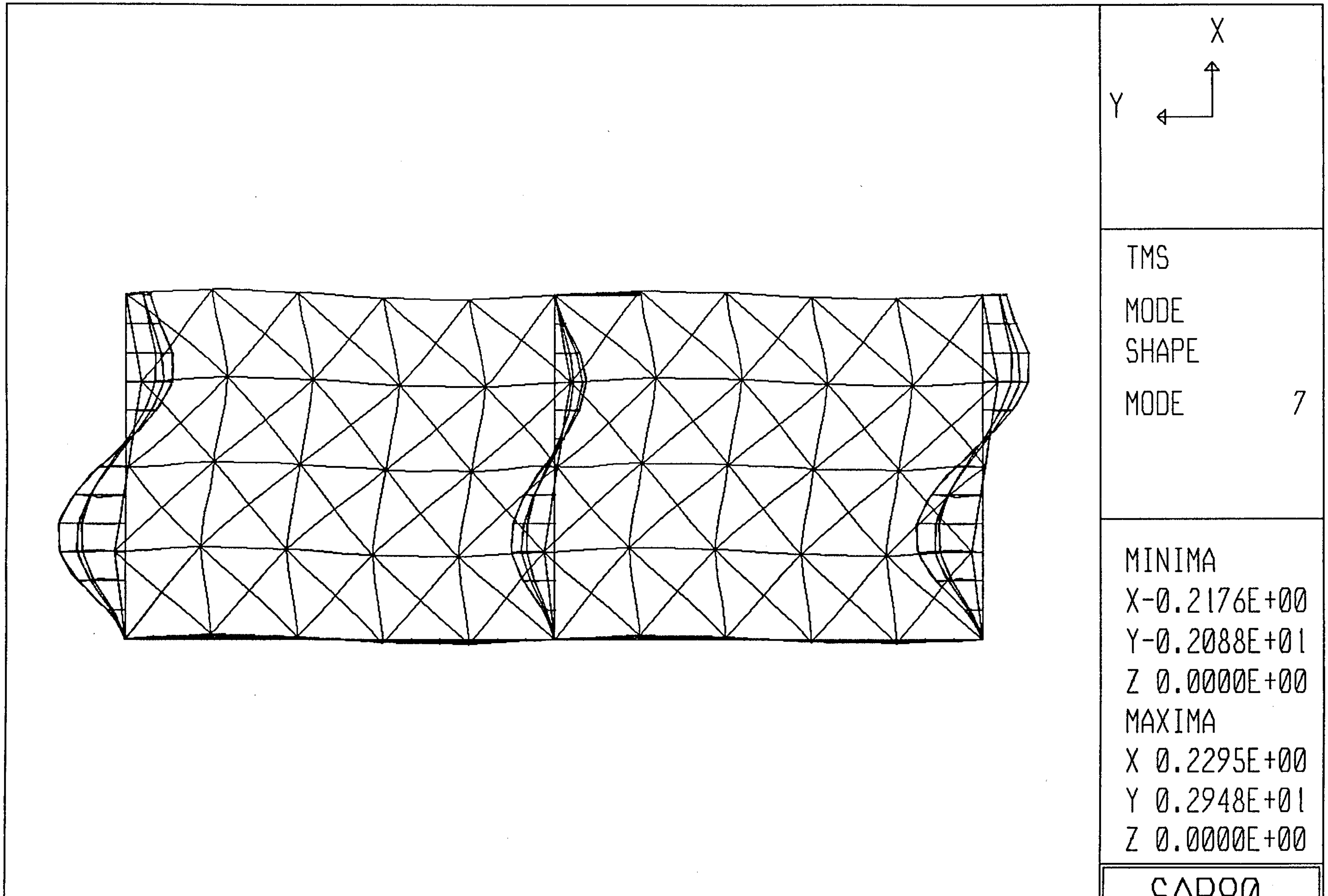


FIGURE 4.2-B - SECOND MODE - EW DIRECTION

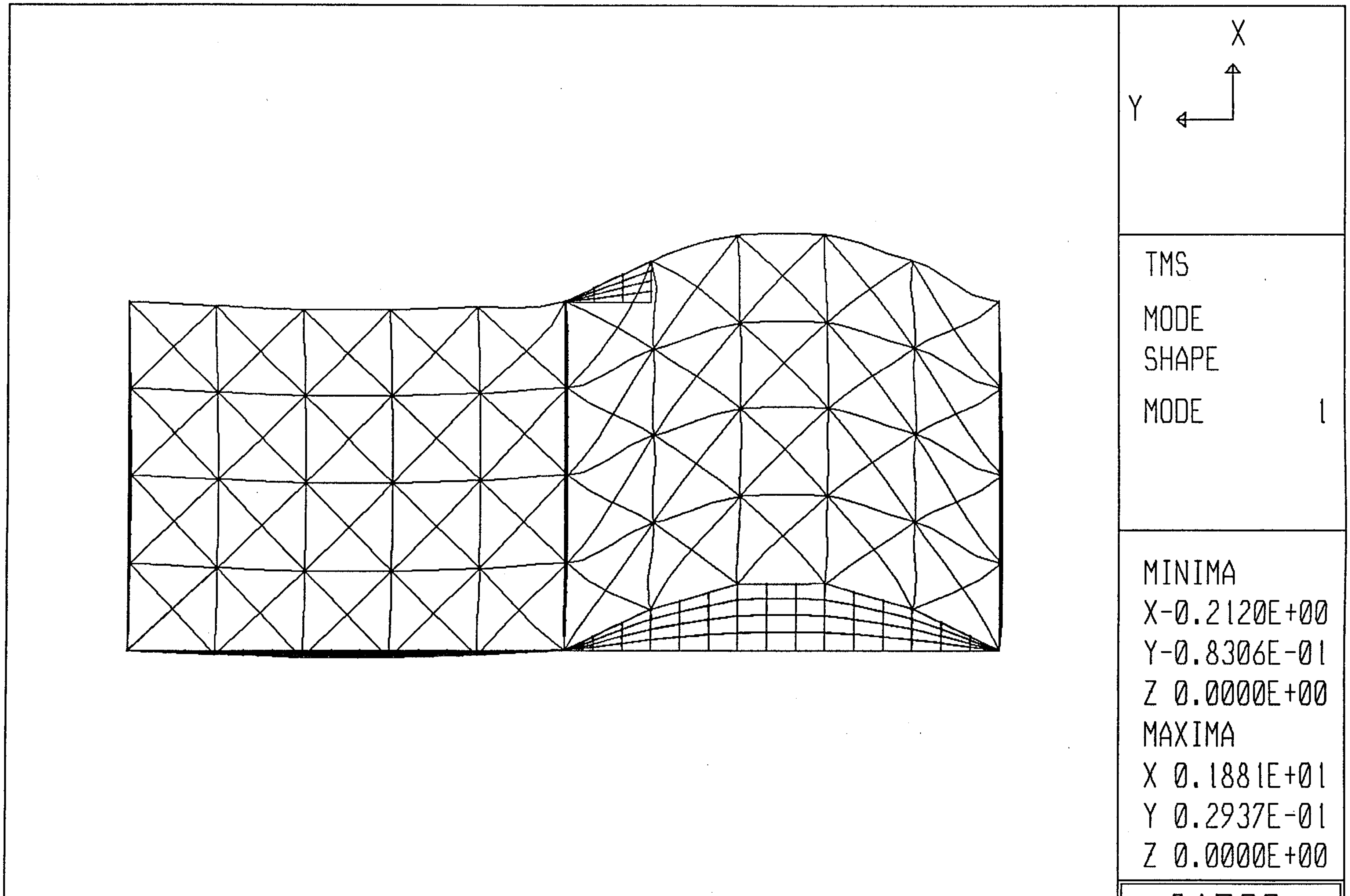


FIGURE 4.3-A - FIRST MODE - NS DIRECTION

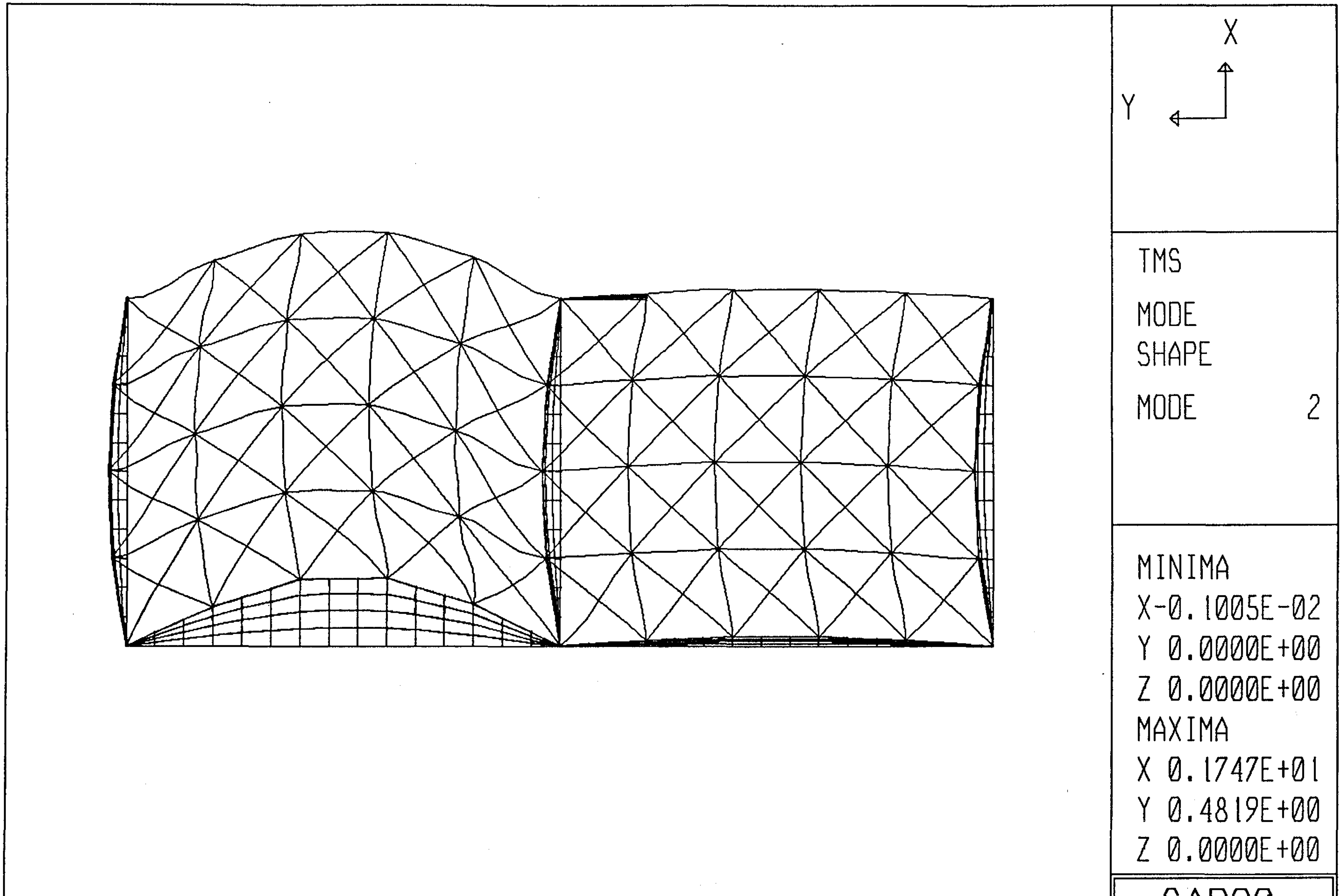


FIGURE 4.3-B - SECOND MODE - NS DIRECTION

TABLE 4.1

TMS CENTER – NATURAL PERIODS OF VIBRATION

		Metal Deck Diaphragm
Direction	Mode	Period (s)
EW	1	0.31
	2	0.18
	3	0.14
NS	1	0.33
	2	0.32
	3	0.12

and 30% of the gross stiffness. Hence, the natural period of vibration of the structure did not vary significantly for each of the nine earthquakes.

4.3 Results

The results of the analyses are shown in Table 4.2. Figures 4.4 to 4.9 also show the time history of selected responses for the steel deck diaphragm.

TABLE 4.2 – DISPLACEMENT RESPONSE RESULTS

Metal Deck, EW Motion

	Building Centerline		Panel Nearest EW Wall			
Motion No.	Roof Displ (in)	Mid–Wall Displ (in)	Roof Displ (in)	Mid–Wall Displ (in)	Drift (%)	Base Shear Coef.
1	−0.16	−0.51	−0.13	−0.41		0.50
	0.18	0.51	0.15	0.43	0.09	
2	−0.17	−0.60	−0.14	−0.44		0.56
	0.23	0.60	0.19	0.48	0.12	
3	−0.30	−0.85	−0.23	−0.68	0.16	0.47
	0.23	0.83	0.19	0.64		
4	−0.27	−0.57	−0.22	−0.47	0.14	0.37
	0.15	0.57	0.12	0.44		
5	−0.24	−0.65	−0.19	−0.53	0.13	0.49
	0.19	0.59	0.15	0.45		
6	−0.26	−0.76	−0.21	−0.62	0.14	0.66
	0.22	0.80	0.17	0.59		
9	−0.27	−0.98	−0.22	−0.69	0.14	0.54
	0.15	0.79	0.12	0.57		
10	−0.18	−0.75	−0.15	−0.54	0.09	0.61
	0.13	0.86	0.10	0.64		
11	−0.34	−1.13	−0.27	−0.75		0.77
	0.44	1.19	0.36	0.90	0.23	
Average	0.23	0.75	0.18	0.57	0.14	0.55

Notes:

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

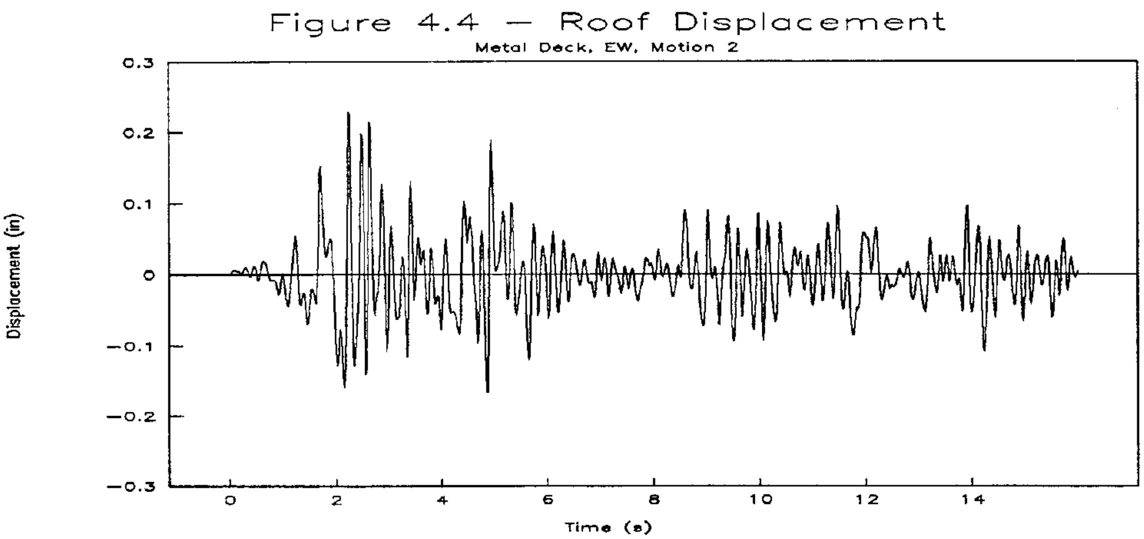


Figure 4.4 — Roof Displacement
Metal Deck, EW, Motion 2
Displacement (in)
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
0
2
4
6
8
10
12
14
Time (s)

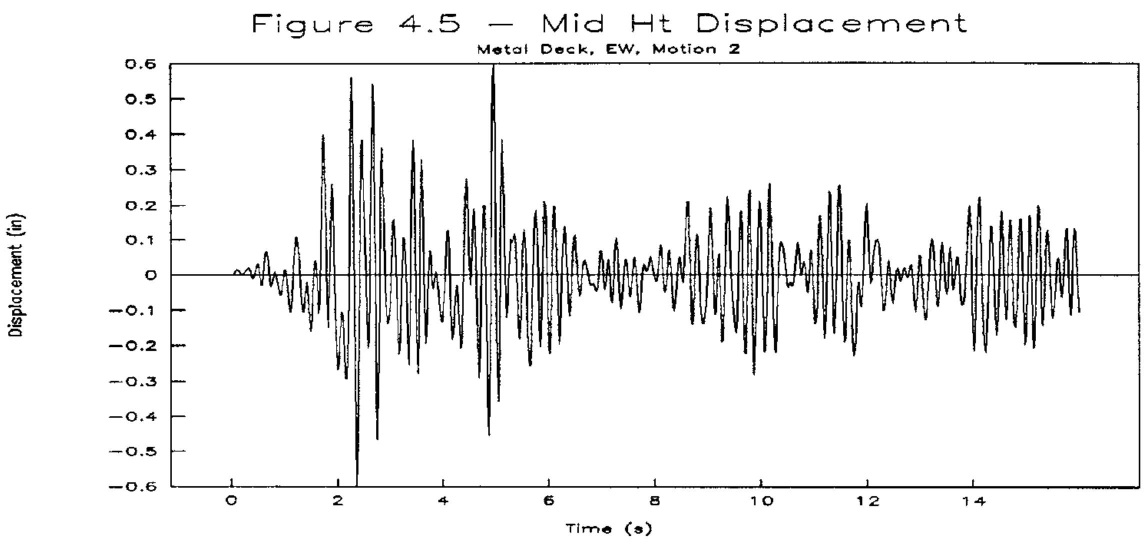


Figure 4.5 — Mid Ht Displacement
Metal Deck, EW, Motion 2
Displacement (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
−0.4
−0.5
−0.6
0
2
4
6
8
10
12
14
Time (s)

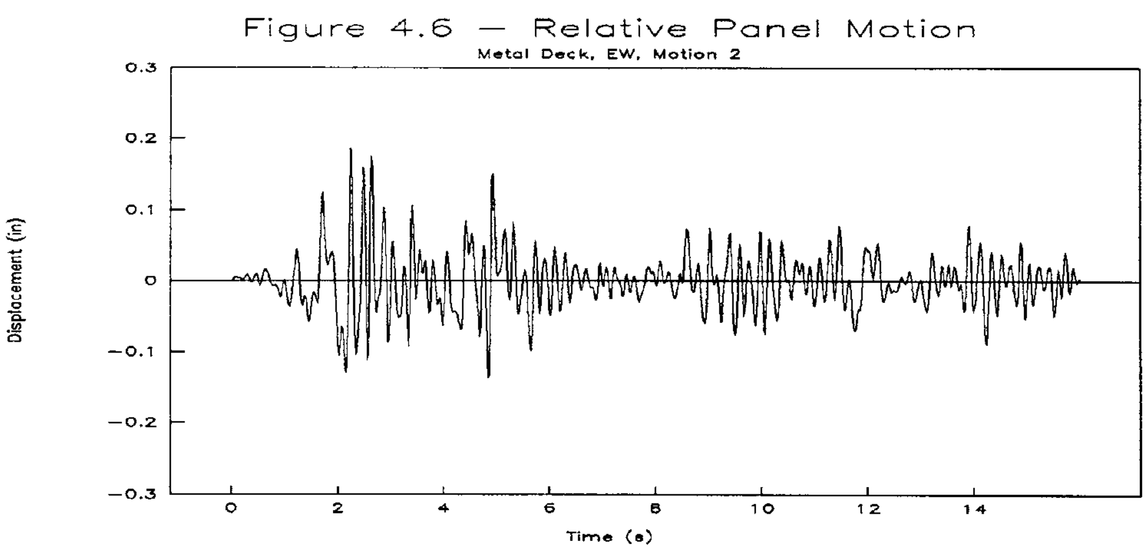


Figure 4.6 — Relative Panel Motion
Metal Deck, EW, Motion 2
Displacement (in)
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
0
2
4
6
8
10
12
14
Time (s)

Figure 4.7 — Roof Displacement

Metal Deck, EW, Motion 3

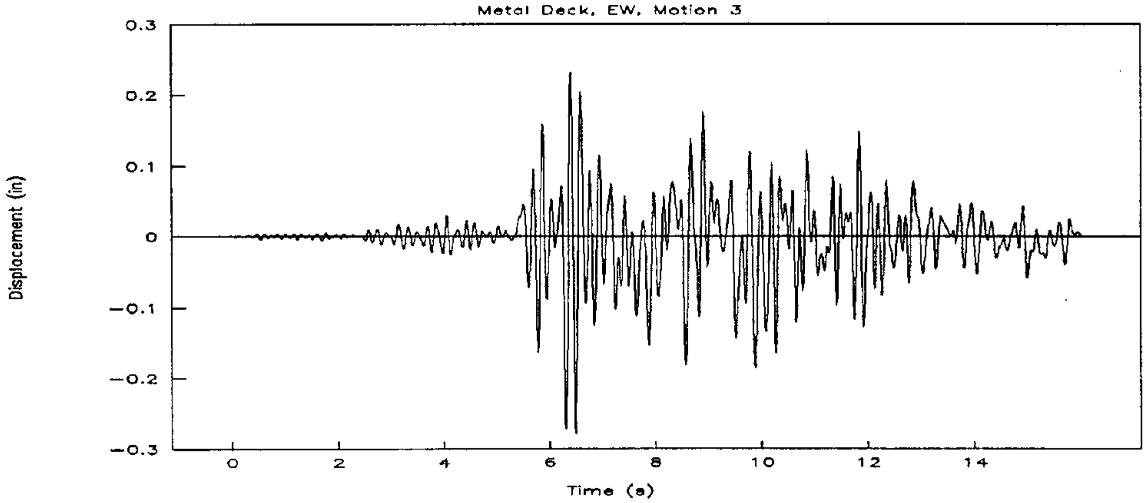


Figure 4.8 — Mid Ht Displacement

Metal Deck, EW, Motion 3

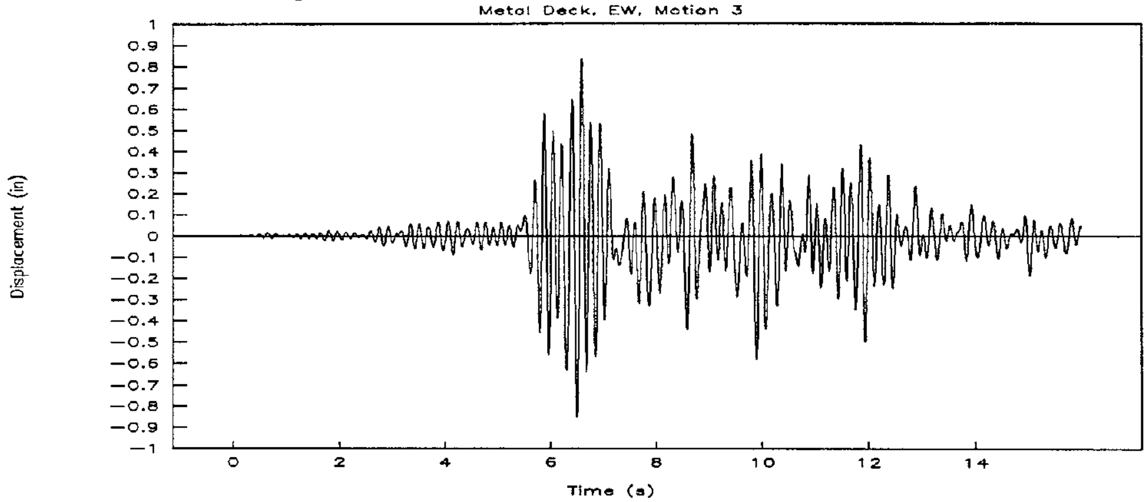
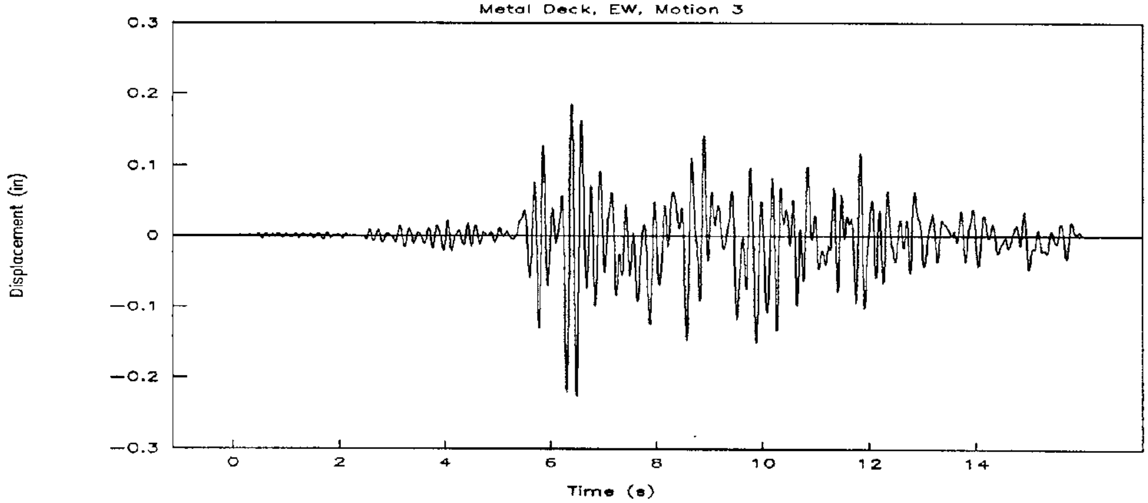


Figure 4.9 — Relative Panel Motion

Metal Deck, EW, Motion 3



CHAPTER 5
CONCLUSIONS

Results from the analysis of the TMS center can be divided into those describing 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 comprises 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 force/deformation levels. The more refined the mesh, the greater the complexity of the anlysis and the 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 help in reducing the size of the task.

The flexible truss model for the diaphragm is highly recommended due to its ease of modeling. It is able to show the displaced roof shape quite well. However, the actual diaphragm stiffness should be available for scaling the model.

Only the steel deck diaphragm was studied for this shopping center. It is of interest to note that the overall drift ratios for this building were larger than those of the DPC Gymnasium studied in the EKEH Report 2.1-8 [4] even though the TMS center is not as tall and the same steel deck diaphragm was used. A contributing factor is probably the lack of walls on the south side of the building (there is only a 20 feet long wall). It was found that the 20 feet long wall carried about ten percent of the total base shear during the ground motion which is roughly in proportion to its length. However, a similar set of ground motion runs without the 20 feet wall caused a five percent increase in the base shear - this shear will have to be factored into the diaphragm design. Therefore the presence of this wall does not seem to have any detrimental effect on the overall structural behavior.

Another interesting feature was seen in the fundamental modes of the building for East-West motion. The presence of the middle wall effectively acted like a dead zone. In effect, it uncoupled the diaphragm motions on the east and west sides of the building. It would have been of interest to study the load path across this wall for North-South motion.

A comparison of the elastic and inelastic model results for this building will be presented as a part of the TCCMAR Phase 10 in the near future.

CHAPTER 6
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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

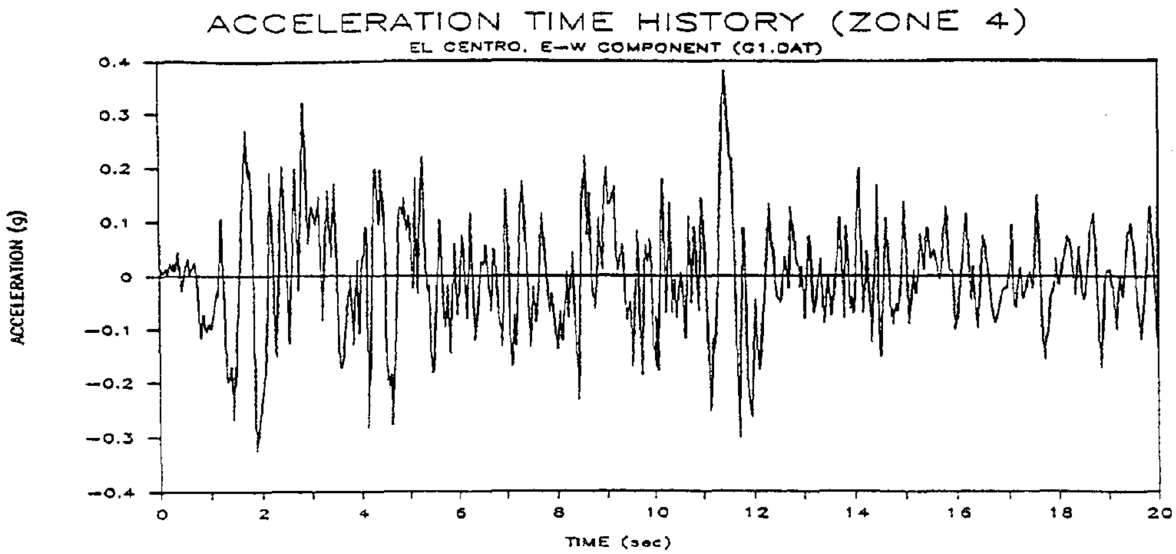
1. Kariots, J.C., and Waqfi, O.M, "Trial Designs Made in Accordance with Tentative Limit States Design Standards For Reinforced Masonry Buildings", Kariotis and Associates Report 9.1-2, February 1992, Kariotis and Associates, South Pasadena, California.

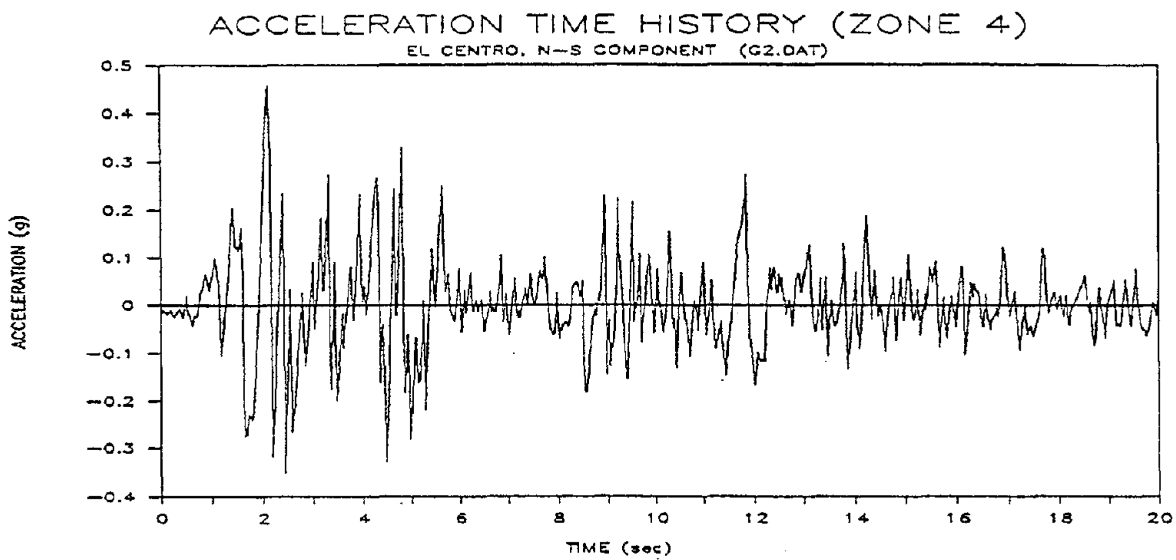
TABLE A

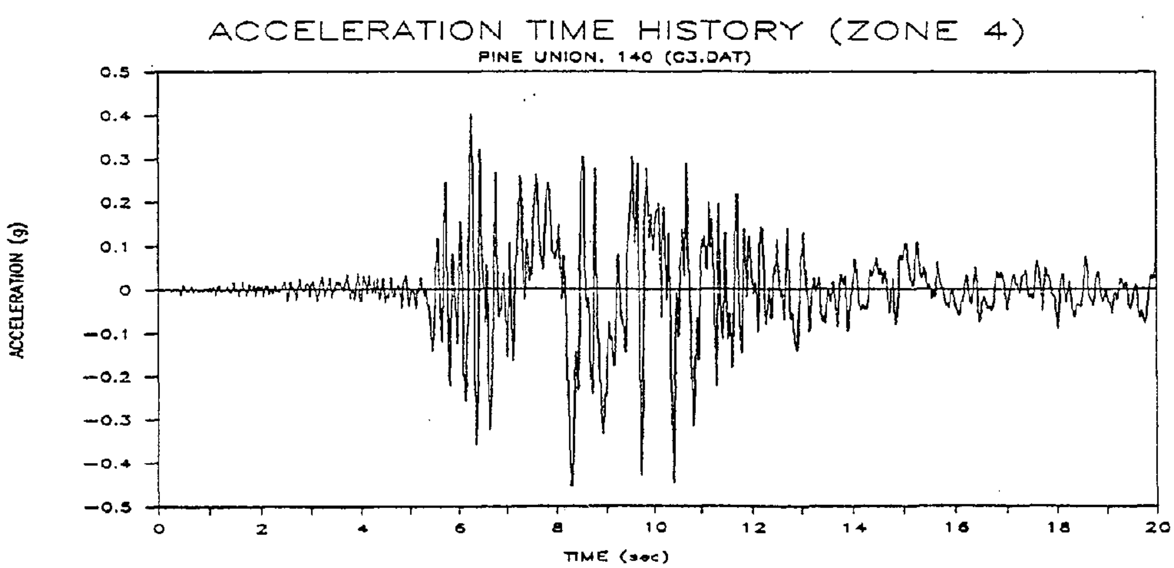
Earthquake Ground Motions

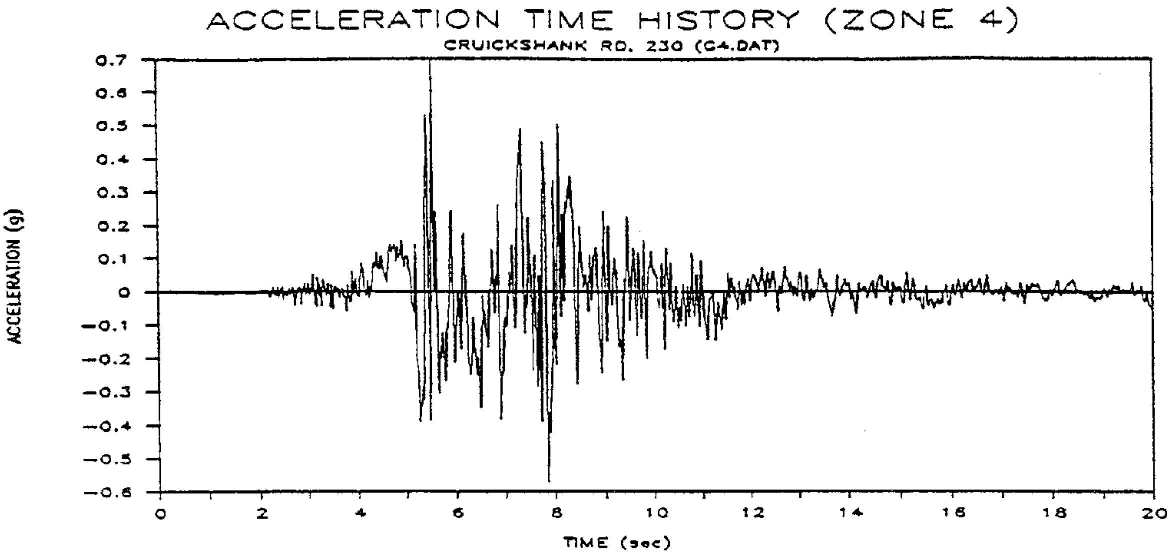
Earthquake	Δt (sec)	Duration (sec)	C_S [1]	C_1 [2]	C_2 [3]	Designation
El Centro, E-W	0.02	53.0	0.03937	0.9255	1.7875	G1.DAT
El Centro, N-S	0.02	53.0	0.03937	0.6777	1.3145	G2.DAT
Pine Union, 140	0.01	29.0	0.003937	0.8622	1.7067	G3.DAT
Cruickshank Rd., 230	0.01	34.0	0.003937	0.7632	1.4951	G4.DAT
James 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

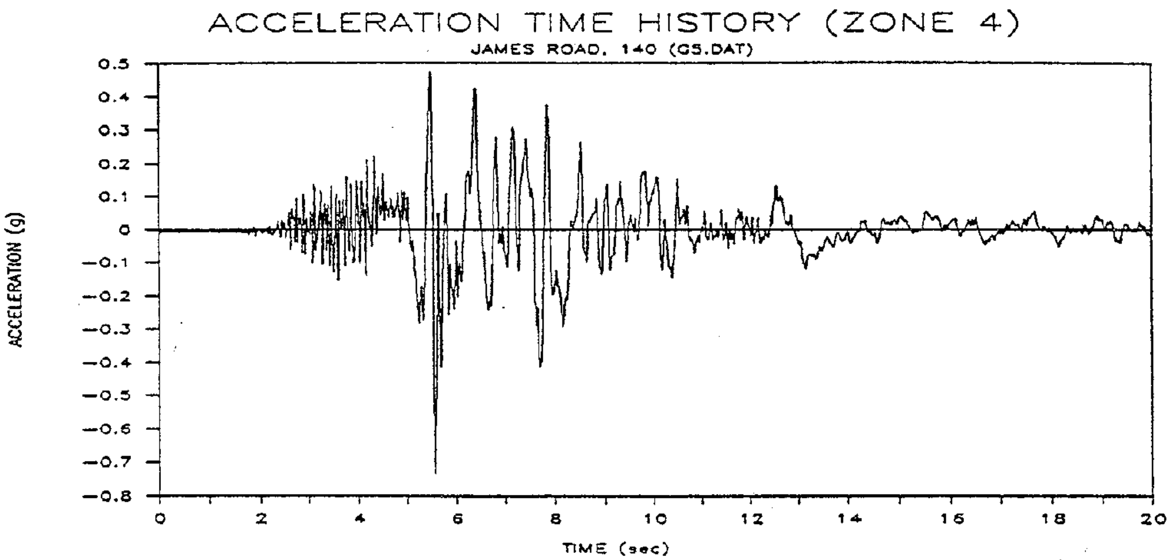
[1] C_S = Scaling factor for converting acceleration units to in/s/s.
[2] C_1 = Scaling factor for Seismic Zone 2
[3] C_2 = Scaling factor for Seismic Zone 4

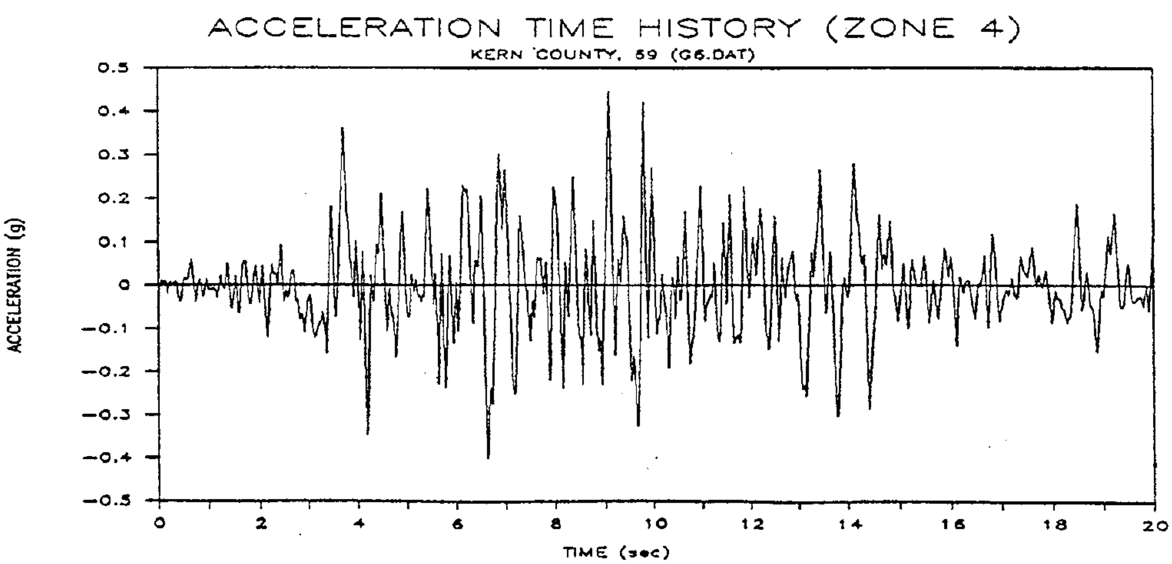

ACCELERATION TIME HISTORY (ZONE 4)
EL CENTRO, E-W COMPONENT (G1.DAT)
ACCELERATION (g)
TIME (sec)


ACCELERATION TIME HISTORY (ZONE 4)
EL CENTRO, N-S COMPONENT (G2.DAT)
ACCELERATION (g)
TIME (sec)

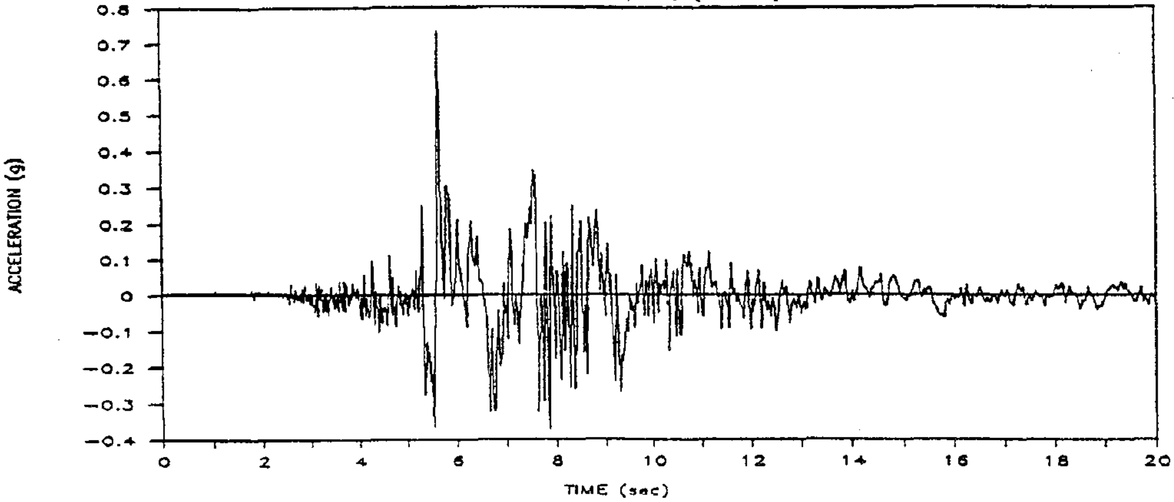

ACCELERATION TIME HISTORY (ZONE 4)
PINE UNION, 140 (G3.DAT)
ACCELERATION (g)
TIME (sec)


ACCELERATION TIME HISTORY (ZONE 4)
CRUICKSHANK RD. 230 (G4.DAT)
ACCELERATION (g)
TIME (sec)

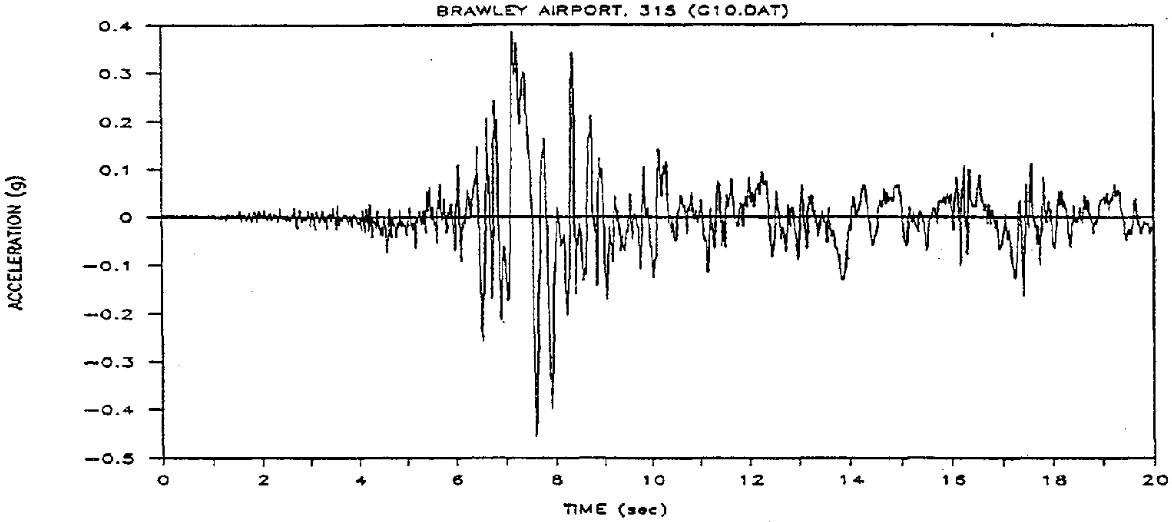

ACCELERATION TIME HISTORY (ZONE 4)
JAMES ROAD, 140 (G5.DAT)
ACCELERATION (g)
TIME (sec)


ACCELERATION TIME HISTORY (ZONE 4)
KERN COUNTY, 59 (G6.DAT)
ACCELERATION (g)
TIME (sec)

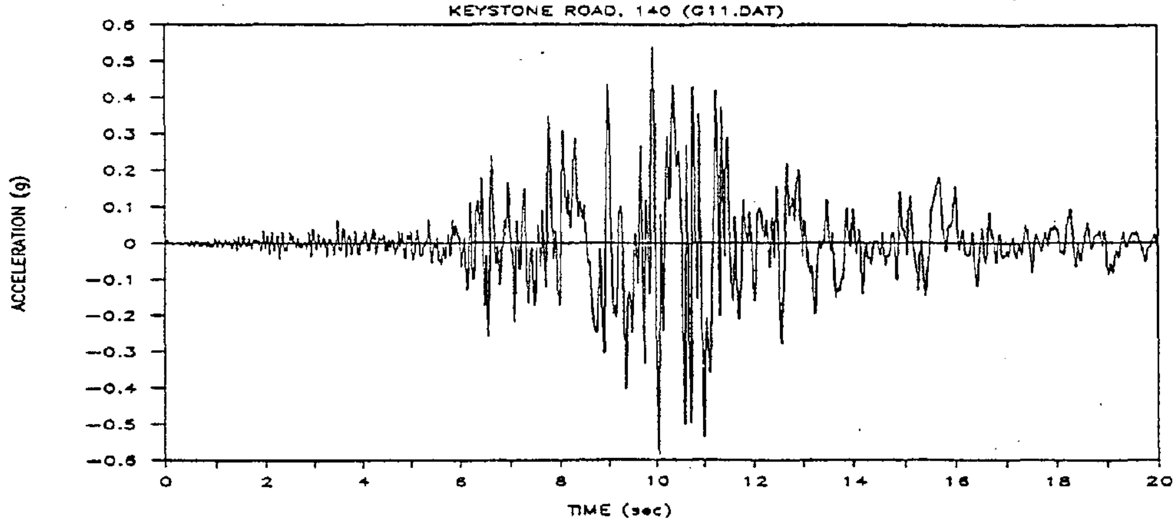
ACCELERATION TIME HISTORY (ZONE 4)
CRUICKSHANK RD, 140 (G9.DAT)



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:

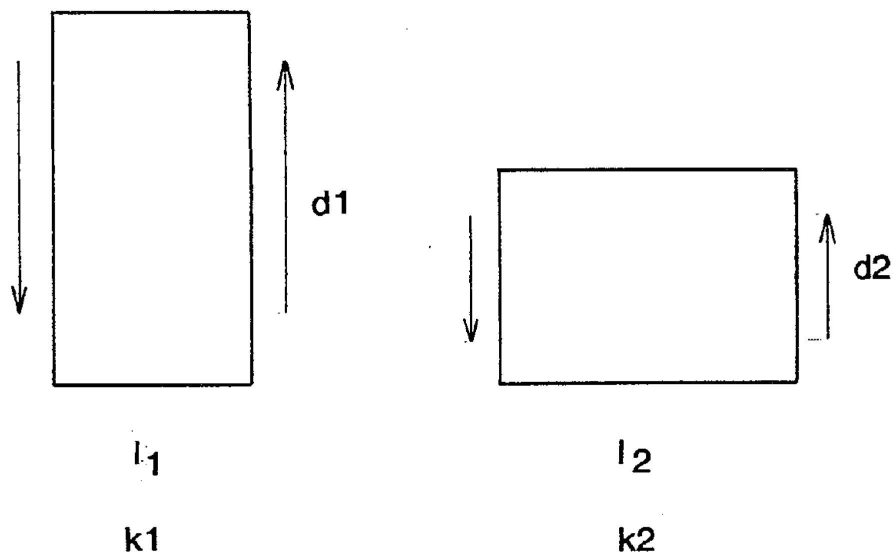
1. Type N - 1/2" Plywood Deck, Blocked and Chorded

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